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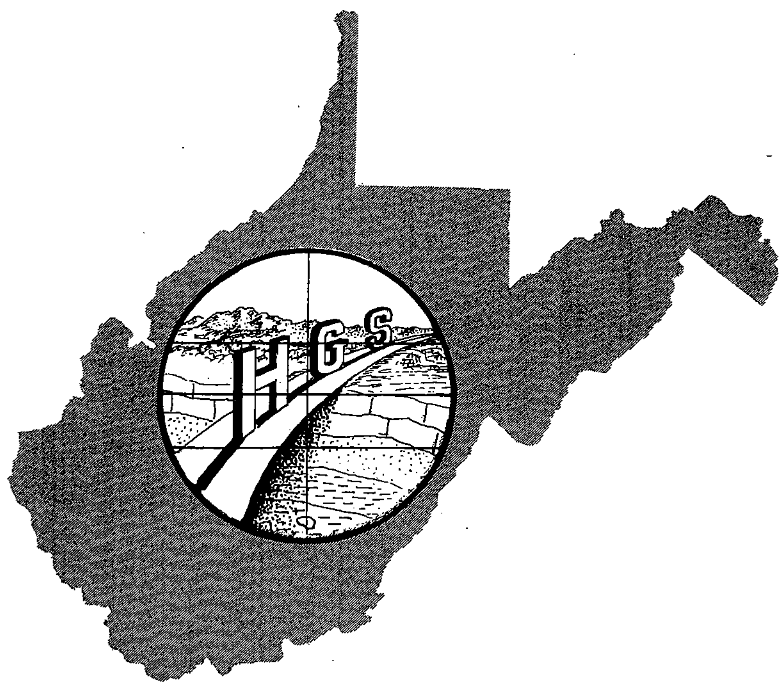

STATE OF WEST VIRGINIA
Hulett C. Smith, Governor
WEST VIRGINIA GEOLOGICAL AND ECONOMIC SURVEY
Paul H. Price, Director and State Geologist

PROCEEDINGS OF THE 19TH ANNUAL
HIGHWAY GEOLOGY SYMPOSIUM

The Towers

West Virginia University
Morgantown, West Virginia

May 16-17, 1968

HOSTS

West Virginia Geological and Economic Survey
State Road Commission of West Virginia
West Virginia University Department of Geology
West Virginia University College of Engineering

EDITOR

Robert B. Erwin
West Virginia Geological and Economic Survey

December 1968

FOREWORD

Perhaps no other State presents the number and variety of examples of geologic problems encountered in highway construction as does West Virginia.

Our great geologic column, embracing some 30,000 feet of exposed rocks consisting essentially of indurated shales, sandstones, limestones and clays, is crossed by many State and Federal highways. These, together with extensive unconsolidated stream deposits in all our river valleys, and our unusually rugged topography, offer many examples of construction and design problems for visiting geologists and engineers from the other states.

It was, of course, a great disappointment to me, a member of the National Steering Committee for the Highway Geology Symposium and Chairman ex-officio of the local committee not to be able to attend, because of a temporary incapacity. I was indeed pleased, however, to be informed by many of those in attendance, that it was one of our best Highway Geology Symposium meetings.

The sponsoring agencies, in addition to the State Geological Survey were the State Road Commission of West Virginia, and the West Virginia University Department of Geology and the College of Engineering. The program committee was composed of Dr. Robert B. Erwin of the Geological Survey, as Chairman, Mr. Berke L. Thompson of the State Road Commission, Dr. Arthur E. Burford of the Department of Geology and Dean Chester A. Arents of the College of Engineering.

Special thanks are due to members of this committee, who devoted long hours in arranging for the many fine papers and field trips following the regular program.

I want, especially, to thank my long time friend and colleague Norman F. Williams, State Geologist of Arkansas for his delightful contribution to the meeting as the Banquet Speaker. The fine social hour, under the direction of Ernie McKay, Manager and Owner of the Kingwood Inn, was made possible by liberal contributions from the Engineering, Construction and Supply firms who have varied interests in highway construction in West Virginia.

A great deal of the work and worry, usually connected with a meeting such as this, was handled by the Conference Office of the University under the supervision of Mr. Robert B. Conner and Mr. Samuel A. Agnew, III. Also, our thanks are due to Mr. Harold J. Shamburger, representing the University administration, and Mr. Joseph S. Jones, representing the State Road Commission, for welcoming the group to our State and Campus.

Special thanks to Mr. A. Carter Dodson, Chairman of the National Steering Committee of the Highway Geology Symposium, for presiding at one of the technical sessions and for keeping the Symposium going year after year.

Paul H. Price, Director
and State Geologist
Chairman Ex-Officio

19th Annual Meeting, May 16-17, 1968
HIGHWAY GEOLOGY SYMPOSIUM
Box 879
Morgantown, West Virginia 26505

EDITOR'S PREFACE

The 19th Annual Highway Geology Symposium was held in the Towers at West Virginia University on May 16 and 17, 1968. The technical session was planned to bring out general problems of road building in mountainous areas, whereas the field trip was concerned more specifically with construction problems in areas of incompetent bed rock, unconsolidated deep glacial lake fill, and mined and unmined coal seams lying above, below, and at or near road grade.

The first day of the meeting consisted of two technical sessions, each broken midway by an extended coffee break and equipment demonstration. A total of nine papers were given and all are represented here in the order in which they were presented. In some instances, it was necessary to leave out certain illustrations used in the oral presentation because of technical difficulties.

At the Thursday evening banquet Mr. Norman F. Williams, State Geologist of Arkansas, spoke on "The Ecology of the Eocene Frigate Bird ! " His remarks were extended also to include some interesting insight into the ecology of a few modern rare birds.

As Program Chairman and Editor I want to express my thanks to all who made the meeting such a success, Mr. Berke L. Thompson of the State Road Commission of West Virginia and Dr. Arthur E. Burford of West Virginia University who served with me on the Program Committee, and especially to the authors for their contributions to the program.

Robert B. Erwin, Editor
Assistant State Geologist
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GEOLOGY OF WEST VIRGINIA WITH SPECIAL REFERENCE TO THE SYMPOSIUM FIELD TRIP AREA

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ABSTRACT

West Virginia is a "show case" of Appalachian geology for earth scientists. A trip from the Ohio River eastward to historic Harpers Ferry traverses the following three physiographic provinces respectively: Plateau, Valley and Ridge, and Blue Ridge. The Paleozoic sedimentary rocks are increasingly deformed toward the east. Morgantown is located in the Plateau province and the geology of this area is characterized by gentle folds, nearly vertical joint sets, and a predominance of shales and sandstones.

The rocks of the Morgantown area were deposited as sediments during the Pennsylvanian Period, approximately 325 million years ago. A shallow inland sea inundated the central interior of the United States. Rivers flowed northward and westward across West Virginia building deltas along coastal plains. The variety of rocks viewed in the Morgantown and Fairmont areas (Field Trip) reflect the different depositional environments. Sandstones mark the position and trend of ancient rivers; the red shales and thin siltstone layers represent former flood plains and natural levees; the gray to black shales and coal-bearing plant materials formed in poorly drained marshes and swamps; the limestones lacking fossils except for ostracods, worm burrows, and occasional snails developed in fresh-water lakes and brackish bays; and the shales and limestones containing marine fossils were deposited as sediments synchronously over a wide regional area as is occurring in the Mississippi delta along the Gulf of

Mexico today. The occurrence of different rock types stacked vertically, one above the other, suggests that these same depositional environments shifted laterally with time.

The sediments were transformed into rocks when they were buried by additional sediments and subsequently uplifted and slightly flexed during mountain building about 300 to 200 million years ago. Streams flowing across the ancestral Appalachian Mountains carved out our present hilly landscape. In relatively recent times, less than 1 million years ago, glaciers blocked the northward flow of the Monongahela River near Beaver Falls, Pennsylvania, causing a natural lake to develop. Varved clays and sands accumulated in this lake. Removal of the ice dam and diversion of the Monongahela River into the present Ohio River resulted in deepening the river valley. River terraces and a small, discontinuous flood plain are relatively modern additions to the area by the Monongahela River.

GEOLOGIC AND ENGINEERING IMPLICATIONS OF POOR QUALITY SANDSTONES FROM SIMULATED HIGHWAY TESTS

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This paper will summarize briefly a ten year study of sandstone by the Engineering Experiment Station of West Virginia University for the State Road Commission and with the cooperation of the Bureau of Public Roads. The work was begun because of a lack of satisfaction by the Commission with the Los Angeles Abrasion Test as a means of accepting or rejecting a given stone for base course construction. Highways had been constructed with sandstone which had not passed the Los Angeles Test, yet had given good performance over the years. And, other highways, constructed with sandstone passing the test, had given poor performance.

Compressive and shear strength tests made early in the project on solid stone gave values of several thousand pounds per square inch, with little spread between good and poor stone. Compressive tests on crushed stone confined in cylinders also gave strength values many many times greater than those which would ever occur in a highway.

Concurrently with this early testing, graduate students in the department of geology began to assist with location studies of state sandstones and with petrographic examination of many samples with a view toward their suitability for highway use. Their examination of thin sections provided the project with considerable insight into the effect of content, grain size, types of cement, grain contacts, porosity, etc. John Dean worked especially with argillaceous sandstones, James Luzier with calcareous and Robert Yedlosky with all types, but particularly high silica. All worked under the supervision of Dr. M. T. Heald. They furnished beautiful micrographs of typical stones showing: the wonderful uniformity (for a naturally occurring material) of high silica stone; the more varied appearance of the calcareous, particularly those with "floating" texture; and the most inhomogeneous appearance of the argillaceous. The silica, hard and brittle, inclined to break into thin chips and had low friction at surfaces making it difficult to compact. At the same time, it usually weathered much less than other types. The calcareous, softer, was more absorptive of impact, yet apt to leach out and lose cement in slightly acid waters if the calcareous content was too low. The argillaceous was frequently non-uniform, often poorly cemented, soft, perhaps porous, and inclined to weather rapidly. This kind of insight makes a knowledge of the type of cement in a stone extremely valuable. The overall understanding of sandstones contributed by these geologists was of great value to the project.

It was felt that a compressive test of crushed, graded stone was needed wherein stress levels occurring in base courses would be applied and produce deformations comparable with those in highways. This brought up the thought of the triaxial test, but its rubber membrane is most awkward with granular materials. Metal cylindrical sectors were tried, held together with special springs which provided about 10 psi lateral pressure on a cylindrically shaped sample even though considerable deflection occurred. A static test using this partial containment gave ultimate stresses 2 to 3 times those of the regular triaxial test, and 200 to 300 psi was common, still much higher than the stress in base courses. Finally, the same container was used with a repeated load machine which more closely approximated highway experience. Stresses of the order of 40 to 2000 psi, with repeated application of load, gave increasingly more deformation. While this machine was not practical for use as an acceptance test, it did give valuable information about the importance of keeping moisture out of crushed stone if it is to resist repeated load successfully, etc. Extensive tests with this machine compared stabilizing materials for crushed stone. They indicated that bituminous stabilizers do not materially increase load carrying capacity; they may even appear to "lubricate" stone particles in an undesirable way. The best of these appear to hold the particles together better, assist in elastic rebound after load removal, and provide waterproofing, all quite important to good base course functioning.

Lacking good information as to actual highway performance of base courses under heavy traffic, the special machine shown in Fig. 1 was designed and built to more nearly simulate this action. This Rolling Load Testing Machine is a single wheeled, 40 load passes per minute, reciprocating loading device which employs a single 10.00 x 20 liquid filled truck tire inflated to 80 to 100 psi pressure. The wheel loads used in this testing ranged from 7100 to 11,500 pounds although the machine is capable of applying 20,000 pounds to the specimen. Some other features of the machine include a roller and a screw drive system for moving the specimen under the wheel in a very slow lateral motion in order to reduce rutting of the surface. The motion of the specimen was adjustable and was programmed in the last phase of testing.

The highway specimens tested in this investigation were quite massive specimens, 4½ x 5½ x 3 feet deep, and weighed approximately 6 tons. The base course, surface thickness, and aggregate size were of the same general dimensions as equivalent courses found in actual highways. Many measurements were made during construction and testing. Those of interest here are measurements made during testing which consisted of an automatic total of load passes and intermittent surface and interface residual deflections.

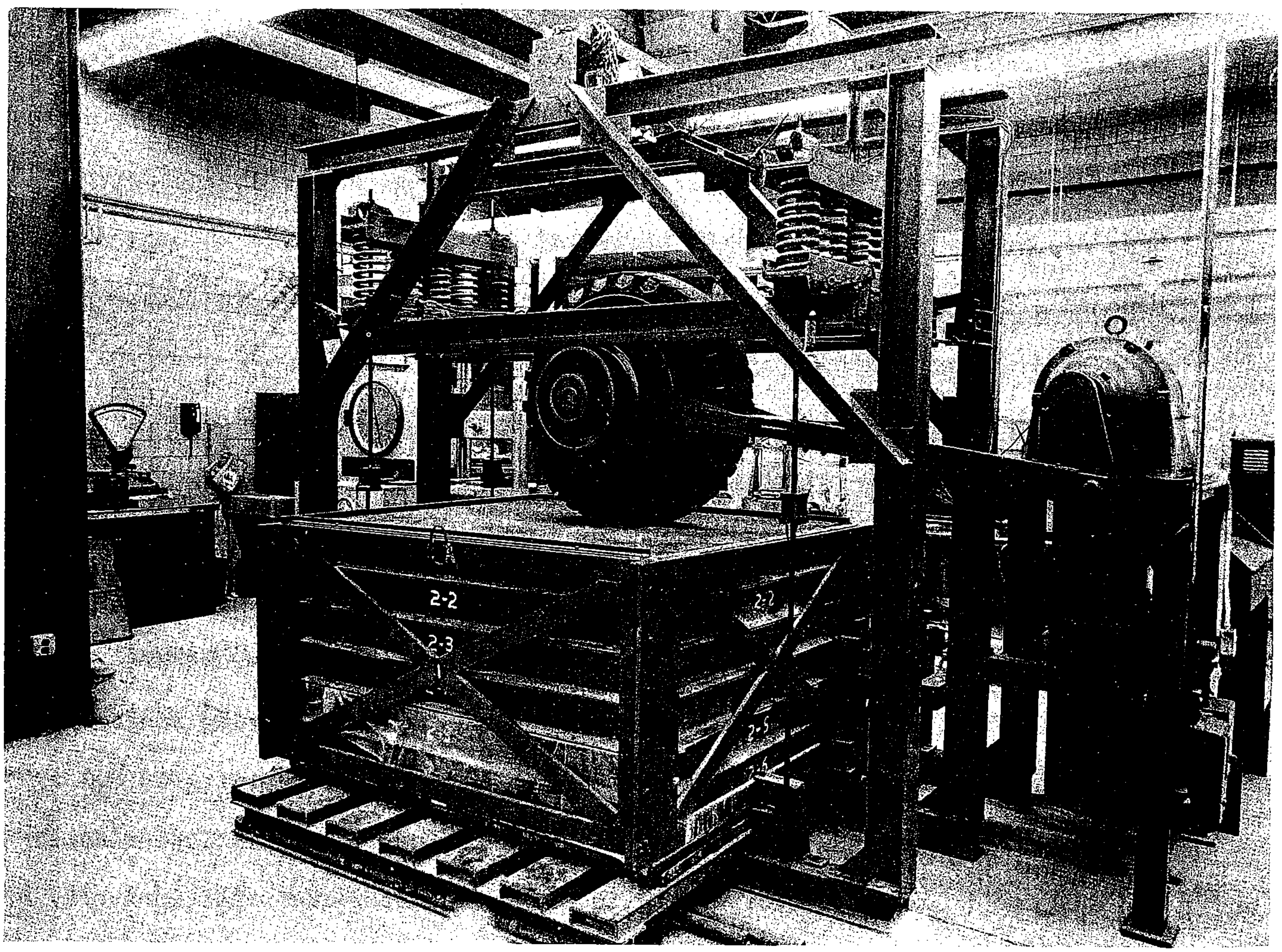

Figure 1. Rolling Load Testing Machine

The surface deflections were measured by hand while the interface motions were detected by linear variable differential transformers which were buried in the cell as it was constructed. These devices were calibrated and read out through a voltmeter. Many tests were performed after testing, some of which were overall deflection contours, moisture tests, and gradations.

A total of 106 highway specimens, divided into four phases, have been tested. Phase I was directed toward determining the minimum thickness of a 65 L. A. sandstone which would effectively spread an 11,500 pound wheel load to the subgrade. In this phase it was found that 9 inches of this material, compacted to its standard AASHO T-99 Method D density, would keep subgrade motion negligibly small. Moreover, it was found that additional thickness did not improve the performance of the specimen.

Phase II was an investigation into the performance of calcareous, agrillaceous and high silica sandstone, each having L. A. values of approximately 67, 77, and 87 percent. These materials were used in a 9-inch base thickness and were subjected to a load of 7100 pounds. Results of this phase show that the L. A. test did a reasonable job in rating calcareous and siliceous materials as to their load carrying characteristics. The argillaceous material gave poor performance regardless of its L. A. value. For a given L. A. value, the calcareous sandstone tends to show the best performance although it is frequently difficult to distinguish between it and the siliceous but both perform much better than argillaceous material, as shown in Fig. 2.

Phase III testing was for the evaluation of various stabilizing agents when employed with an 85 L. A. argillaceous sandstone. This material was chosen because it usually requires stabilization and does occur in large quantities in areas where higher quality stone is not locally available. The stabilizing agents were 85-100 pen hot mix asphalt, SS-1 slow set emulsion, portland cement and untreated limestone in place of treated sandstone. These treated bases were tested in 4- and 6-inch depths with the remainder of an 11-inch base being untreated sandstone. Results show that portland cement yields the most stable base in that we were not able to fail such a base with an

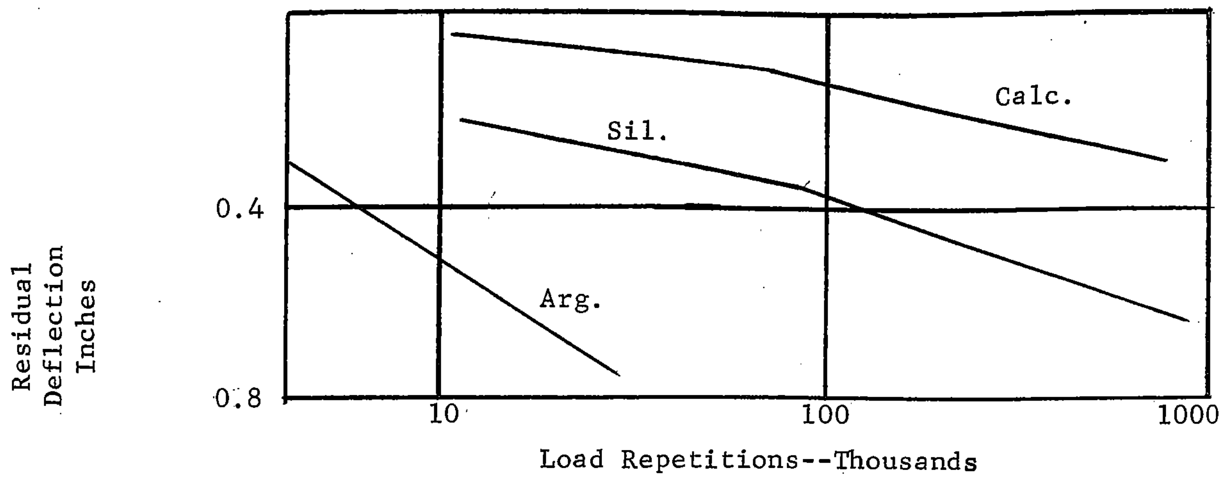

Figure 2a. Comparative Sandstone-Asphalt Interface Deflections for the different type sandstones, all having a Los Angeles Wear of approximately 69 percent.

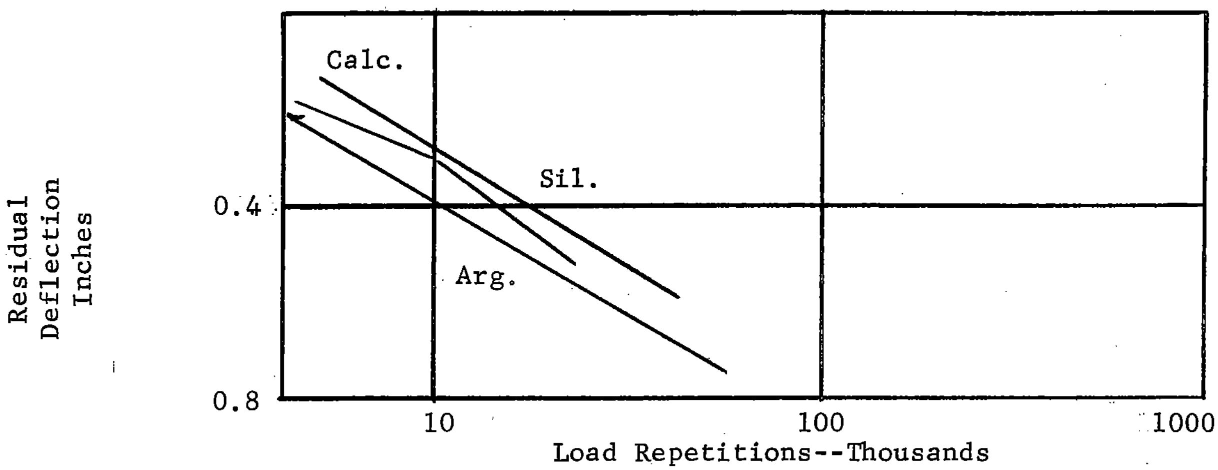

Figure 2b. Comparative Sandstone-Asphalt Interface Deflections for the different type Sandstones, all having a Los Angeles Wear of approximately 79 percent.

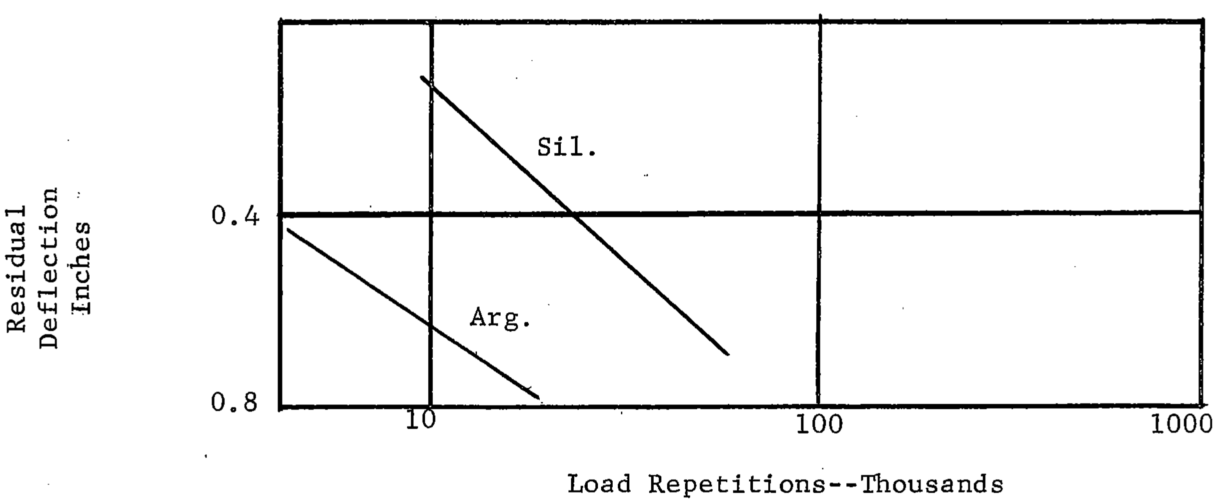

Figure 2c. Comparative Sandstone-Asphalt Interface Deflections for the different type sandstones, all having a Los Angeles Wear of approximately 89 percent.

11,500 pound wheel load. The remaining treating materials show little improvement over untreated bases when initial deflections are eliminated. It was difficult to get a complete cure of SS-1 emulsion. It is believed that the ability of these materials to assist stone in resisting weathering is most important. The weather resisting effects of the stabilizing agents employed were not investigated.

Phase IV was an investigation of the effect of controlled gradation on specimen performance. The base material used in this phase was a 65 L. A. argillaceous sandstone and was subjected to a 9000 pound wheel load. Base gradation, in these tests were all within State Road Commission limits but were either fine, coarse or step and gapped. Results of this phase show, in general, that the effects of gradation of a base course can be masked by moisture and density variations even though all three of these conditions are interrelated. This result was unexpected to say the least since control of gradation (straight line on the sieve size to the 0.45 power) had reduced data scatter in previous phases.

A computer analysis was attempted on all of the Rolling Load Machine data in the form of a multiple linear regression analysis. Three regression equations were realized from this work. They employed the slope of the deflection-log repetition curve as the dependent variable and test or construction parameters as independent variables. The most evident omission of this type, especially in Phase II, where it has been shown that for any given L. A. range, the different types of sandstone perform differently. Although some statistical insight was gained from the computer analysis, the most concrete findings came directly from analysis and study of test data in the form of gradation and performance curves.

Conclusions reached due to this testing include: ASSHO T-99 tests for aggregate indicated excessively high moistures; the inability of the L. A. test to determine or predict the performance of a base course; the profound effect of stabilization on the load carrying capabilities of a sandstone.

In summary, we must admit we found no test better than the Los Angeles Test. And, we did not run a series of tests with the Rolling Load Machine using what we now consider the most important single variable, the moisture content of the base course.

In general, we believe that most sandstones have the strength necessary to resist traffic loads in a well-designed, well-constructed highway. We believe the most critical characteristic of a sandstone for highway use is the ability of that stone to resist weathering. But we made no tests and have no direct proof of this statement.

We are proud of the performance of the two unique machines developed in the project--the modified triaxial repeated load machine and the heavy duty Rolling Load Machine. The latter has been purchased by the Civil Engineering Department and will be used in future testing of highway materials.

THE PROBLEM AND CORRECTION OF LANDSLIDES IN WEST VIRGINIA

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INTRODUCTION

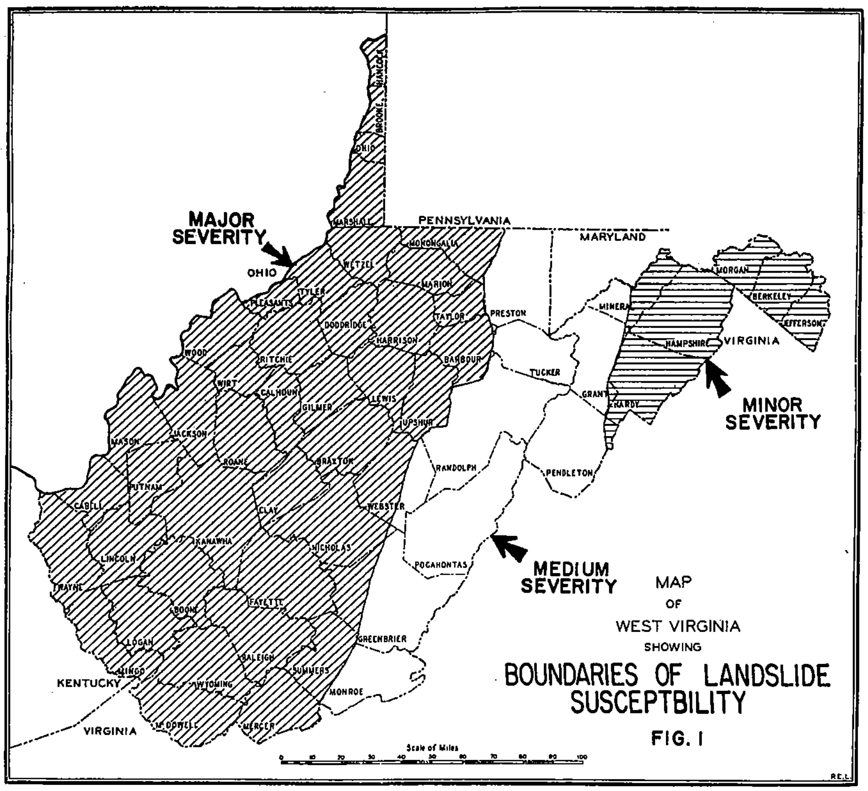
Landslides are a major problem in the West Virginia Highway Program, due to the high susceptibility of the soil to sliding. The types of slides encountered vary throughout the range of the classifications. Therefore, emphasis must be focused on the prevention aspects of the problem.

Probably the most valuable tool necessary for the prevention of landslides is the recognition of the problem. However, economics and time schedules limit the scope of the boring and testing programs for a particular project. Thus, it is necessary to recognize potential problem areas prior to and during the site investigation. One phase of problem recognition consists of correlating the proposed construction features with similar features that have precipipated slides in the past. A prime example would be the case of a side hill fill situation. It has been found that side hill fills greater than approximately twenty (20) feet, placed on the natural overburden will result in an embankment failure, regardless of the safety factor obtained in the stability calculations. Although it is a simple matter to recognize this situation, other conditions are more complex and require greater scrutiny to be recognized as a potential problem.

Therefore, the purpose of this paper is to present the landslide problem in West Virginia in terms of susceptibility, types of failures and general case histories, which may be of assistance in the phase of problem-recognition, when compared to problems in other areas.

LANDSLIDE SUSCEPTIBILITY

West Virginia is divided into three areas of Landslide Susceptibility based on highway occurrences. The three areas are rated as Minor Severity, Medium Severity and Major Severity as shown in Figure 1. In respect to a nationwide comparison, West Virginia has been rated in the category of Major Severity with the exception of the area of Minor Severity in Figure 1, which was rated nationally as being of the severity classification of medium.



MAP OF WEST VIRGINIA SHOWING BOUNDARIES OF LANDSLIDE SUSCEPTBILITY

FIG. I

THE GENERAL TYPES OF LANDSLIDES ENCOUNTERED IN WEST VIRGINIA

CATEGORIES

Soil Cut Slopes - The soil cut slope can be separated into four categories of usage: (1) The normal cut slope from the roadway ditch; (2) The soil slope at the top of a rock cut where the intersection of the original ground line is made; (3) The transition slope from cut to fill at each end of a rock cut and (4) The soil slopes required for channel relocations.

Soil Slopes From the Roadway Ditch - The common soil cut from the roadway ditch presents a problem primarily in the area of ancient lake deposits. Here the overburden depths are greater and the performance of the varved clays encountered are less predictable than the residual soils. A section of Interstate Route-64, between Charleston, West Virginia and Huntington, West Virginia, passes through deep deposits resulting from the ancient Teays Lake. Many slope stability problems were encountered in the design and construction phases in this area.

Interstate Route-64 in the area under discussion has been open to traffic for almost ten (10) years. The slopes in general are performing satisfactorily. However, landslides have occurred at several locations along the roadway. The majority of the slides were small pocket failures and were corrected by state maintenance crews. Presented here are two slides of the several failures in soil cut slopes of major proportion which have occurred requiring analysis.

One of the major slides occurred in a 2:1 cut slope which affected a high pressure gas line and approached a large two-story residence near the right of way limits, at the head of the slide. Lake deposits were discovered behind as well as in front of the dwelling. Several methods of correction were investigated for this slide. The cost of each method for retaining the movement was considerable. Therefore, in lieu of retaining the movement a ditch line was re-established at the toe of the slide and a bench provided for accumulation of future movement. When the bench fills, and ditch drainage is impaired, additional material is removed. The rate of movement has been an average of approximately 2000 cubic yards per year over the past four years. However, it appears the rate is decreasing at present.

A second slide of major proportion occurred in a 15' cut on 2:1 slopes, consisting of lake deposits. The natural ground behind the cut was laying on approximately a 5:1 slope. After the highway was open to traffic, tension cracks appeared at a distance of 200 feet from the ditch line. Shortly thereafter displacements were noticed in the ditch line and in the macadam shoulder at roadway level.

Here again the cost of methods for retaining the movement was considerable. A redesign with flatter slopes and a benching system was considered, however, the possibility of removing additional lateral support and unleashing the entire hill side made this method undesirable. Therefore, similar maintenance procedures were employed for this slide as in the aforementioned problem.

Top of Cut Soil Slopes - The soil slopes at the top of a rock cut which makes the intersection with the original ground can cause problems usually when the soil overburden is greater than approximately ten feet, with the slope of the natural ground steeper than approximately 3:1. The usual design procedure for this type slope consists of providing a bench (usually 10' wide) at the soil-rock contact and providing a slope with the ratio based on the strength and conditions of the soil. The slope usually found satisfactory for this condition consists of a 2:1 ratio.

Although the 2:1 slope performs satisfactorily in general, failures still occur during construction in areas of concentrated ground water. The failures are usually small and the slopes are cleaned and reshaped during construction. Very few slides of this type occur during the maintainence phase of the program and those that do are contained on the benches.

Transition Slope - The transition slope at each end of a rock cut is usually in soil. In general there is very little boring information in this area which results in the design of slopes without sufficient data and results in problems during and after construction. The roadway side ditches in this area are designed for flow approaching their maximum capacity. Therefore, when material from a failure encroaches upon the ditch affecting the surface drainage, other problems are precipitated in the shoulder and pavement areas.

The trend at present is to use conservative slope ratios in the transition areas. The height of cut is usually small and the flatter slopes do not noticeably increase the earthwork quantities.

Channel Relocation Slope - The side slopes of a channel relocation can present a tremendous problem when located in conjested areas, at the toe of a natural hillside or too close to the toe of an embankment. Each condition requires a special investigation and analysis to prevent landslides in conjunction with this type construction.

An example of a failure in a channel change slope, is near Wheeling, West Virginia on Interstate 70 in the vicinity of the east portal to the Wheeling Tunnel. The channel relocation was constructed at the toe of, and parallel to, a railroad embankment. A dual structure carrying I-70 crossed at this location, and piers were located within and adjacent to the problem area. There were several factors that contributed to this failure which occurred during construction. The factor which triggered the movement, however, was believed to be a rapid draw down situation created by the receding flood waters of the channel. Concrete had not been poured for the piers in the area. However, excavation for one of the piers at the toe of the slope had begun further complicating the situation. The mainline track of the railroad was located within the active slide area for a distance of 800 feet. Railroad crews were on the site twenty-four hours a day adding ballast and realigning the track in order to maintain rail traffic. This operation had to be continued until corrective measures were applied and the movement ceased.

The correction consisted of removing the material that had failed and reconstructing the railroad embankment with rock. The excavation and rock backfill operations were carried out in 50 foot sections to prevent the possibility of losing the entire excavation back slope in one massive movement.

ROCK CUT SLOPE

Inclined Bedding Planes - The major problem encountered as a result of inclined bedding is due to local geological structure. The design of a rock cut, (as based on the boring information as well as outcrops and all other sources of information) can perform satisfactorily for the majority of a cut. However, when the presence of detrimental local structure cannot be discovered until the cut is made, failures have occurred.

A failure which occurred during construction on Interstate Route 77 was located in the southern portion of the state, between Princeton and Bluefield, West Virginia. The cut slope is in the over turned beds resulting from the St. Clair fault. The bedding in the majority of the cut dipped into the hill and away from

from the roadway as anticipated. However, approaching the end of the cut the bedding changed from dipping into the hill going through a section of horizontal bedding and then into a section where the bedding dipped toward the roadway. The complexity of the changing conditions was not detected in the boring program, due to the relativity short distance through which the change took place. The extreme dip on both ends is approximately 70 degrees. A failure occurred as soon as the bedding planes were undercut in the area where the dip is toward the roadway. Large sheets of rock fell making hazardous working conditions for reshaping the slope. Shortly thereafter large tension cracks were discovered in the natural ground above the slope where the dip was into the hillside. The failure in this area was occurring perpendicular to the bedding and along the cleavage. The area between the failures where the bedding approached the horizontal did not experience any movement.

The correction for this failure consisted of a redesign of the slope which paralleled the bedding where the rock dipped toward the roadway and paralleled the cleavage where the dip was into the hill resulting in approximately equal slope ratios. This same slope ratio was carried through the area of horizontal bedding between the aforementioned areas. The actual slope ratio was determined by field control based on the conditions encountered in the excavation.

Excessive Slope Ratio - At present, in West Virginia, the slope ratio for a given classification of rock is based on performance observations of existing cut slopes. However, when core samples are not representative of the major portion of the rock to be encountered in a cut, the result is excessive slope ratios. The shale formations are the primary source of the problem when this condition exists.

The failures that occur due to a slope ratio that is too steep for a given shale formation, tend to establish a slope which restores equilibrium (in respect to the existing stress). The weathering process, however, continues to further flatten the slope and produce additional sloughage. The benches used in the design of slopes in West Virginia provide a place for this material to accumulate at successive levels on the slope prior to reaching the drainage ditch. The bench also provides a near horizontal surface for the loose weathered material to accumulate, which produces negligible driving forces at the contact, between the inplace rock in the bench and the loose accumulation of weathered material. Where straignt slopes with satisfactory ratios (in respect to the existing stress) have been used, the weathering process has produced material that accumulates in the drainage ditch requiring continuous maintenance. The sloughage on the flatter straight slopes is a result of shallow soil creep, due to the driving forces produced along the inclined contact surface between the inplace and the weathered shale. This surface is established at or in the weathering transition zone, at the point where the permeability of the material creates a concentration of water. Whereas with the bench design, the slope between the ditch line and the first bench is usually limited to a vertical rise of five feet which produces negligible sloughage.

When the slope ratio is too steep and results in a failure during construction and the material fills the benches, the slope is generally reshaped using flatter slopes between benches. After the roadway is open to traffic failures are very rare where the failed material reaches the roadway. However, the normal weathering in time causes sloughage which practically fills the benches.

From observations of cut slopes which have existed for a number of years, it appears that approximately one-third of an individual bench, along with a portion of the underlying slope weathers and sloughs off and accumulates on the next lower bench. This sloughage covers approximately 1/2 to 2/3 of the lower bench and provides some degree of protection from weathering for the lower portion of the underlying slope. This process occurs between successive benches throughout the height of the cut.

When the process of weathering is retracted to a point that is negligible, in respect to the service life of the highway, the slope contour resembles that of a natural hillside. The slope approaches a condition of equilibrium as the material weathers. Thus, sloughage occurs until a slope is established flat enough that the stretch of the weathered inplace material can withstand the forces tending to cause movement. The material that sloughs on to the next lower bench seeks equilibrium as it is accumulated. It is true that in time the weathering action will produce the same slope ratio that existed in the natural hillside. However, the bench widths are such that this condition should not occur until an extremely long time after the service life of the roadway has been terminated.

Overstressed Shale In Deep Cuts - In a rock excavation the newly exposed surface has no normal stress. Therefore, since the rock is under a condition of triaxial stress and strain, the rock begins to deform at first elastically and then permanently. These deformations are dependent on time. When the stresses are great and the rock, because of weak material, joints or other defects, cannot resist the stresses, localized failures will occur, causing a redistribution of the forces, which tends to restore equilibrium.[1]

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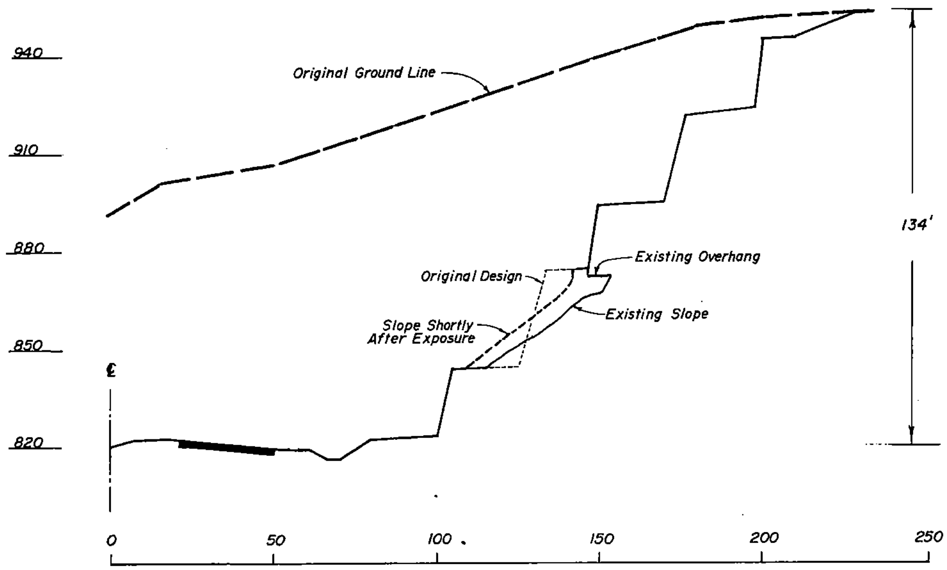
[1]Thomas A. Land, "Theory and Practice of Rock Reinforcement", Presented at the 45th Annual Meeting, Highway Research Board, January, 1966.

An example of a failure of a shale layer in the lower portion of a cut occurred on Interstate 64 near Nitro, West Virginia.

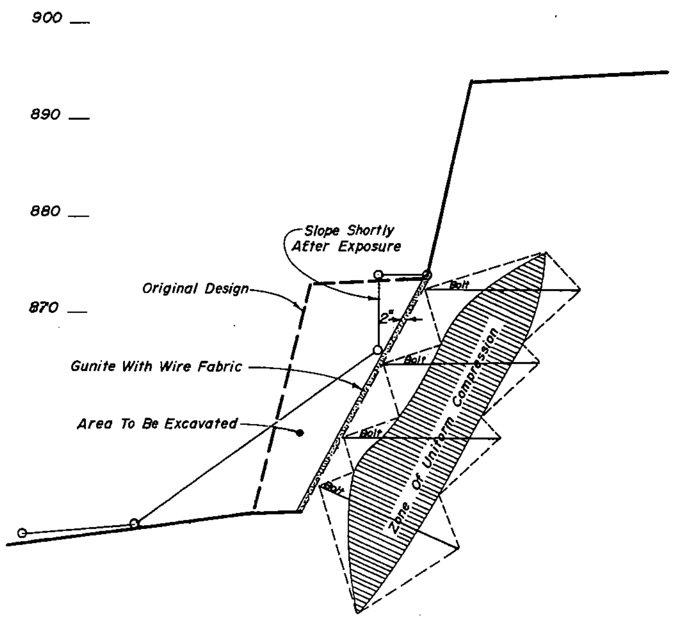
The failure occurred in a through cut with a maximum height of approximately one-hundred and thirty-five feet. The failure began during construction, shortly after the shale surface was exposed. Inspectors on the project reported that as cracks and breaks occurred in the shale, sounds comparable to a mild explosion echoed through the cut.

Figure 2 illustrates the cross section depicting the original design slope, the slope shortly after exposure and the existing condition. The contractor removed the debris that had fallen on the bench and reshaped the slope. At that time two methods of treatment were investigated. Redesign of the slopes of the entire cut was considered and involved approximately 117,300 cubic yards of excavation. Rock reinforcement techniques were also considered utilizing the beam or slab concept provided through the use of rock bolts. In conjunction with the rock bolts a gunite and wire fabric surface treatment for protection from weathering was proposed. The bolts were designed in a horizontal pattern, (Figure 3) when properly installed form a zone of uniform compression between the anchorage end of the bolt and the bearing plate at the head of the bolt. This zone of compression is compared to a beam or slab which acts at least partially as fixed at both ends. This is the feature of rock bolting that provides stability in the roofs of a mine excavation.

It was felt equilibrium had been restored in the shale layer shortly after the failure, however, this was not certain. Therefore, it was decided to observe the slope for a period of time since the corrections would still be applicable if the condition worsened. The benches would protect the roadway in the event of a large failure.



I-64 SLOPE FAILURE NEAR NITRO, W.VA.
FIG. 2



I-64 SLOPE FAILURE NEAR NITRO, W.VA.

FIG. 3

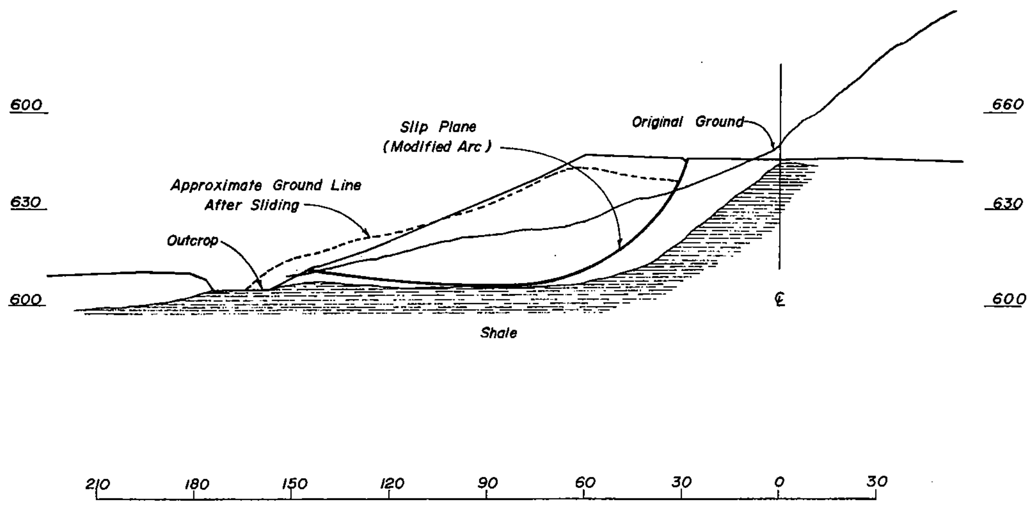
The condition at present is one of weathering of the shale layer which has produced an overhang condition as shown in Figure 3. The rock above the overhang would have to be removed in conjunction with either of the original corrective methods. Therefore, with the problem reduced to weathering it is now proposed that the stone above the overhang be removed and used to construct a rip-rap type slope protection for the shale layer.

Embankment Foundations - The early roadways built for the first automobile traffic in West Virginia were primarily built by a method which came to be known, locally at least, as the cut and cast method. The roadbeds constructed by this method consisted of excavating half of the roadway into the hillside and casting the material over the hill to form the other half in a sidehill fill condition. As the traffic requirements dictated, improvements in the geometrics produced larger fills in the sidehill condition, therefore, introducing the problem of embankment failures.

Practically one-hundred percent of the embankment failures occurring in West Virginia, in the maintenance of construction phases of the highway program, were constructed in a sidehill fill condition. Therefore, at this time a general policy has been adopted whereby the sidehill fills are constructed by first undercutting and then constructing fill benches into the rock prior to placing the embankment. The lower portion of the embankments re constructed with a granular material to provide drainage.

On some of the smaller sidehill fills, attempts have been made recently o construct them without providing fill benches. It appears that the maximum eightsof sidehill fill that can be constructed without fill benches are approxi-ately those less than 20 feet. However, this is only an average and each case epends on the conditions encountered at the individual sites.

An example of a failure of a side fill with a maximum height of 20 feet occurred on Interstate Route 77 near Parkersburg, West Virginia (Figure 4). The failure occurred shortly after the embankment reached subgrade elevation. The contractor excavated the slide, fill benches were cut into rock and granular material was used for back fill. This section of roadway has been open to traffic for approximately three years and the correction is performing satisfactorily.



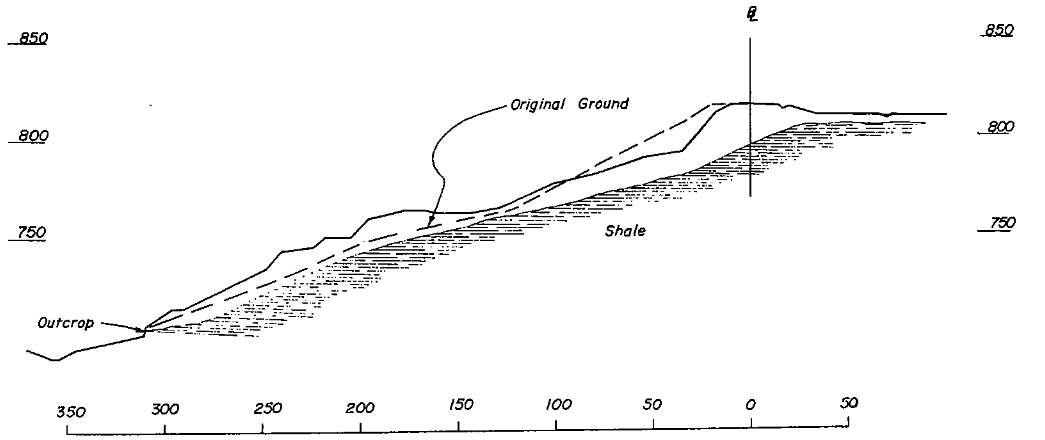
X-SECTION AFTER SLIDE

FIG. 4

CASE HISTORIES

SLIDE ON ACCESS ROAD TO TRI-STATE AIRPORT

In 1966 a slide occurred on the access road to Tri-State Airport located near Huntington, West Virginia. The fill had been constructed in a sidehill condition with shallow soil cover over a shale formation (Figure 5). The embankment



SLIDE ON ACCESS ROAD TO TRI-STATE AIRPORT
X-SECTION AFTER SLIDE
FIG. 5

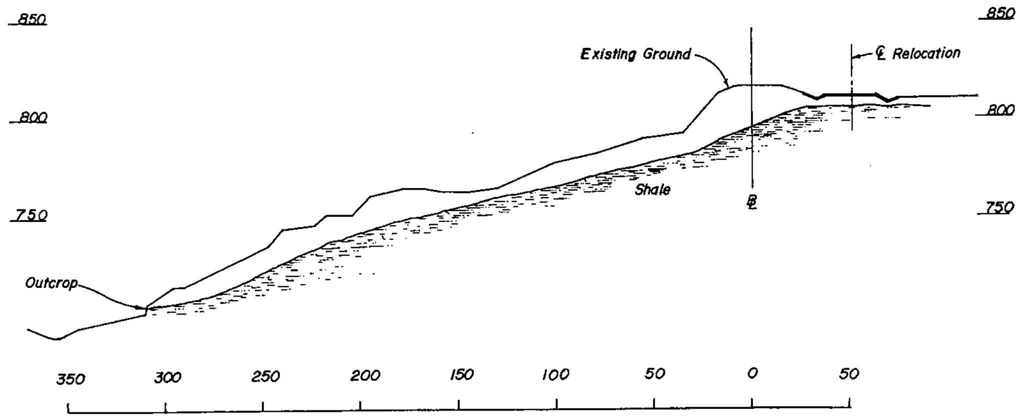
oundation soil and the soil in the natural ground slope below the toe of the mbankment became saturated. It appeared that small progressive failures ccurred in the natural slope until the embankment was affected and resulted n a condition resembling the type of landslide known as a flow.

After the slide a hazardous condition existed with one-fourth to one-alf of the pavement width dislocated for a distance of 125 feet parallel to he centerline. One-way traffic was maintained by utilizing the gravel houlder as part of the driving lane, until the situation could be corrected.

The correction of this slide consisted of revising the alignment and elocating the roadway away from the slide area. (Figure 6). This method of lide correction has been used successfully throughout the State where condi-ions encountered make it economically feasible, as was the case with this lide.

JSSET SLIDE

In 1963, a slide occurred on local service Route 7 in Calhoun County, ear Russet, West Virginia. It would appear that an embankment could not xist for any length of time, in a location as depicted in Figure 7. However,



CORRECTION
FIG. 6

this roadway has been in service for a number of decades, prior to the slide.

The length of service performed by the embankment was attributed to the granular soil which made up the foundation material as well as the embankment itself. The site investigation indicated that the sandstone layer near elevation 195*, sandwiched between two shale layers was contributing considerable water to the slide area. It was believed that through the years an accumulation of silt was increasing the permeability of the granular material. Therefore, the seepage forces reached a point in time, of being excessive and resulted in a failure (Figure 8).

In order to retain the slide and provide as much free drainage as possible a gabion wall was chosen as the method of correction.

The gabion wall consists of wire basket units filled with durable rock, (Figure 9) which when installed in a manner similar to that of a crib wall, form a free drainage retaining device. According to the manufacturer the wire baskets (Figure 10) which have a thick protective coating will provide long term durability. To date this installation is performing satisfactorily.

*This elevation was taken from a bench mark established just for the slide, with an assumed datum.

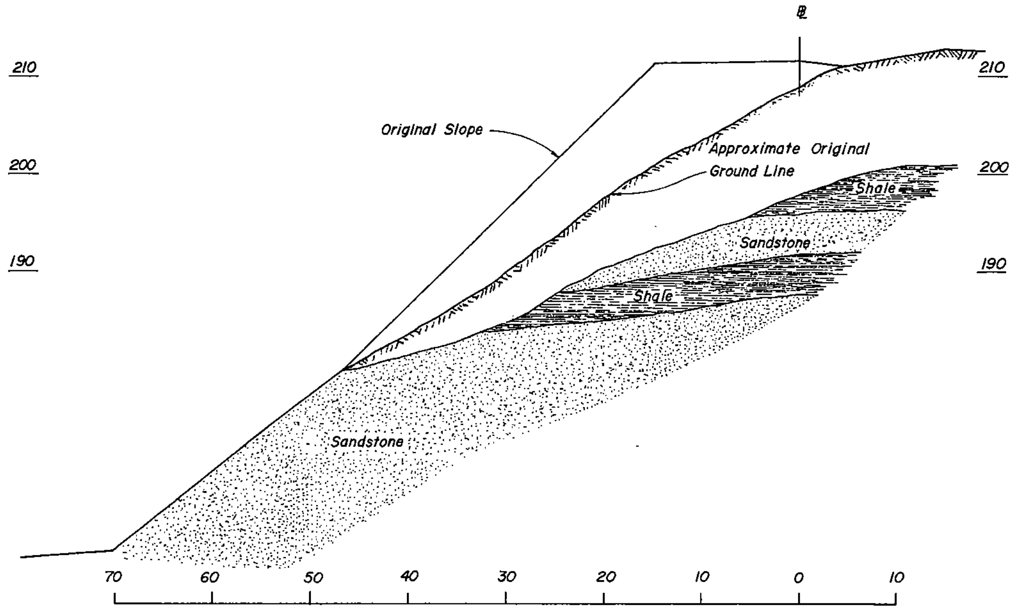
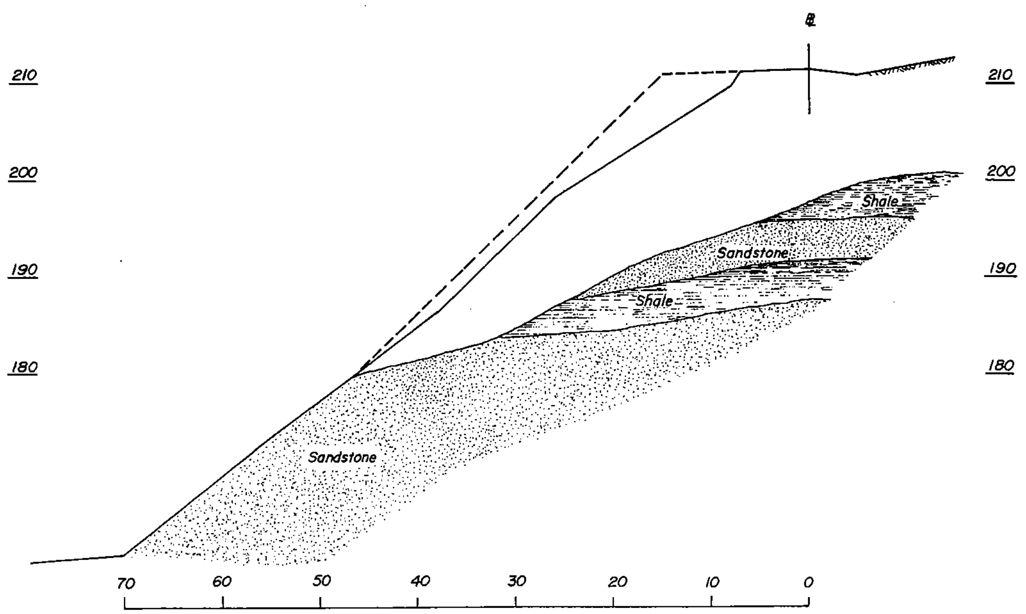
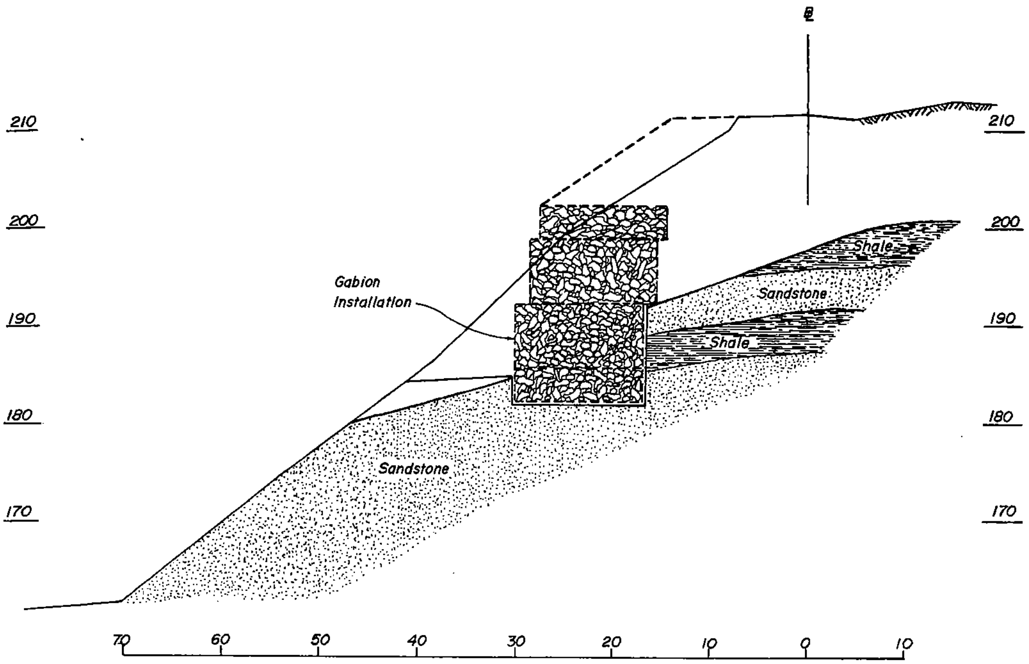


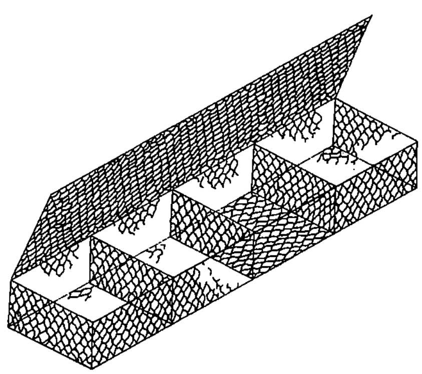
FIG. 7 SLIDE NEAR RUSSET W. VA.
X-SECTION PRIOR TO SLIDE



X-SECTION AFTER SLIDE
FIG. 8



CORRECTION
FIG. 9



TYPICAL GABION BASKET

FIG. 10

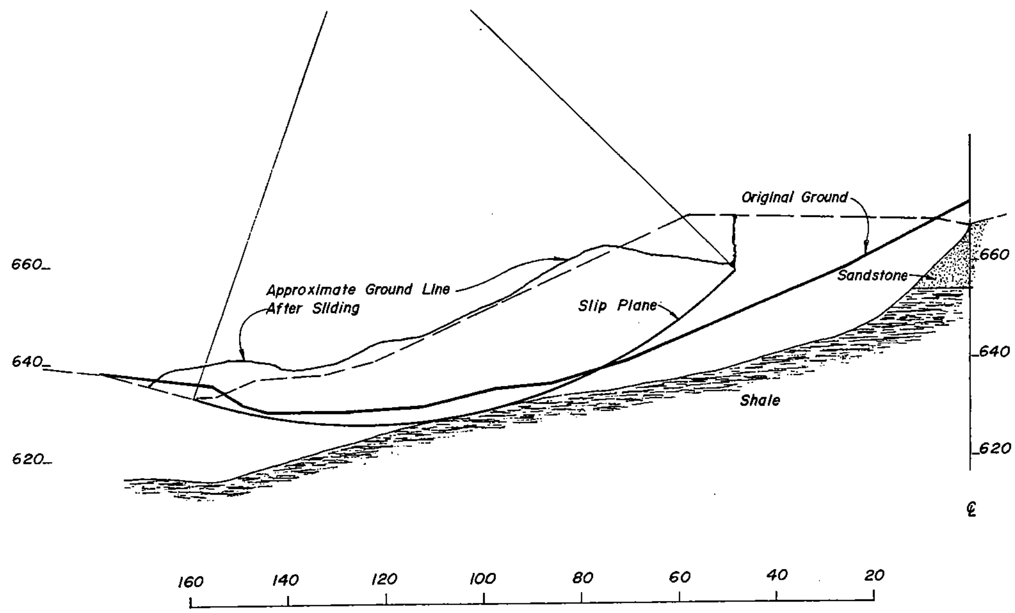
LIDE ON INTERSTATE ROUTE 64 NEAR HUNTINGTON, W. VA.

The embankment was originally constructed in October of 1963, in a ide hill condition as shown in Figure 11. The original fill failed by sliding during January 1963, and was excavated and replaced during May 1963, nder the direction of the project supervisor. Immediately after reconstrucion tension cracks appeared in the top of the embankment.

The pavement was not included in the grading contract, thus a study of he slide area was initiated and if necessary, corrective measures would be ncluded in the paving contract. The condition of the embankment remained unhanged over the winter of 1963-1964. In the spring of 1964 considerable round water was detected in the embankment.

Numerous corrective measures were investigated; however, only three ethods were considered feasible and were compared in respect to cost and ffectiveness. They were (1) excavation into the shale with granular backill; (2) a rock buttress and (3) a horizontal drain installation was considerd to be the least effective method, with results not guaranteed, the cost was egligible in respect to the other two methods. Therefore, it was decided to etermine if the situation could be corrected with horizontal drains. In the pring of 1965, the condition of the embankment was still unchanged. The orizontal drains were installed at that time and the paving contractor was nstructed not to pave in the area of the tension cracks until the effectiveess of the drains could be evaluated.

Upon installation, several of the drains intercepted a large source of ater, which created a near full flow condition, however, the flow tapered ff very quickly to a small trickle. Other drains never functioned or else ceased



X-SECTION AFTER SLIDE
FIG. 11

flow after a short time. When it came time to place the rigid pavement there was still no indication of additional movement, however, water was still observed in the fill. The area within the limits that could possibly be affected by movement was then paved with a minimum thickness of flexible pavement and opened to traffic, since it could not yet be determined if the drains were going to provide satisfactory results.

In January, 1967, a complete failure occurred. The outside pavement in the west bound lane dropped approximately five feet for a length of approximately 400 feet. The condition was then re-evaluated and plans were prepared for correcting the slide using the excavation and granular backfill method.

When the excavation operations uncovered the shale during the construction of the slide correction, the condition of the shale was not as good as anticipated. When exposed the shale weathered extremely fast into a plastic clay because there were fractures throughout the shale which were carrying water and the shale was weathered on each side of the fractures. There were also zones in the shale that were in a condition that when pressure was applied by hand the shale shattered into small aggregate like pieces. There had also been delays in the construction operations which left the excavation backslope for a period of several weeks. This resulted in failures in the backslope in the soil and in some areas failed through the soil and the shale itself.

The excavation had not reached the proposed bottom elevation when this condition was realized and upon investigation, the analysis indicated that it was possible, in time, for a slip plane to develop in the shale below the proposed bottom of the excavation. Therefore, the excavation was terminated at the elevation that existed at that time and granular backfill was placed for drainage. A soil buttress was then incorporated into the design at the toe of the new slope designed to provide stability against the potential deeper movement.

A CONTROLLED FILL OVER SEDIMENTS OF ANCIENT LAKE MONONGAHELA
Near Fairmont, West Virginia

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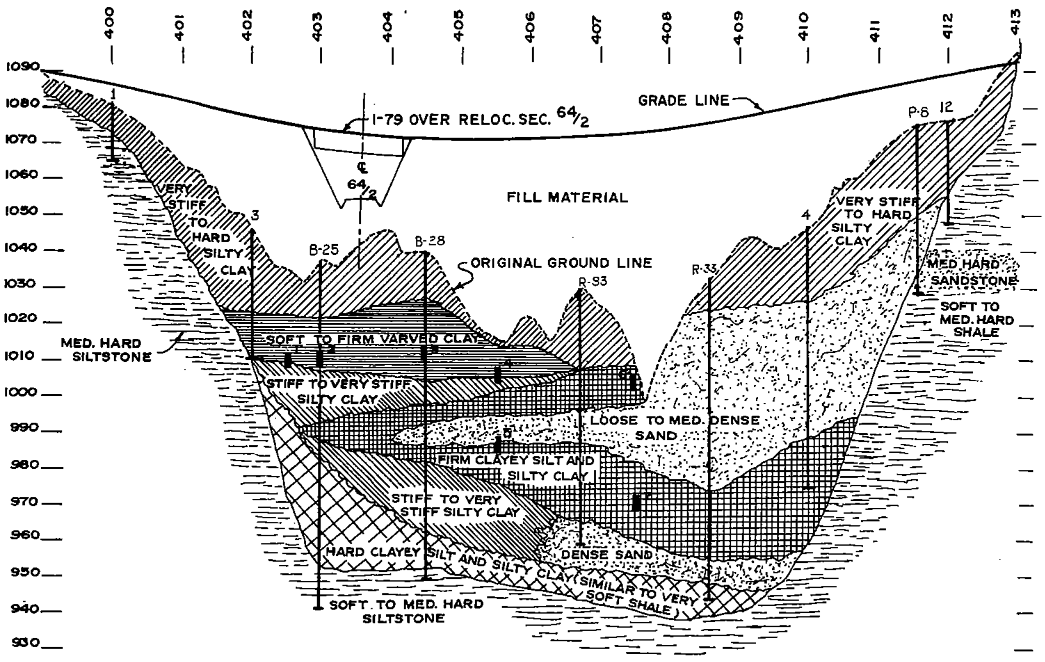
The great Pleistocene ice sheets did not penetrate into West Virginia, but eir influence on the established drainage of that epoch is clearly discernible. ior to the Ice Age, the major drainage in West Virginia was to the North. The vance of the glaciers over Ohio and Pennsylvania intercepted and blocked this ainage. Several lakes were formed which produced extensive deposits in the io, Kanawha (including the abandoned Teays), and Monongahela River Valleys, well as in most of the major and many of the minor tributaries to these reams.

At least five (5) terraces marking various levels of deposition have been entified. Even above the ground water table, these deposits often exhibit isture contents of as much as 40%, and with few exceptions are extremely sceptible to landslides if the natural equilibrium is disturbed. Most of the ilures occur as the classic rotational or slump landslide, and once failure occurred, a slope of five horizontal to one vertical, or gentler, is often essary to regain stability. Most of the deposits are covered by varying unts of residual soils from the adjacent valley walls and thus the true ger is not always apparent unless a comprehensive investigation is conduct- There are, of course, extreme potential dangers involving stability when structing roadways, buildings, or other structures which will be founded on will cut through these terrace deposits.

One such deposit occupied the corridor of Interstate 79 between Bentons Ferry and Fairmont, West Virginia. The routine subsurface investigation for design purposes in the valley known locally as Pleasant Valley, disclosed a vast deposit of ancient Lake Monongahela. It is interesting to note that past records concerning the various terrace deposits in the Fairmont area indicate that the rock floor of the pre-glacial river should be at elevation 972 and the top of the highest terrace should be at elevation 1067. The drilling and geologic investigations conducted to gather data for design, produced evidence that the previously assumed floor of the pre-glacial stream may be in error. The investigation discovered a deep valley filled with varved clays over other deposits of silts, sands, and clays (Figure 1). The floor of the filled valley is at approximate elevation 945, indicating that the floor of the pre-glacial river was some 27 feet lower than previously reported. It had been assumed, prior to this finding, that the present Monongahela River had cut 122 feet from the pre-glacial floor. From the foregoing evidence, however, it appears the post glacial stream has eroded no more than about 95 feet.

It is also believed that a thorough geologic investigation would reveal that Pleasant Valley (location of the filled valley) may actually have been a meander of the pre-glacial river which flowed in a northerly direction (Figure 2). During the interval of impoundment the meander was filled with lake sediments. There is evidence that the lakes were drained in a southwesterly direction and for some reason the meander was not reopened even when the river resumed its original northerly flow. The significance of these findings lies in the fact that other zones of unstable soil may be found in this area at a lower elevation than previously supposed.

The presence of these lacustrine deposits in the Pleasant Valley area resulted in the study and rejection of many alignments within the I-79 corridor.



SOIL PROFILE
STA. 399+00 TO 413+00
I-79 MARION CO.
FIG. 1

It was found to be impossible to completely avoid the deposits, but the line and grade finally selected reduced the problems associated with this material to only one of major significance -- that of constructing a relatively large embankment over thick deposits of compressible material. Alignments for the Interstate and Secondary road were studied on both sides of the valley but the cost of insuring stability of sidehill fills over the lake deposits was prohibitive.

Borings and laboratory tests provided the designer with the information to develop a detailed soil profile for the area (Figure 1). The deposition does not appear to be uniform. Lenses and pockets of sand, clay and silt are interspersed irregularly throughout. Laboratory tests on undisturbed samples indicated a potential for as much as 30 inches of settlement. The lack of continuity of stratification made it difficult to determine the nature and extent of lateral drainage within the deposits and thus the validity of consolidation tests, especially with regard to time, is suspect.

Analysis of available data indicated that placement of the load of the proposed embankment too quickly could result in the build up of critical pore water pressures in the foundation soils which could, in turn, be translated into a major failure down the valley normal to the profile. To guard against this possibility the designer proposed a controlled rate of construction. It was specified that when the construction reached elevation 1040, the rate of fill should be no greater than 12 feet per month. It was further specified that construction should be halted

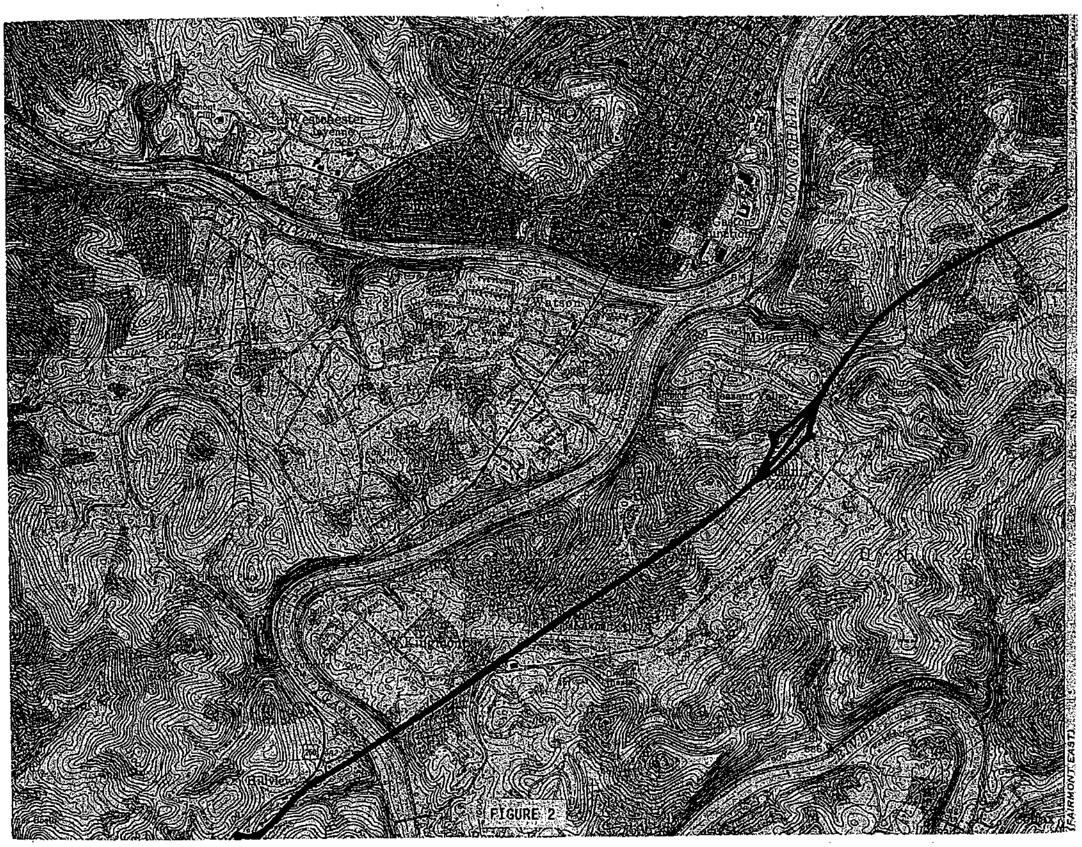
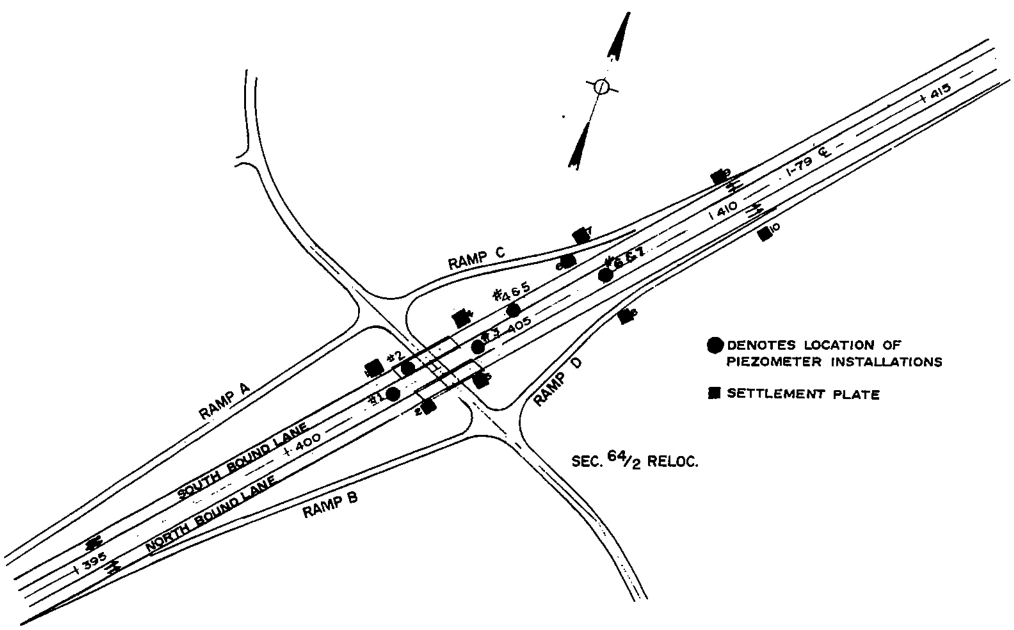


FIGURE 2

at elevation 1055 and a four month waiting period observed to permit the foundation soils to adjust to the new load. At the end of the four month period, construction of the remainder of the embankment was limited to twelve feet per month.

Besides this predetermined construction schedule, a system of seven piezometer installations positioned at various locations (Figure 3) and depths in the foundation soil was specified in order to provide a rapid and constant record of actual pore pressure development. A pore pressure of two tons per square foot was calculated to be the critical stress and all operations were to cease and pressures allowed to dissipate should this value be exceeded. Other devices used in the control of this fill were settlement plates (Figure 3) to determine if anticipated settlement was being realized; and slope stakes along the embankment to provide early warning of any loss of lateral stability.



PLAN
SCALE 1" = 200'
I-79 MARION CO.
FIGURE 3

The main feature of the embankment control was, of course, the piezometer installation. A piezometer, simply stated is a device for the purpose of observing pressures of liquids in compression. There are several types of piezometric observation systems in use, ranging from simple open pipes permitting water level measurements, to intricate systems of tubing and sensitive gages, or even electrical indicating devices. The type used in the control of this fill was a closed hydraulic system consisting of a porous stone tube acting as a sensor which is connected through fluid filled lines to a pressure gauge (Figure 4). As will be seen later, however, our system differed somewhat from the normal installation of this type.

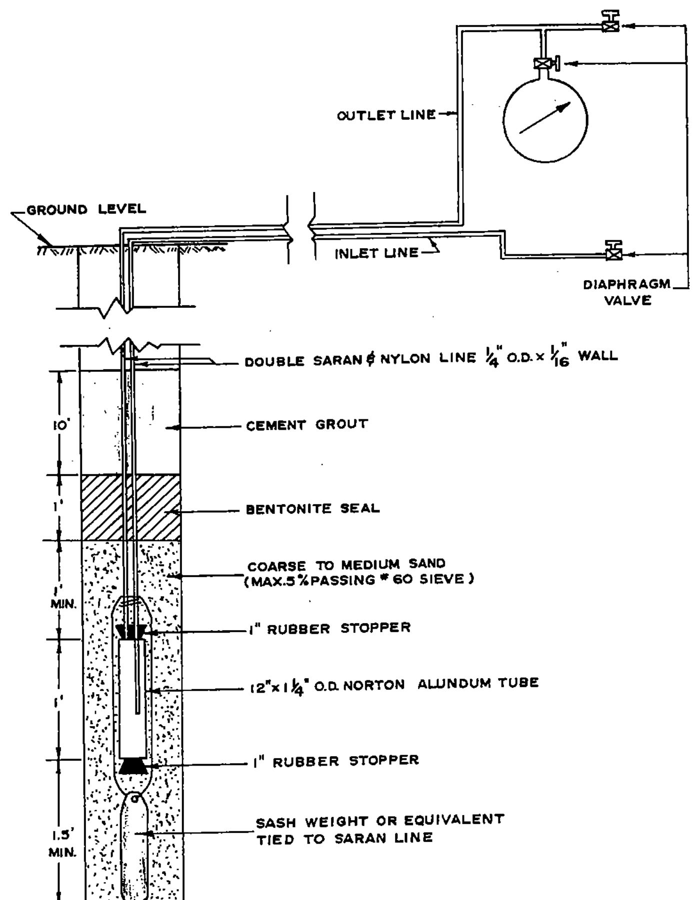


FIG. 4
DETAILS OF
PIEZOMETER SYSTEM

The use of piezometers is not uncommon. Most earth dams incorporate elaborate piezometric monitoring installations. In such cases, they are normally placed at prescribed levels in the dam as construction proceeds. In the subject case, the zone of interest was the inplace soil and thus the installation had to be made through drilled holes. The installation procedure was as follows: A drill rig was moved onto the specified location and a six inch hole was drilled to the desired depth. A dozer mounted core drill was used to negotiate the terrain which was, while more accessible than the average for West Virginia, fairly rough. Casing was used only where necessary to maintain an open hole. Next a piezometer was readied for insertion. The Saran Tubing was placed through the rubber stopper and the stoppers cemented into the porous stone. A sash weight was attached and a framing device was affixed to keep the tube off the wall of the boring. The piezometer was then lowered into the hole to the desired depth and the backfill started. Ottawa sand was emplaced to act as a filter and to insure a porous medium for at least 1.5 feet below and 1 foot above the porous stone tube. A bentonite seal of at least 1 foot was placed on top of the sand. The next ten feet or to the top of the hole, was filled with cement grout.

Trenches were excavated to conduct the saran tubing to the gage sites outside the fill. The purpose of the trench was to protect the tubing during the early stages of construction. A common trench was used to contain the tubing from more than one installation when possible. The lines were placed loosely in the trench and covered with a layer of sand to allow for a certain amount of movement as the fill settles. A total of approximately 5900 feet of saran and nylon tubing was used to connect the seven sites with their individual gages. When splicing was required extreme care was taken to insure an air tight connection.

Because of the depths of some of the installations, it was not possible to establish the gage sites at a lower elevation than the piezometric level as is required to maintain positive pressures. This would have required placing the gages far down the valley away from the interchange and relocated secondary road. The procedure was to connect the gage at the hole site, fill the system with fluid, close the valves sealing the system, and move the gages to their permanent observation sites. When the gages were moved to their permanent sites, vacuum was developed relative to the height above the piezometric level. Changes in pore water pressure in the foundation were reflected in changes in vacuum at the gage.

Most of the installations went reasonably smoothly, but there were some problems. For one thing, the weather and terrain combined to give trouble at times. The system was emplaced in the early Spring and freezing and thawing, snow and spring rains caused equipment to mire up. The cold weather also slowed operations by making the saran tubing brittle and very difficult to manipulate. Nylon tubing, purchased when we ran short of saran, was much easier to handle. Some of the holes were plagued by sand running in. It is difficult to use casing to seal off such a condition. When the piezometer is in place, the lines running out the top of the hole make standard casing removal difficult. It is necessary to either cut and splice the tubing (and it is desirable to keep splices and connections to a minimum), or the entire length of tubing must be "threaded" through each piece of casing.

The first valves purchased had brass seats and were unable to contain the vacuum and it was necessary to replace them. The replacements were Midget Diaphragm Valves and when the systems were reconnected, the seal was improved at all connections by wrapping the threads with teflon tape. The gages used were of the liquid filled diaphragm type with a range from 30 inches vacuum to 60 psi pressure. They were factory sealed and calibrated.

Construction of the embankment and bridge is now complete, and from the records and the performance of the embankment it appears that the controlled construction was successful. Except for minor discrepancies the piezometers functioned as expected.

SITES 1 and 4

Note (Figure 1) that Site 1 and Site 4 were installed at the top of a stiff to very stiff silty clay. This material would be very susceptible to pore water pressure build up under loading due to its low permeability. [mmediately above the layer is a bed of soft to firm varved clay. The piezoneter graphs for Sites 1 and 4 (Figures 5 and 6) show overall similarity except that Site 1 readings show many more short term fluctuations. Site 1 :eached slightly higher values initially due to greater construction activity .n that area. The differences in the actual readings are due to elevation ifferences between gages and installations for the different sites. As the mbankment height increased the piezometer readings increased indicating a buildp of pore water pressure. After construction ceased, the pressure dropped nly to rise again when construction resumed. At 500 days (approximately months after construction was completed), the pore pressure was steadily ecreasing.

ITE 2, located in a varved clay was experiencing a similar rise and fall orresponding to embankment construction until destroyed by a pile driving peration (Figure 7).

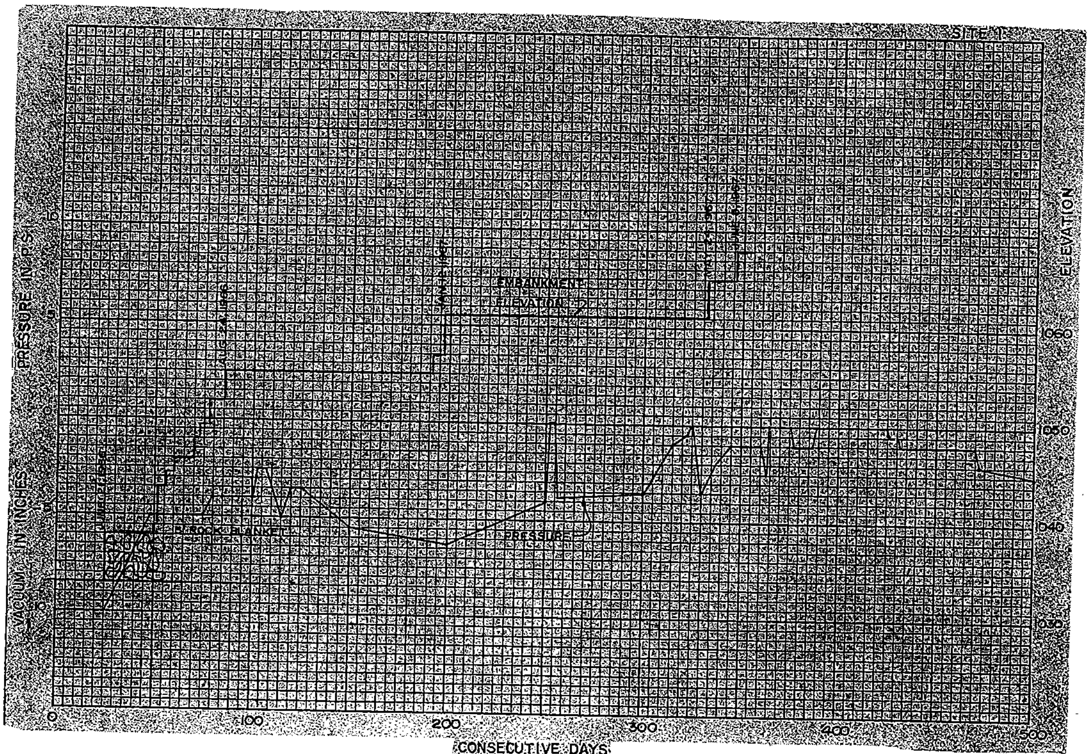
[TE 3, is also located in a varved clay. This site appeared to be inactive itil approximately 350 days when it showed a sudden increase in pressure igure 8). Site 3 is located at the north end of the bridge over the local road. In order to go ahead with the bridge construction, the contractor was permitted to build the fill to final grade in a small area over Site 3. Approximately 20 feet of fill was placed very quickly and probably accounts for the sudden increase. The water being squeezed quickly found an escape and the pressure returned to normal. A pile driving operation near the site could also have effected the readings.

SITE 5, located in a loose to medium dense sand, showed little effect due to pore water pressure. This was expected due to the high permeability of the sand as long as the bed had drainage in some direction for dissipating the water as consolidation occurred (Figure 9).

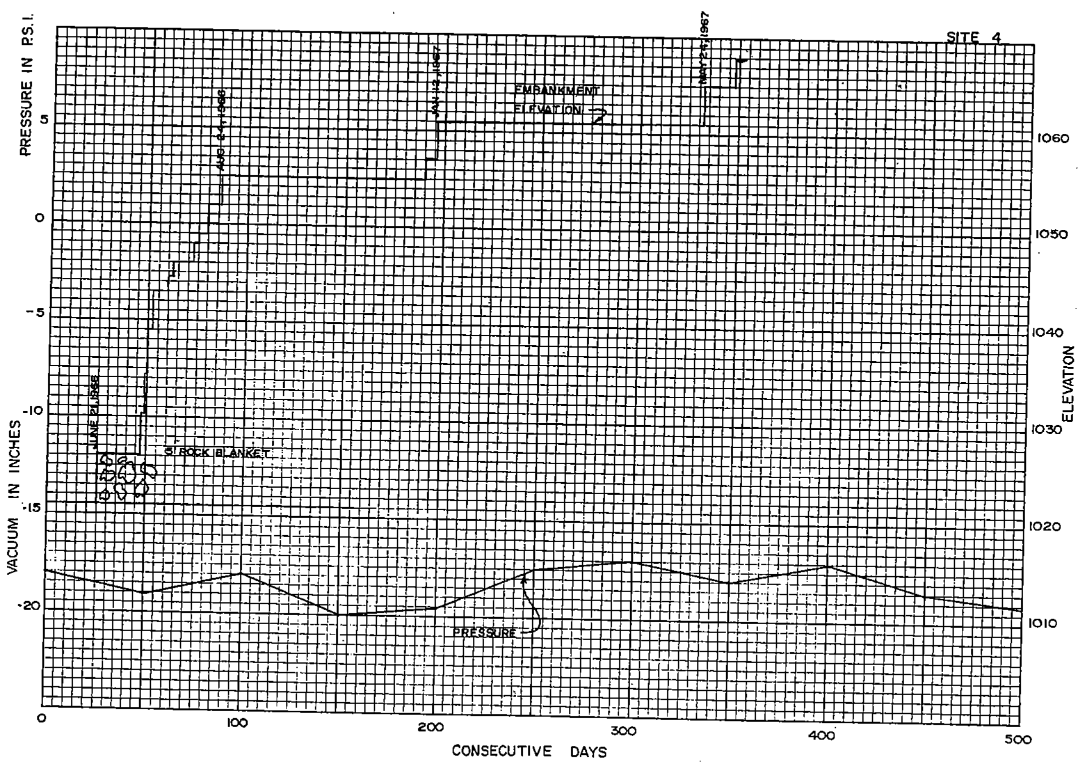
SITE 6, placed in a clayey silt and silty clay bed, seemed to experience difficulties similar to Site 3 (Figure 10). The Designer's Soil Report states that the top of the inplace soil often exhibits vertical and horizontal desiccation cracks. This site could have been near enough to the surface for relief to have been provided through these cracks until the fill consolidated the material enough to block the drainage and cause the rapid build-up.

SITE 7, located deep in a bed of clayey silt and silty clay, registered a steady rise until construction was completed. This would be expected due to the low permeability of this material, the thickness of the bed and the height of fill (Figure 11).

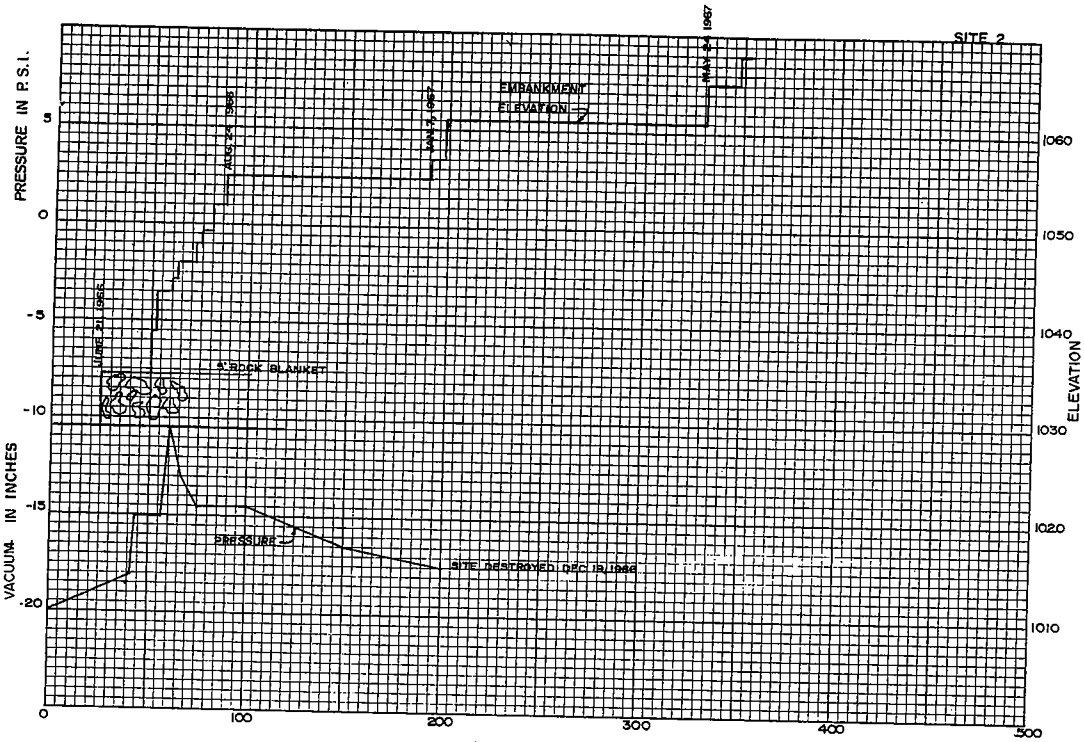
As stated previously, settlement is difficult to predict in lensed deposits such as these. The settlement registered by the settlement plates was much less than that estimated by laboratory tests (Figure 12). The largest settlement occurred at 405 + 50 on centerline. Thirty inches were predicted, but only 9.6



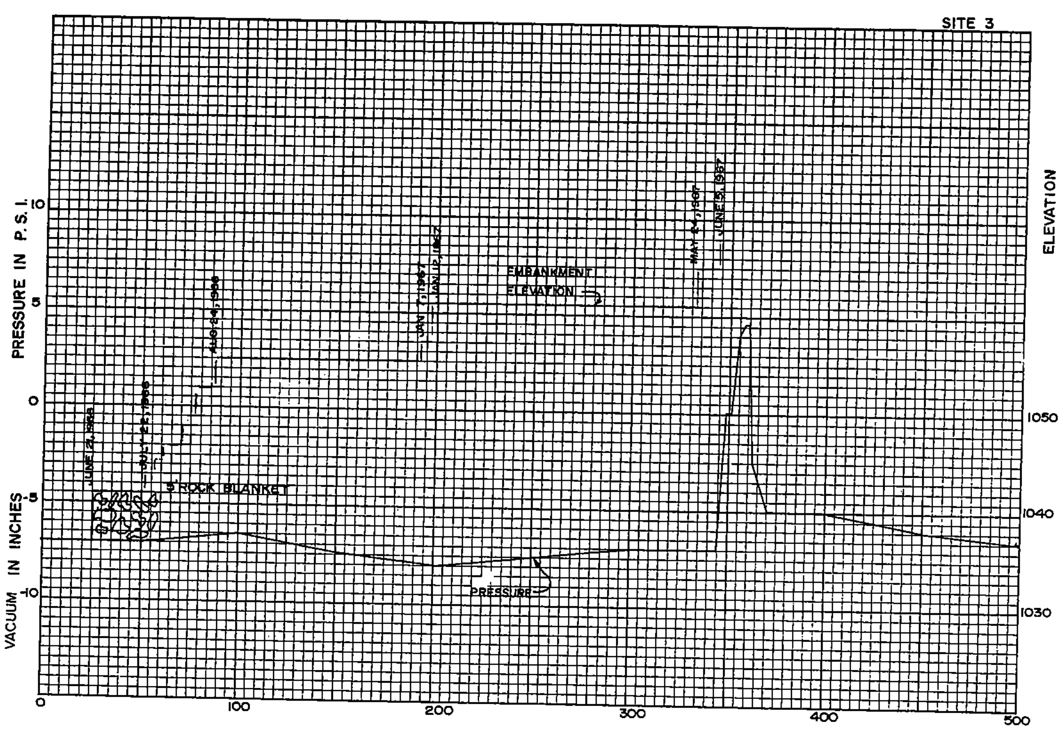
CONSECUTIVE DAYS
FIGURE 5



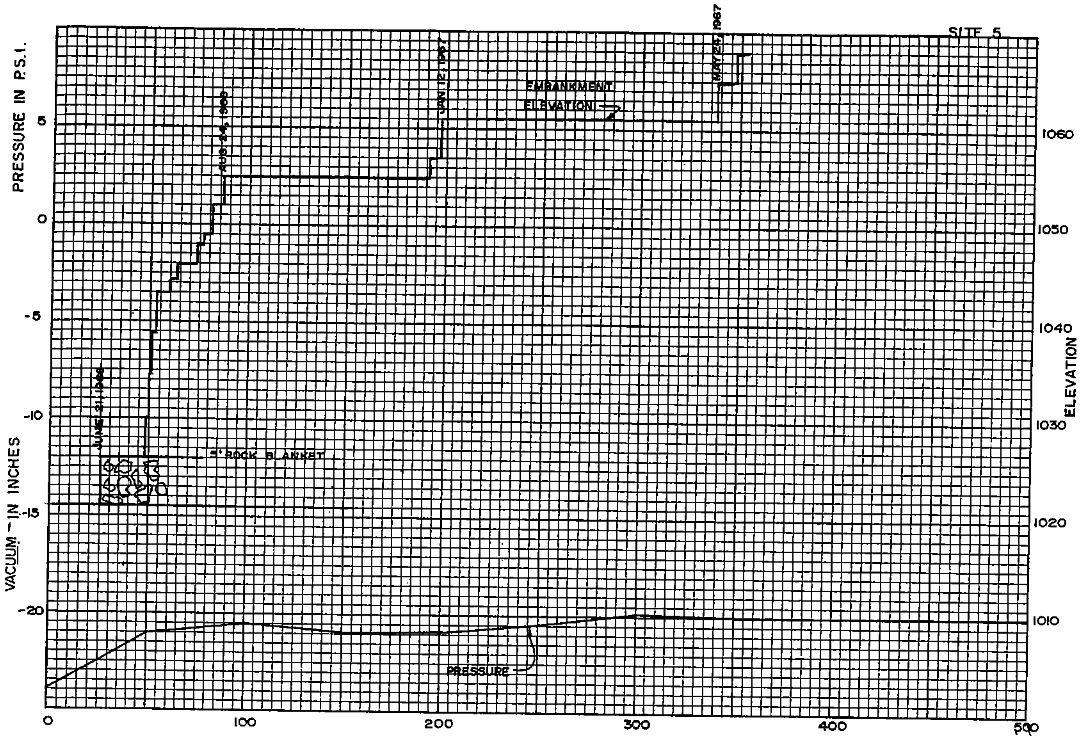
CONSECUTIVE DAYS
FIGURE 6



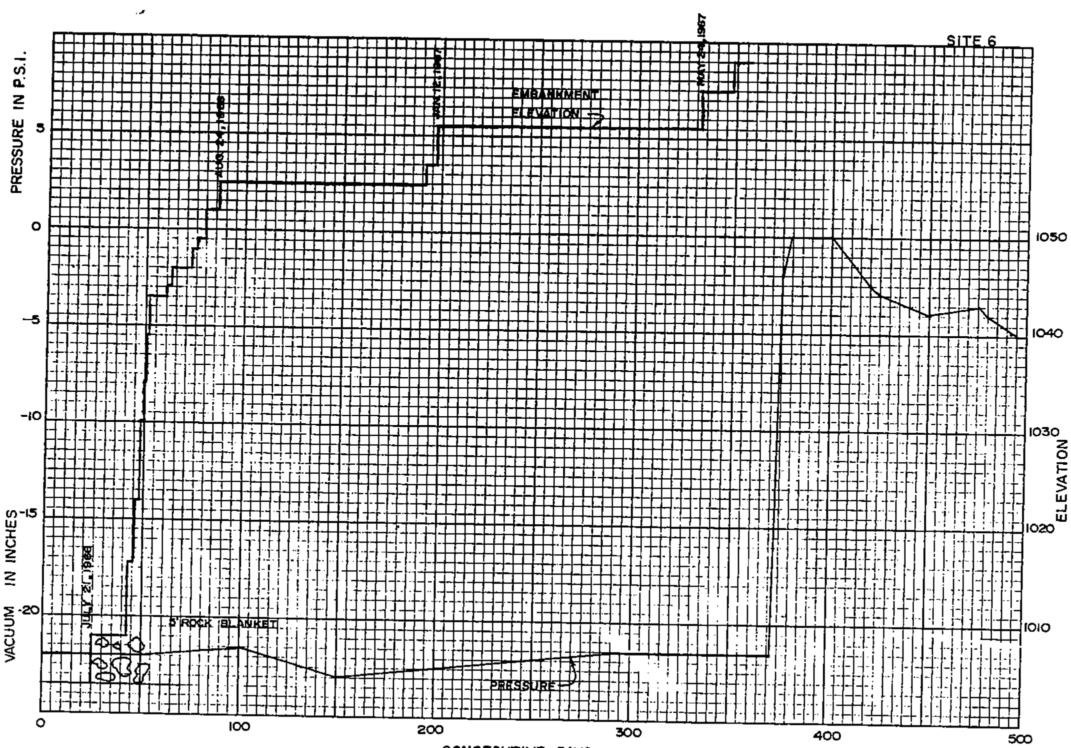
CONSECUTIVE DAYS
FIGURE 7



CONSECUTIVE DAYS
FIGURE 8



CONSECUTIVE DAYS
FIGURE 9



CONSECUTIVE DAYS
FIGURE 10

inches of the anticipated settlement was recorded.

It is evident then that neither the settlement nor the attendent)uilt-up of pore water pressures reached the predicted degree of severity. :hus, either the deposits were not as susceptible to consolidation as the .aboratory tests indicated; or the time for consolidation is much greater :nd the critical period has not been reached. Actually the settlement curves Figures 13 and 14 are typical) developed from the settlement plate readings, evel reassuringly after loading ceased indicating that primary consolidation as likely been achieved and that the most critical period has passed.

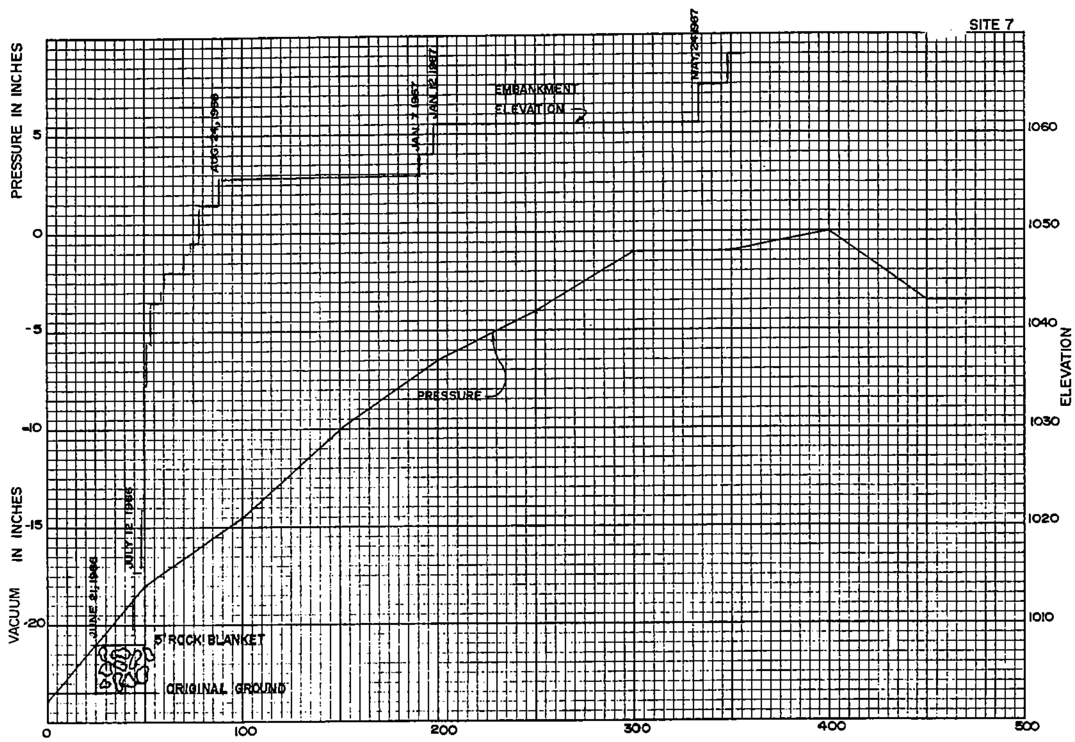
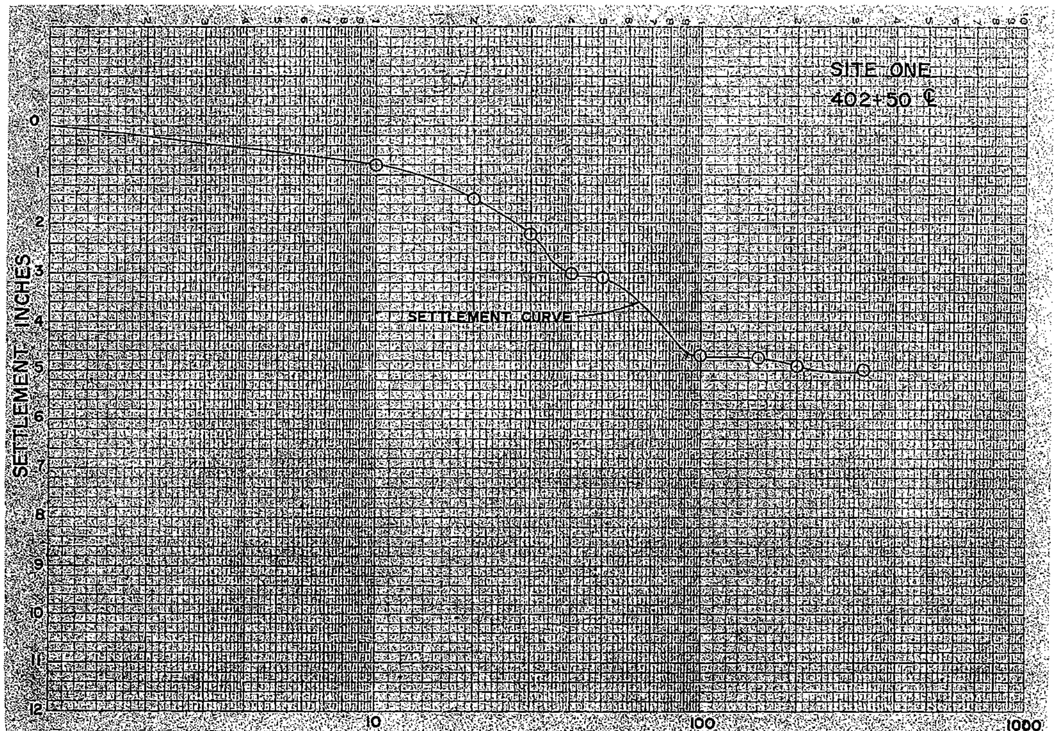


FIGURE 11

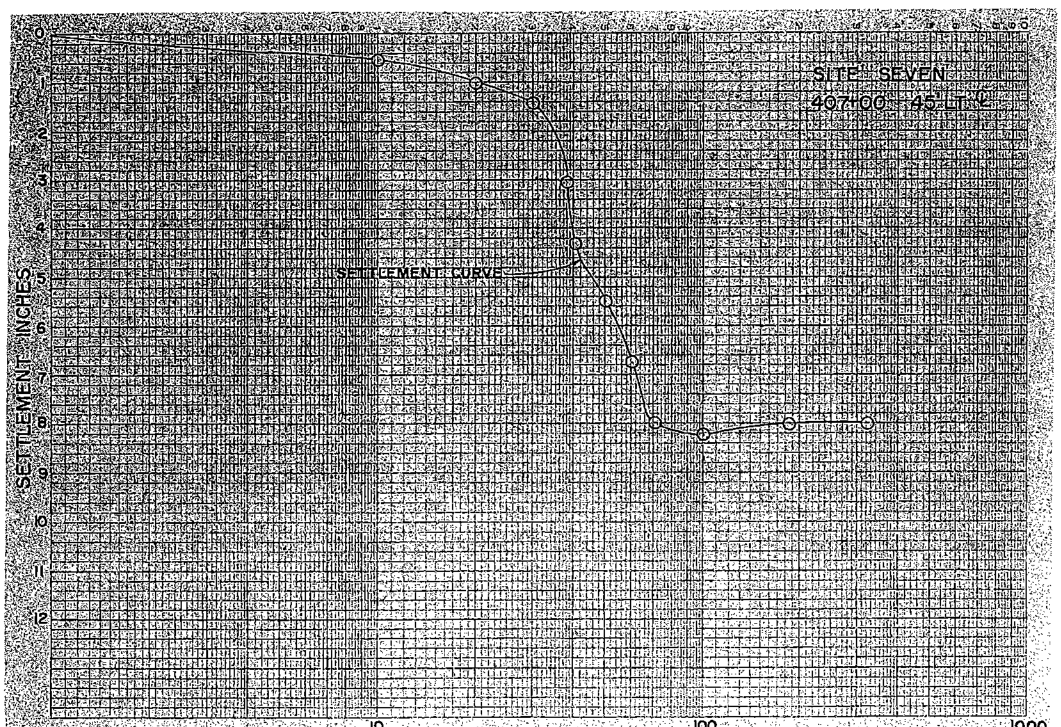
SETTLEMENT PLATES

SITE	STATION	OFFSET	FEET OF FILL OVER PLATE	ESTIMATED SETTLEMENT	ACTUAL SETTLEMENT	PERCENT OF ESTIMATED SETTLEMENT
1	402+ 50	65' LT.	40	15"	5.3"	35 %
2	403 + 00	65' RT.	22	—	2.9"	—
3	404+ 25	65' RT.	26	20"	2.4"	12 %
4	404 + 50	65' LT.	24	20"	4.6"	23%
5	405+ 50	℄	52	30"	9.6"	32 %
6	407+ 00	65' LT.	73	25"	7.1"	28%
7	407+ 50	92' LT.	62	24"	7.3"	30 %
8	407+ 50	90' RT.	35	—	5.2"	—
9	410 + 50	65' LT.	27	10"	2.2"	22 %
10	410 + 75	82' RT.	27	10"	0.0"	—

FIGURE 12



TIME DAYS FIGURE 13



TIME DAYS FIGURE 14

GEOLOGICAL INVESTIGATION FOR A TRANS-ANDEAN HIGHWAY

Ralph W. Seeger
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The Republic of Columbia is situated in the northwest part of the South American continent. It is the only South American country bordering on both the Atlantic and Pacific Oceans. The Andes Mountains dominate the western sector of the country and split into three ranges fanning out northward from the Ecuadorian border. East of the Andes is a vast area known as Los Llanos. The north and west part of this region is grassy plains while the rest is moist jungle.

The Eastern Llanos (or plains area) of Columbia have become an important factor in providing produce and livestock to the rapidly growing capital city of Bogota. The Llanos region comprises 25% of the total area of Columbia, yet presently is inhabited by only 1.4% of the nation's population. In addition to sharing in the support of Bogota, Los Llanos has great potential as a future developing area within itself. Therefore, an efficient method of land transportation connecting Villavicencio, the gateway to Los Llanos, with Bogota is vital.

The Government of Columbia desired to know the present status of the existing Bogota-Villavicencio Highway and the engineering requirements, costs and economic impact of providing a new facility or improving the existing highway to a higher level of performance. This study was conducted to achieve this purpose.

The present highway is subject to slides, washouts and rock falls. The highway is narrow; in some places being restricted to one-way travel for short stretches. Grades are steep, being as high as 12%. Several sharp hairpin turns and limited sight distances make it desirable to modernize the route, whether by relocation or upgrading the existing highway. Drainage on the existing road is veritably non-existent and is probably responsible for many of the failures of the slopes and road surfacing.

The geological studies for the proposed route cover an area approximately 750 square kilometers confined to a strip averaging 7 kilometers in width extending between Bogota and Villavicencio. The proposed route begins at the southern edge of the city of Bogota, elevation 2,570 meters, and ascends the Eastern Cordillera, crossing the divide at elevation 3,170 meters. From this point of maximum elevation, the route descends to terminate at Villavicencio, elevation 625 meters. This city is located at the western edge of the vast Llanos area. The area studied is part of the Rio Orinoco drainage basin.

The detailed photogeologic study was made utilizing aerial photography on a 1:25,000 and 1:60,000 scale. This is the first time that this area has been mapped both topographically and geologically, using air photos. Geologic formations and soils were identified and located with photogeologic interpretation and field control. A topographic map was made from air photos on a scale of 1:10,000 using a contour interval of 10 meters. The major geologic and soils features were plotted on this map, thus providing a base map for all alignment studies. The geological aspects were considered in the engineering study so that route evaluation was made from a geologic as well as an engineering standpoint.

The topography of the area varies from flat terrain both at Bogota and Villavicencio to undulating and rugged mountains in the rest of the area. Severe tectonism and deep erosion are responsible for the topographic expression as well as for courses of the rivers. Terraces, alluvial fans and talus slopes are located adjacent to the mountainous area.

Rocks of sedimentary origin representing all geologic periods between the Cambrian through the Quaternary, except Permian and Silurian, outcrop within the studied area. No igneous outcrops were observed within the area of this study. Metamorphism has altered Cambrian, Ordovician and Devonian sediments to varying degrees.

The Cambro-Ordovician time is represented in the area by phyllite with some quartzite and schist present. The Devonian System contains black fissile shales, slates, and quartzites. The Carboniferous System principally contains red shales with some sandstone interbeds, but no coal seams were noted. The Jura-Triassic contains shales capped by conglomerates. These conglomerates are composed of fragments of phyllite, quartzite and shales from the older formations. The Cretaceous System is represented by a marine deposit containing yellow sandstone and maroon and black shales. The Tertiary continental sediments were deposited after the Cretaceous marine sedimentation. They are to be found at both ends of the study route; and are primarily clays or clay shales with interbedded sandstones. The Quaternary is represented by unconsolidated deposits of alluvial and fluvio-glacial origin. Also included in this group are recent talus, residual soils, and heterogeneous materials from the slopes.

Soil deposits of diverse origin are irregular in shape and generally average 2 meters in thickness in the area. Origin of the soil can be traced to: fluvio-glacial, alluvial, fluvial, slides, talus, weathering of parent rock, etc. Rock blocks, gravel, sand, silt, clay and loose materials are all irregularly mixed according to location of parent material.

Unconsolidated deposits include terraces, straths, alluvial fans, fluvio-glacial cones, and mudflows. Terraces are composed of subrounded to subangular rocks ranging in size from pebbles to boulders. Terraces at the Bogota end of the route consist mainly of sandstone fragments while at the lower end of the route the terraces contain phyllite, quartzite and sericite schist in clay matrix. The alluvial fans contain subangular, poorly graded particles. A few fluvio-glacial cones are present and are generally well-drained with flat slopes. Mud flows are semi-consolidated and may or may not contain large rock particles. They are subject to saturation and consequent viscosity and are to be avoided whenever possible.

The rocks in the study area are severely folded and faulted. The entire structure is further complicated with different periods of folding. The number of parallel north-northeast trending thrust faults are abundant and appear to follow a regional pattern. Normal faults also occur at numerous places. In general, tectonism increases in intensity in the direction of the Llanos. In some places folding and faulting were so intense that geologic structures could not be identified. Faults producing intense fractures affect the stability of the rocks. Close faulting in finely stratified rocks produces brecciated material subject to easy erosion, slumping and slides, as evidenced on the existing roadway.

Studies resulted in the conclusion that isolated faults or groups of sufficiently separated faults are not critical in highway construction. Since local faults in massive rock produce large blocks, they are not considered a menace to construction. However, finely stratified rock that has been subject to close faulting produces a highly disintegrated material that is susceptible to erosion agents. During the construction stage, the excavation of brecciated material in a fault zone may cause some sliding, but proper design and construction should prevent continuation. In design, care will be taken not to concentrate drainage in these areas which could create erosion problems.

Three types of soil failures are prevalent in the study area and may be classified as slab slides, creeping soils and mud flows. Slab slides occur in residual soil deposits developing over steeply dipping strata. Infiltrating water lubricants the contact zone and failure results. These slides occur throughout the study area but are predominant in the Cretaceous shales. Road cuts in this material upset the delicate balance of stability and accelerate movement. Creeping soils are caused by a strength reduction in the soil. This type is most prevalent on the existing road in the vicinity of alluvial fans. Highly saturated soil deposits have created mud flow conditions also in the Cretaceous shales.

Alluvial fans and terraces composed of material primarily derived from hard rock normally have good internal drainage. These fans and terraces may be subject to erosion on unprotected slopes, heavy infiltration of water to impervious bedrock or undermining by adjacent streams. Any one of these conditions will create slope failure.

Based on the kind of materials encountered and field observations of existing slopes, design criteria for cut and fill slopes for the proposed highway were established. Depending upon rock type the cut slopes varied from $\frac{1}{4}$:1 to as much as $1\frac{1}{2}$:1 for clay shale and loose and weathered rock. Slopes in soils were held at 2:1. Twenty-foot wide benches were recommended at an average vertical spacing of 30 feet. Embankment slopes varied from $2\frac{1}{2}$:1 to $1\frac{1}{2}$:1 for heights over 10 feet. Slopes of 4:1 were recommended for embankments less than 10 feet high. Flat bottom ditches were recommended wherever economically feasible.

Unstable cut slopes and embankments, erosion, mud flows and natural slides were considered in the stability studies. Correction of unstable cut slopes involved the removal of excess material down to a stable slope. For embankment failures, both retaining walls and toe berms were considered. Erosion protection included interceptor ditches, flumes, paved ditches and stream diversion. Benching of cut slopes, sand drains and retaining walls were considered in mud flow areas. In natural critical slide areas, removal of excess materials and by-passes were utilized. Any cutting and filling in unstable areas where unavoidable, will require consideration of drainage for inplace materials. Undercutting and backfilling with granular material may be required in areas of weak soils. Use of sand drains and other methods to control known soil slides is anticipated.

Benching, where required for side hill cuts, will generally be made with dozers. Most shales, some phyllites, and minor soft sandstones should be rippable. Blasting will be required for rock excavation but will not be allowed in terrace deposits. Only the absolute minimum required blasting should be allowed in phyllites and thin bedded shales and then only under rigidly controlled conditions to prevent shattering and subsequent slope failures. Controlled disposal of surplus material is required. Dumping over the edge of the roadway section can overload the slope or expose parental rock to eroding agents.

The Sabana de Bogota is a plain of the Eastern Cordillera formed by filling of an ancient lake. The City of Bogota is built on the southeastern edge of this plain. The recommended route leaves the city to the southeast through the Rio Tunjuelo Valley, an enlargement of the ancient lake. The soils here are in general thick and composed of clays and clayey silts. The bedrock is shale with some sandstone and conglomerate belonging to the Tertiary Jsme Formation.

Continuing up the valley, the topography becomes more rolling, indicative of shale bedrock. The soils become more silty. Sandstone boulders, remnants of glaciation, are scattered over the surface. Near the high point of the road the soils are thin and become sandy clays to clays. The bedrock consists of the varicolored shales interbedded with thin layers of sandstone of the Tertiary Bogota Formation.

After crossing the divide, basal sandstones of the Bogota Formation nd shales with thin sandstone beds of the Guaduas Formation of basal Eocene r Upper Cretaceous age are encountered. Next the massive sandstone of the pper Cretaceous Guadalupe Formation is crossed.

Continuing on toward the Town of Chipaque, the route will traverse hales with some sandstone and thin limestone layers of the lower Guadaloupe nd upper Villeta Formations of Cretaceous age. Soils here are sandy clays nd clays. Selection of the less steep areas for these crossings was made o minimize stability problems.

From Chipaque to Caqueza the topography is rolling and becoming more teep. The bedrock here is sandstone, shale, siltstone and limestone interbeds of Cretaceous age.

The town of Caqueza is built on an alluvial fan of clay soils overing black fissile shales. The area, including the town, shows evidence f creep. Because of slumping during deposition and subsequent folding and ulting, the bedding planes are at all angles. These shales weather rapidly and when sufficient soil has accumulated and becomes saturated, slab slides occur.

At the junction of the Rio Caqueza and Rio Negro the valley is choked with sandstone boulders and gravel. The Rio Negro valley is a fault zone and strata on opposide sides of the river are not always the same. The route follows the river and the shales of the basal Cretaceous Caqueza Formation are exposed in the valley.

Continuing onward we find the hard sandstones of the Devonian Floresta Formation locally becoming quartzitic and with a few shale interbeds. After crossing several faults the bedrock changes to phyllite representing the Cambro-Ordovician Quetame Formation. Here the canyons are deep and the phyllite exhibits schistosity. In this area care will have to be exercised so that blasting will not unduly fracture the rock and cause later failures.

The route passes through several alluvial fans and terraces. Then crossing Carboniferous fractured red shales of the Gachula Formation the route passes an area where the attitude of the strata is unfavorable and a deep soil overburden exists which will require special consideration in design and construction.

From here the route will rise slightly to cross the black shales capped by the Jura-Triassic Giron Formation of brecciated conglomerate containing phyllite and shale fragments. The route then descends into the massive Tertiary sandstones and finally terminates on an alluvial fan where the town of Villavicencio is located.

A careful study of geometric and geologic design was made seeking favorable geological conditions to coincide with acceptable geometric criteria established. Variations and alternative solutions were studied from a geological standpoint considering stability, drainage conditions, ease of construction, etc. Evaluation as proceeded considering susceptibility to slides in soils and rocks, specially in interbedded sedimentary rocks. Studies indicate the im- ossibility of avoiding all geologically adverse areas. The geological tudy has recommended relocation of the Bogota-Villavicencio highway. Length f the selected route occurring in unstable areas was minimized and the ad- erse effects will be diminished with appropriate design. The new alignment as reduced the highway length over predictable unstable areas from 37% to 14% r from 41.2 kilometers to 15.5 kilometers. Once again the value of a geo- ogical study for a highway program has been proven.

GEOLOGY, ITS RELATION TO THE DESIGN OF THE EAST RIVER MOUNTAIN TUNNELS

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The largest construction contract of its type in the history of the West irginia State Road Commission will begin in the near future on the outskirts f Bluefield, West Virginia. I'm referring of course to the twin-tunnels on the est Virginia - Virginia border.

The East River Mountain Tunnels, Interstate 77, will penetrate East River ountain with an approximate north-south alignment. The North Portal will be ituated several hundred feet south of Cumberland Road (West Virginia-Route 29), pproximately four and one-half miles northeast of Bluefield, West Virginia. The outh Portal will be several hundred feet north of U. S. Route 21 and 52, approxi-ately two miles north of Rocky Gap, Virginia (Figure 1).

The underground excavations (Twin Tunnels) will each be horseshoe-shaped Figure 2) approximately 32'-3" high, 36'-4" wide and 5,037' in length (this ength does not include 300'+ of soft-ground tunneling at the South Portal).

The feasibility studies for the tunnels and roadway approaches were made n 1962 by Meissner Engineers of Chicago for the Commonwealth of Virginia and he J. E. Griener Co. of Baltimore for the State of West Virginia.

In October 1964, a contract agreement between the State Road Commission f West Virginia and Michael Baker, Jr., Inc., was signed for the design of the ast River Mountain Tunnels.

In April 1965, the preliminary core-boring work began and by November 1966 the operational phase of the final sub-surface investigation and boring program was complete.

The preliminary borings consisted of one angle and 14 vertical holes, with the boring depths varying from 120 to 800', NX size core was retrieved for all holes with soil samples in overburden or unconsolidated strata.

The final boring program consisted of one angle and 8 vertical core borings with a depth range of from 110 to 451 feet. Four of the borings in the South Portal area were 6" diameter and the remainder were NX.

Two additional 6" borings were completed in the South Portal area prior to the final borings in order to determine tunnel alignment at the South Portal.

As most of you know, we can never take as many borings as we would like, mainly because it becomes too time consuming and costly, so we must make full use of that which is available.

This boring program was designed to provide sufficient overlap from hole to hole, taking into account the dip and strike of the formations to be penetrated, to obtain an unbroken sequence from the North to the South Portal.

Correlations between bore holes were made by field surveys and core examinations using fossil, mineral and petrographic horizons, such as the milky quartz pebble layer at the base of the Tuscarora near the Juniata contact.

It is not until a geologist moves into a region where he has not worked before, that he fully appreciates the fine work that has been done by others. In my case, I am very grateful to Dr. Paul H. Price and his co-workers whose work through the years was of considerable help to me on this project, to Dr. Cooper whose work is often cited in Woodward's reports and I must also give credit to M. R. Campbell for his Pocahontas Folio 1896.

EAST RIVER MOUNTAIN TUNNEL

LOCATION MAP

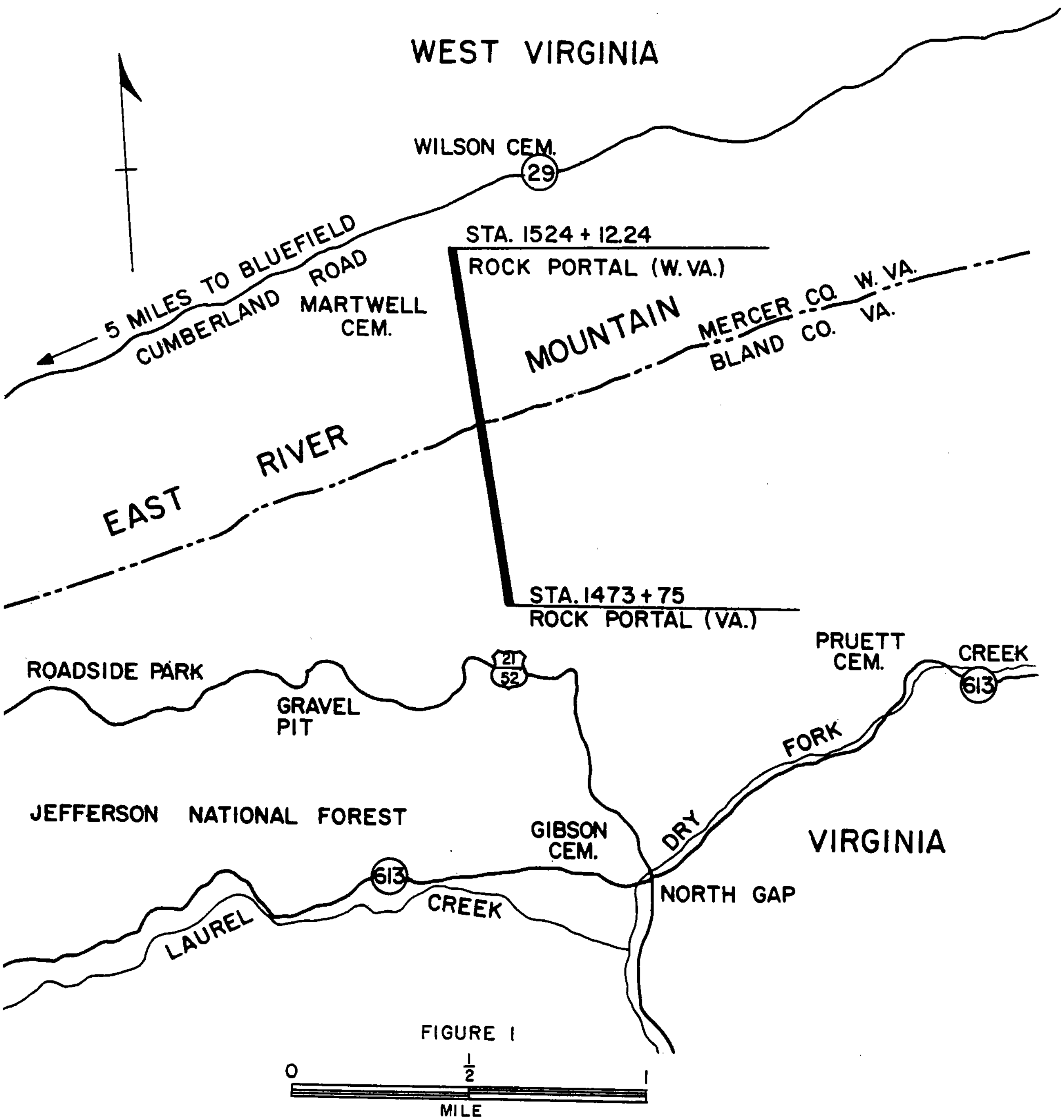


FIGURE I

Because this project involves two states, Virginia and West Virginia, the formational nomenclature became a bit touchy, my search of the literature on this area turned up, in a few cases, several names for the same formation. I attempted to resolve this by using the preferred and most recent names being used by either the Virginia or West Virginia Geological Survey.

The "back-bone" of East River Mountain is the Tuscarora formation which outcrops at the mountain crest. It withstands weathering better than any formation in this locale and is the predominant factor in preserving the ridge which forms the boundary between Virginia and West Virginia in this area.

East River Mountain is structurally the north limb of a syncline (Figure 3), with the St. Clair overthrust fault lying north of West Virginia Route 29 (Cumberland Road) and running parallel to the mountain. The tunnel site and much of the surrounding area is within the overthrust block of the St. Clair fault. The synclinal structure creates a steep northern slope and a gentle

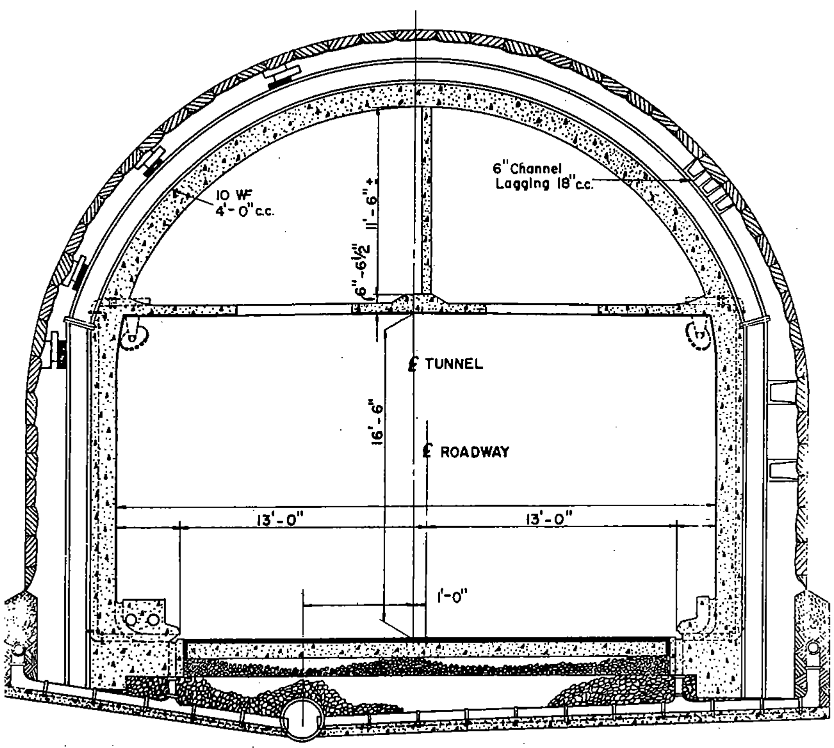


FIGURE 2
ROCK TUNNEL- NOMINAL SUPPORT
TYPICAL TUNNEL CROSS SECTION
Not To Scale

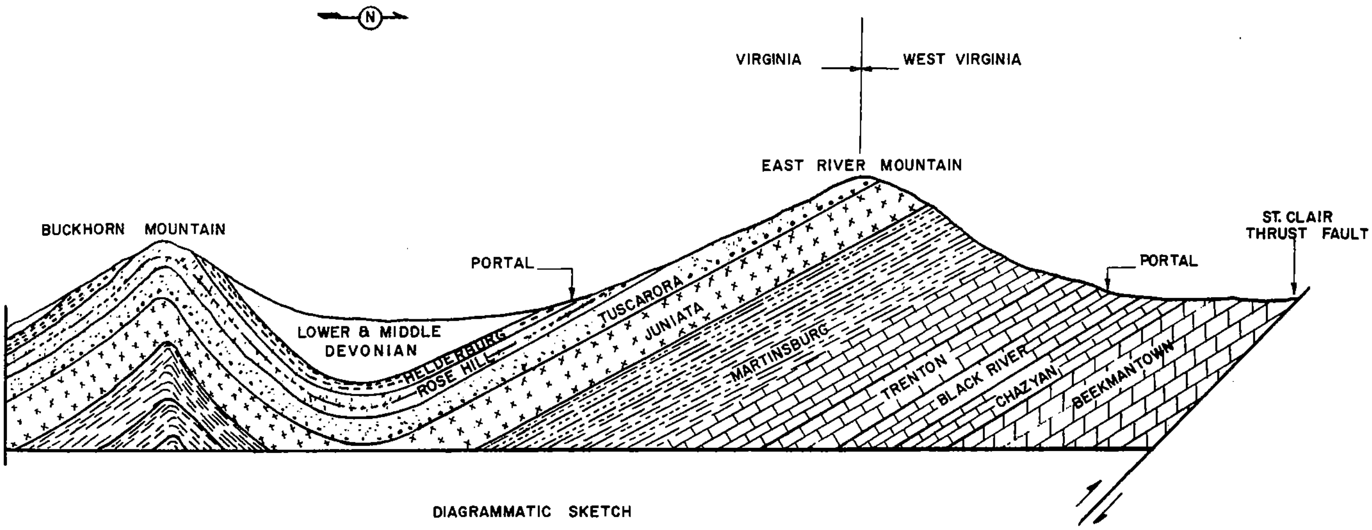


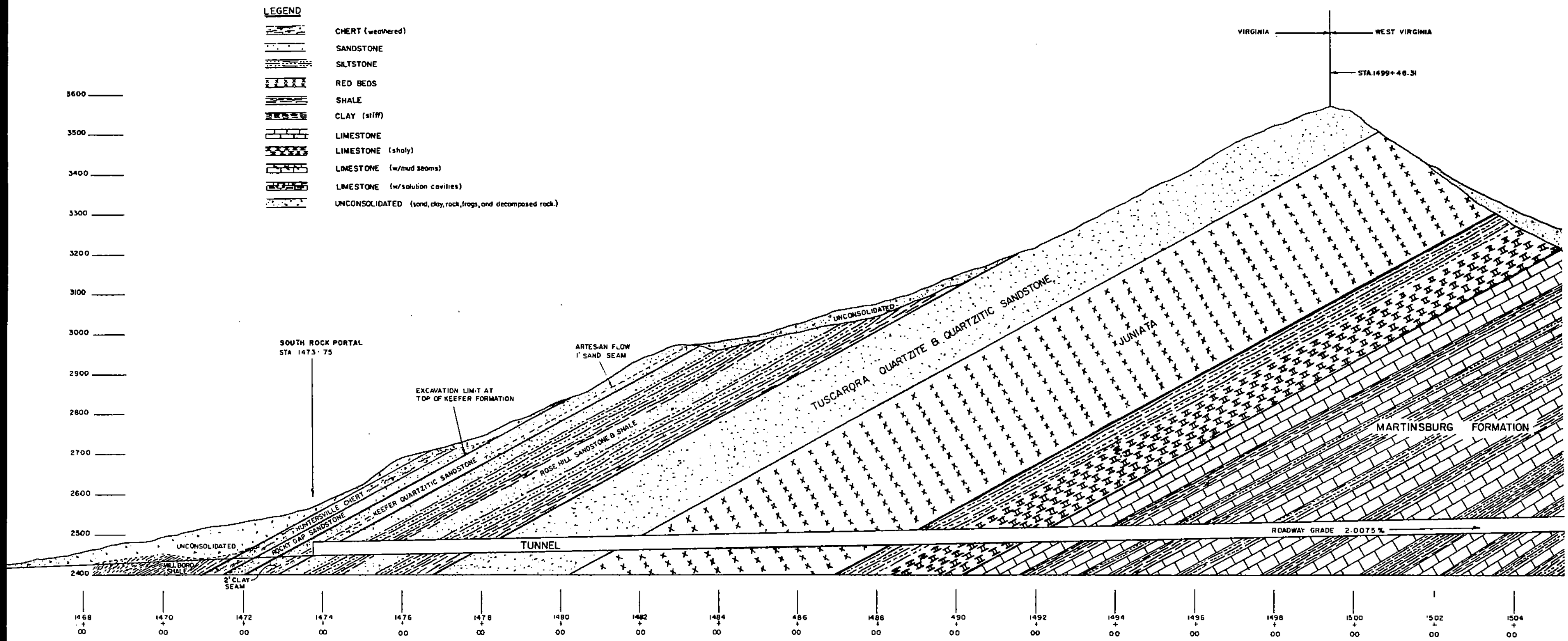
FIGURE 3
GENERAL GEOLOGIC STRUCTURE
EAST RIVER MOUNTAIN AREA

southern (dip) slope. The average formational strike is N66°E and the general dip is 28° South.

The formations encountered along the tunnel alignment vary from the Huntersville chert, Onondaga age - Middle Devonian (South Portal) through the Benbolt limestone, Black River age - Middle Ordovician (North Portal). The intervening formations include quartzite, sandstones, limestones and shales.

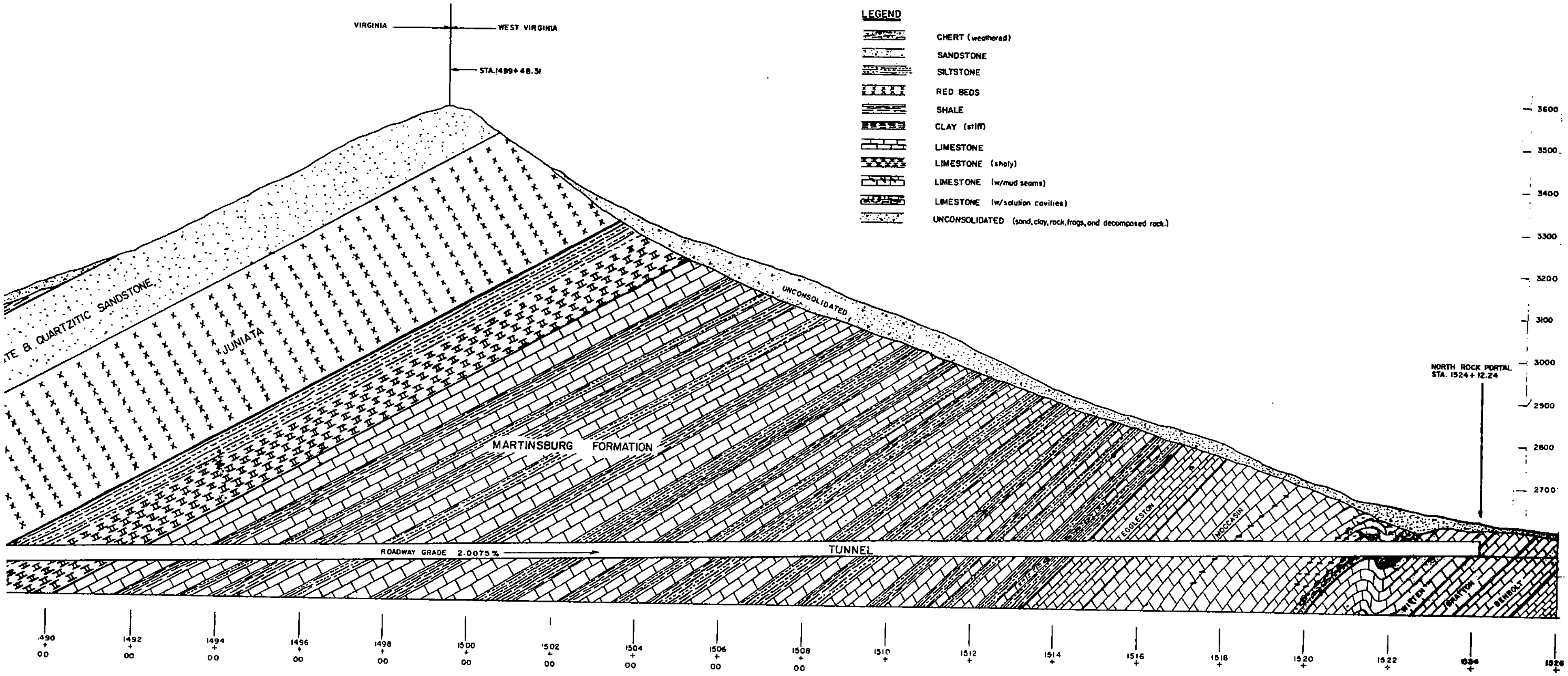
The sub-surface conditions as determined by the core borings indicate a considerable thickness of scree, decomposed shale and sandstone, sand and clay in the area of the South Portal (Figures 4A and B).

Although the previously mentioned general dip is 28° South, approximately 1,400 feet south of the North Portal the dip begins to steepen with the maximum approaching the vertical in a folded area in the vicinity of Station 1521+80. From the folded area, north to the portal, the dip gradually becomes less steep approaching 37° South at the North Portal.



GEOLOGIC PROFILE AT CENTER LINE OF TUNNELS
EAST RIVER MOUNTAIN
SCALE- 1"=100'

Figure 4a



GEOLOGIC PROFILE AT CENTER LINE OF TUNNELS
EAST RIVER MOUNTAIN
SCALE- 1" = 100'

Figure 4b

A line of well-developed sinkholes, that trend nearly parallel to umberland Road, are located a few hundred feet to the north of the North ortal. The rock portal is in an area of southerly dipping hard, massive imestone.

Two small areas, one at 1,000' and the other at 625' south of the North ock Portal and a third, more extensive area (extending 150' to 380' South of he rock portal) were detected by the borings to contain mud-filled seams nd/or solution openings. The extent and location of these areas were deter-ined by core-logging and projection. Although most of the cavities were ud-filled, a few were found to be open with the size range of both varieties unning from 2 inches to one which measured 7 feet.

When discussing design requirements for tunnels or similar projects, e must realize what is desirable for design is not always convenient for onstruction, in many cases we have to shift locations of structures in rder to take advantage of a better location as dictated by the local geo-ogical conditions.

In developing an economical as well as safe design it is essential or the geologist to be able to communicate with the design engineer; each hould have some knowledge and be more than familiar with the other's field f endeavor.

Before we go into how geological features have affected the design f the tunnel project I would like to bring out one point -- the human actor.

A great deal of the preliminary work on a project such as this in-olves getting along with people, in consultant-client relationships and in ealing with landowners on whose property you plan to run surveys or take core borings, just to mention two. As you know, it takes some degree of diplomacy to explain to a disgruntled property owner why you are using his taxes to acquire his land.

Along this same line, I feel we are quite fortunate because problems that have developed on this project were not due to any breakdown in communication between our company and the State Road Commission. We both have an appreciation of what can be encountered in a project of this magnitude.

Our company, Michael Baker, Jr., Inc., works directly with the West Virginia State Road Commission, who in turn deals with the Commonwealth of Virginia and the U. S. Bureau of Public Roads - you might say that this project is a double-jointed venture.

Now, as to how geology has influenced the design of this particular project:

One of the first steps in tunnel design is portal location, portaling against a steep, natural slope in poor material can be very risky, and should be avoided unless precautions are taken to prevent disturbances to the original slope above. Mainly, because of drainage and sedimentation resulting from drainage, valleys or draws are not considered favorable locations and topographic noses are preferred.

In our case, the two additional 6" diameter boreholes previously mentioned were used to investigate a topographic nose, this projection is located several hundred feet East of the final South Portal location.

The boring results and subsequent tests revealed this spur or nose was protected by a thin layer of Huntersville chert which was sufficient resistance to retard erosion, but the underlying Rocky Gap sandstone was almost devoid of cohesion, since the cement had been leached out. Unconfined compression tests licated this formation had no rock-like characteristics; in fact, it subject to flow failure if initiated by shock or a rapid rise in the lraulic gradient.

As a result of these findings we moved back to our first loca- on in which our plan had been to excavate all the unconsolidated material, :ee, sand, clay and the Huntersville and Rocky Gap, thus exposing the efer sandstone which would provide the required thickness of competent :k over the tunnel arch to permit safe and conventional mining opera- ons.

Excavating back to the Keefer has several other advantages, the :onsolidated material would be removed, thus reducing the possibility landslides, the water table would be lowered to a point where it could easily handled during tunneling operations and the foundation condi- ns provided by the Keefer, Rocky Gap and Huntersville would be adequate the South ventilation buildings.

After considerable work had been done using this approach, we e informed that the excavation required would be more than had been icipated and in addition would mar the natural beauty of the South pe. In keeping with the idea that beauty lies in the eyes of the be- der, I personally think a broad expanse of exposed rock is beautiful.

Although requiring less excavation, the alternate design for the th Portal involves over 300 lineal feet of soft-ground tunneling for h "bore", that is, Northbound and Southbound tunnels.

Southward from the Keefer-Rocky Gap contact only the Huntersville chert could be considered partially self-supporting. In addition to soft-ground tunneling, this area will add at least three other design features - First, a method to lower the water table in the soft-ground area prior to mining. This we believe can be accomplished by means of two 36" diameter drainage galleries, each one located just outside and approximately 4 feet below the invert of each tunnel. We anticipate that a considerable volume of ground water will be encountered while mining in the South Portal area; this fact became quite evident when an initial flow of 1,000 gpm was measured coming from an NX borehole in the South Portal area. After determining the location, thickness, and possible head on the aquifer, calculations indicate that possible initial flows while mining may approach 15,000+ gpm.

Second, the portal for the soft-ground section will require an interlocking, steel sheet-pile retaining wall with heavy braces or rakers to restrain the considerable volume of overburden back of the portal. Each soft ground tunnel will be approximately 40 feet in diameter; therefore, the piling length must be on the order of 70 feet to allow sufficient depth for anchoring, space for the tunnel opening, plus the overburden above the tunnel crown. When you consider the tunnels are spaced 70' center to center, plus the 40 foot diameter openings, plus extensions right and left to accomodate the transition to the ventilation buildings you have a retaining wall of considerable proportions.

The Third item involves a change in foundation conditions for the South Ventilation building. The southward shift of the tunnel portal has placed the ventilation building over an area where sound rock is at a depth that will require a pile foundation.

I would like for a moment to go back to the subject of ground water, ɔt only in connection with South Portal but for the entire tunnel.

Groundwater can be an important element in the overall cost of a ınnel and its occurrence should be carefully noted. With low permeability ɔrmations and tight joints water flows are small, but with extensive joint-ıg, shear zones or formations that are good aquifers, the presence of :ound water can create very difficult tunneling conditions. Designers are ıterested in knowing as many of the details as possible in connection .th the occurrence of ground water. These should include pH values, ıemical analyses, estimated volumes, possible locations and in some ıstances, temperature. Using this information the designer can make a ːasonable estimate for dry pack, grouting, "panning" and other design ːatures that may be necessary.

The tunnels, during and after construction, are expected to exert ɲnsiderable influence on individual domestic water wells in the vicinity : the South Portal and according to information I have received, the ɲmmonwealth of Virginia has begun a ground water survey in this area.

The conditions which demand attention in the North Portal can : attributed strictly to the solubility of a few of the limestone forma-.ons.

The mud-filled seams and solution openings occur in three loca-.ons. The first and most extensive beginning approximately 150 feet south : the North Portal and the smaller ones at 625 and 1,000 feet south of the ɲrtal were detected by core-boring.

The largest individual opening was measured at 7 feet (in a vertical borehole), but the total void space in a single area may be greater, separated only by thin layers of limestone.

It has been recommended that continuous roof and, in some instances, rib support be provided by linear plates in conjunction with 2 foot rib spacing in these areas. Backpacking may be required to fill some of the larger solution openings and in the relatively dry areas reinforced "gunnite" or "shot crete" may be used.

In areas where clay and/or mud seams are present at the floor of the tunnel, sills may be used to span the soft area and provide a firm footing for the steel supports.

Immediate support should be provided, in some instances crown bars, when clay seams are encountered at the roof line.

Beginning approximately 125 feet South of the Portal, maximum support will be required for about 100 lineal feet, this is necessary because of an area of very deep weathering near the Moccasin-Witten contact which has left only a thin rock cover over the tunnel.

As you drive along Cumberland Road from Bluefield and approach the tunnel area, a very noticeable feature is the line of sink holes that have developed between the Road and the north slope of East River Mountain.

These sinkholes intercept most of the north slope runoff, much of this flow then finds its way by underground channels into the Bluefield Water Supply Reservoir. We are, therefore, recommending that during the open cut excavation phase of construction, that the contractor provide a desilting basin r similar facility to prevent any sediment-laden water, either from work r waste areas, from entering the sinkholes.

When preparing a report for a project, such as the one I have been iscussing, it is well to keep in mind, that aside from design informa-ion, an accurate appraisal of the geologic conditions permits the consultant r client, as the case may be, to develop an accurate estimate of cost and ɔ the prospective contractor this same appraisal means a more realistic id.

GEOLOGY OF BIG WALKER MOUNTAIN TUNNEL ON INTERSTATE ROUTE 77, WYTHE AND BLAND COUNTIES, VA.

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Blacksburg, Virginia

The Big Walker Mountain Tunnel project on Interstate Route 77 in Wythe and Bland Counties, Virginia, constitutes one of the first two major tunneling projects in developing the interstate highway system. Doubtless it will be the answer to the private prayers of many hapless motorists and truck drivers who have crossed this formidable ridge by way of present U. S. Routes 21 and 52 during a driving rain or snowstorm. The total tunnel project costing about $23,000,000 will rank easily as the most expensive 0.8 mile of highway in the Virginia highway system.

Walker Mountain is in many respects a typical Appalachian ridge. The mountain possesses one rather unusual feature, namely, numerous sharp deflections in trend from the usually northeastward trend to a strongly southeastern trend. The formations exposed on the ridge form the homoclinal or the northwest slope of a major synclinorium, parts of which are overridden by the Pulaski overthrust block. Walker Mountain is on the Saltville fault block which is composed of a Paleozoic succession at least 14,500 feet thick. It is made by the Tuscarora Formation of Medinan age and possesses the usual two sets of characteristic flatirons on the dipslope or southeastern side. The upper line of flatirons is made by the Keefer orthoquartzites. A lower line of somewhat rounded knobs is made by the Huntersville cherts of Onondaga-Schoharie age (Fig. 1).

Formations shown on the accompanying cross section (Fig. 2) delineate the geology along the tunnel line.

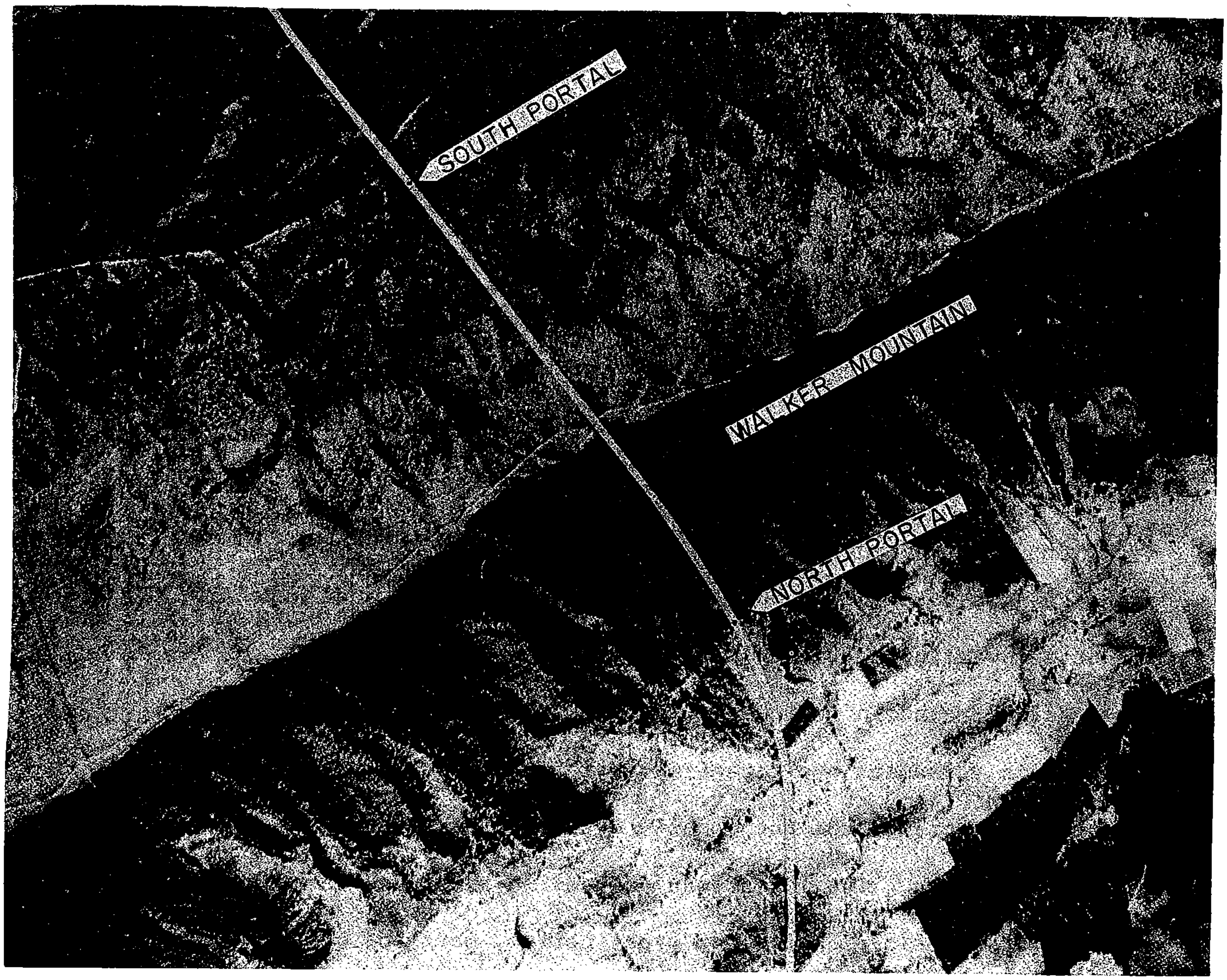

Fig. 1. Photograph of Walker Mountain showing location of tunnel line of Interstate Route 77, Bland County, Virginia.

Nearly all of the Martinsburg Formation -- 1,600 feet thick -- will be penetrated, except for the lowermost 150 to 200 feet. Locally the Martinsburg is deeply weathered to soil and still deeper to an oxidized, leached, but often rock-hard residue. The aggregate thickness of weathered material ranges up to 200 feet.

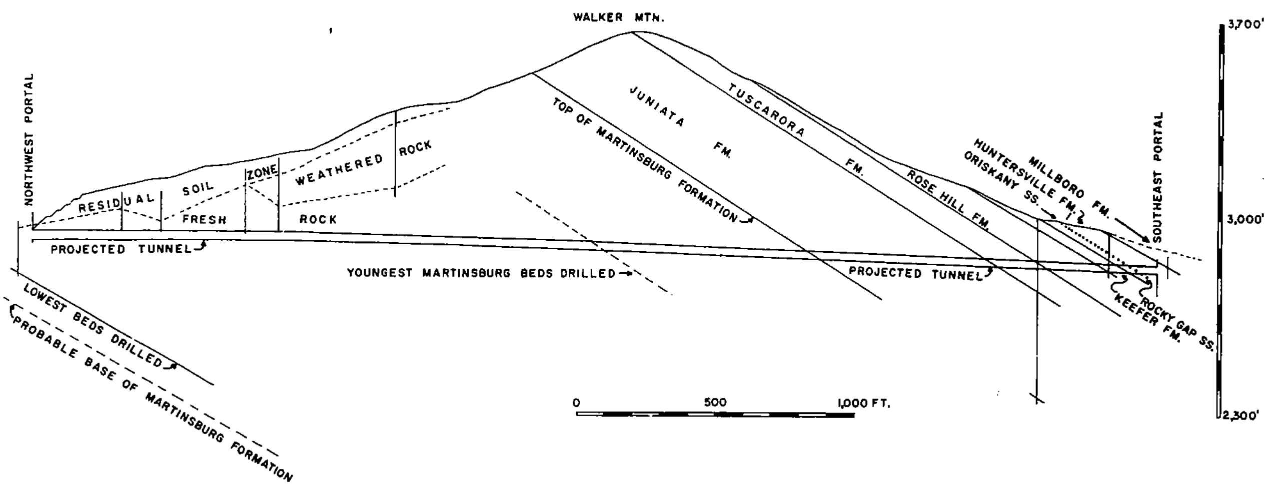

Fig. 2. Geological cross section along projected tunnel line through Walker Mountain showing dip and thickness of formations penetrated. SE is on right, NW on left.

Other, younger formations involved in the tunnel are the Juniata sandstones and shales -- 320' thick, Tuscarora orthoquartzites -- 85' thick, Rose Hill shales and ferruginous sandstones -- 110' thick, Keefer orthoquartzite -- 40' thick, Rocky Gap sandstones -- 26' thick, a thin very ferruginous sandstone of Oriskany age -- 6' thick, and the Huntersville cherts and glauconitic sandstones - about 40' thick. The tunnel will be driven from the southeast portal.

The center line of the tunnel was originally charted to head into a ravine which contains coarse colluvium dominated by large blocks of Tuscarora orthoquartzite -- some up to 15 feet in greatest dimension. The writer was called in by Singstad and Kehart, Inc., after considerable drilling had been done along the original tunnel line. The effect of the colluvium was to encourage deep leaching of the Martinsburg limestones and shales far below the level of the floor of the projected tunnel. This situation was appreciated in the initial visit to the site in 1964 and in view of the conditions encountered in the ravine, the tunnel line was shifted four hundred feet to the southwest so as to benefit from the existence of fresher rock at elevations above the roof of the tunnels. This added slightly to tunnel length but avoided much more costly measures that would have been necessary to drive the twin tunnels along the original line.

Perhaps the most significant feature revealed by all the core drilling done in the Martinsburg Formation was the profound depth of the leached and oxidized succession of the Martinsburg beds. As elsewhere in this general portion of the Appalachians, the Martinsburg consists of impure thin limestones, mostly biosparites, which are intercalated in black pyritic shales. Numerous fractures in the limestones encourage ingress of percolating waters which, circulating in contact with slightly calcareous, pyritic shales, generated sulfuric acid promoting profound dissolution of the limestones and the leaching and oxidation of the shales into a brown-colored residue varying in consistency from rock-hard to soft, loose silt. The cutoff point for leaching and oxidation of the Martinsburg is sharp wherever revealed by drilling and no known recurrences of weathered zones at greater depths were found in any of the drill holes. The dotted line on the cross section shows the thickness of the overburden on the Martinsburg. The Martinsburg biosparites megascopically appear to be fairly pure limestones, but the limestones yield a residue of silt and clay sufficient to fill loosely much of the space occupied by the fresh rock, and bedding is obvious in the leached material.

It is, of course, axiomatic that fresh rock occurs closer to the surface under hills or ridge spurs than under adjacent ravines, and such is the case as revealed by drilling done into the spur beneath which the tunnels as relocated will be driven.

Actually, the relatively great depth to which the Martinsburg is decalcified even beneath the surface of the spur to be penetrated by the relocated tunnel surely must be controlled to a large extent by the depth of circulation of percolating waters under the colluvial fills in adjacent hollows such as the one immediately to the northeast under which the tunnel as originally projected was to run. Much of the water ingressing on mountain spurs of Martinsburg limy beds must move relatively far down dip only because the circulation probably turns along strike, with the waters eventually becoming part of the underflow draining through the colluvial fills. The normal condition of Martinsburg beds is invariably one of close-spaced fractures, so that initial percolation is facilitated. Probably the great depth to which Martinsburg beds are decalcified under the colluvial deposits in the ravines is a function of the steepness of all the obsequent slope of Walker Mountain and the great range in elevation between the mouth of the hollow and the colluvium-bedrock contacts upstream.

Few if any road cuts in southwestern Virginia penetrate the leached, decalcified residuum developed on the Martinsburg Formation. The fresh cores of shale-limestone successions in that formation reveal a feature not generally appreciated by geologists working in the Appalachians. The absolutely fresh Martinsburg shaly beds are black and quite pyritic. Shades of green, gray, tan, and brown so commonly seen in supposedly fresh cuts are not indicative of the true color of the fresh rock but are indicative of varying degrees of weathering. The shales obviously weather faster than the limestones. The surprising thing about the latter is their high content of insoluble silt and fine sand. The residue left after complete decalcification of the biosparite limestones occupies nearly the full thickness of the original limestone layers.

Judging how these formations appear under heavy cover as shown in one deep core hole, really fresh Juniata, Tuscarora, and Keefer sandstones are seldom seen in natural exposures. Pyrite is surprisingly abundant in the cores taken from these formations. It no doubt accounts for the rusty blotches commonly seen on fracture surfaces and in case-hardened surface coatings on Tuscarora exposures. The abundance of pyrite suggests that the formation instead of having been deposited in the beach or intertidal zone, as many have believed, may have been deposited as an extensive submarine fill in which the bulk of the sand was laid down well below wave base in a reducing environment.

Pyrite is also abundant in Juniata beds which are decidedly less abundantly red than as seen at the surface. Greens are commonly associated with the pyritic Juniata sandstones. On the other hand, the Rose Hill or Clinton Formation which is also surprisingly pyritic shows so much pyrite even in the hematitic sandstones of Cacapon lithofacies that one is puzzled about the actual source of the characteristic red color. I would have to subscribe to the theory that the iron was probably all introduced as sulfide but was largely, but by no means wholly, converted to hematite during diagenesis.

Pyrite also occurs in the fresh Keefer as evidenced by deep cores in Bore Hole No. B-28. It is also present in the Millboro shales including lower beds of Needmore affinities. The Juniata-Huntersville succession, unlike that on

East River Mountain to the northwest, contains no limestones or highly permeable fresh rock. The Huntersville-Millboro contact zone is a significant zone of water percolation as evidenced by the 6- to 20-foot zone of oxidized, decalcified, and partially bleached shale at the base of the Millboro. The Huntersville-Millboro contact zone is probably a zone of localized fractures that has piped water down dip. Contact of this circulating water has oxidized the pyrite in the black shale and has converted the normal illite or hydromica particles in the shale to kaolinite as a result of the prevailingly high acidity created by oxidation of the sulfide to sulfate. This is the zone near which the south portal is located. From the borings it is impossible to predict whether the upper part of the Huntersville may have been weakened by percolating water.

The Huntersville Formation is undoubtedly the most interesting rock in the succession. It is more or less a mass of dense, fine-grained sandstones, thin curls of chalcedony, and a chert-like layers containing disseminated iron-rich carbonate rhombs and glauconite, and very glauconitic coarse-grained sandstone. So diverse are the lithologies in the Huntersville Formation that a full description of all lithic types might be titled "Is The Huntersville Chert?"

The Keefer orthoquartzites, every bit as hard as the Tuscarora beds, contain a characteristic granule conglomerate at the base, and associated layers contain abundant interstitial black, iron-rich chlorite. The Rose Hill beds show relatively, turbid iron-rich carbonate as a cementing material, which is rather closely interspersed with pyrite and also hematite. The Tuscarora orthoquartzites are also notably pyritic; this is the probable source of the iron occurring as limonitic stains on weathered surfaces. The Juniata is a much more quartzitic and structurally tough rock than one would infer from surface exposures; much of it is greenish-gray in color. Pyrite is abundant. It is not very red underground.

The rusty, ferruginous Ridgeley or Oriskany is a permeable grit and is enough of an aquifer to supply water for personnel operating tunnel facilities.

From all indications, the structure of Walker Mountain is a relatively simple homocline. All the drill data along the tunnel line fell into place so nicely that no other interpretation could be made. Doubtless disharmonic crumpling in portions of the Martinsburg will be encountered but these are expectable. They will not affect the overall structure as inferred.

Strength tests have been made on selected cores. Some of the quartzites have crushing strengths of over 80,000 pounds/square inch. The weakest units are the Oriskany Sandstone and Martinsburg shales -- both of which rupture with vertical fractures. Martinsburg limestones, Rocky Gap sandstones, Huntersville beds, Rose Hill sandstones, Tuscarora quartzites, and Juniata beds yielded excellent shear cores under rupturing compression tests. Rock strength will not be an issue in driving the tunnel. All the beds are strong enough to meet structural needs.

Acknowledgments

The writer is grateful to Mr. Nathan November of Singstad and Kehart, Consulting Engineers, New York City, for permission to present this brief summary.

THE GEOMETRY OF LIMESTONE AGGREGATE SOURCES IN KENTUCKY'S APPALACHIAN REGION

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Introduction

The Appalachian regional development program, with its proposed construc- :ion projects and potential resulting private development in the region, has :ocused considerable attention on the location of sources of aggregate ıaterials. As defined by Public Law 89-4, March 9, 1965, the Appalachian 'egion includes 49 of Kentucky's 120 counties (Fig. 1). Limestone is presently .nd will continue to be the principal source of aggregate material in this 'egion.

Though the outcropping rocks range in age from Ordovician through 'ennsylvanian, the sandstones, siltstones, and shales of the Pennsylvanian ominate the surface exposures in the largest part of the region, thus limit- ng the areas of limestone. In addition, folding, faulting, erosional uncon- ormities, facies changes, and irregular depositional patterns have produced imestone bodies of varied dimensions and attitudes. These geological factors re reflected in the pattern of distribution and have created problems in the ocation of commercial deposits of limestone aggregate materials in the region.

The region, as defined above, involves several geological provinces. 'he principal structural features which control the geographic distribution of he limestone resources are the Cincinnati arch, Appalachian basin, Pine :ountain fault, and Paint Creek uplift (Fig. 2). The resulting principal lime-

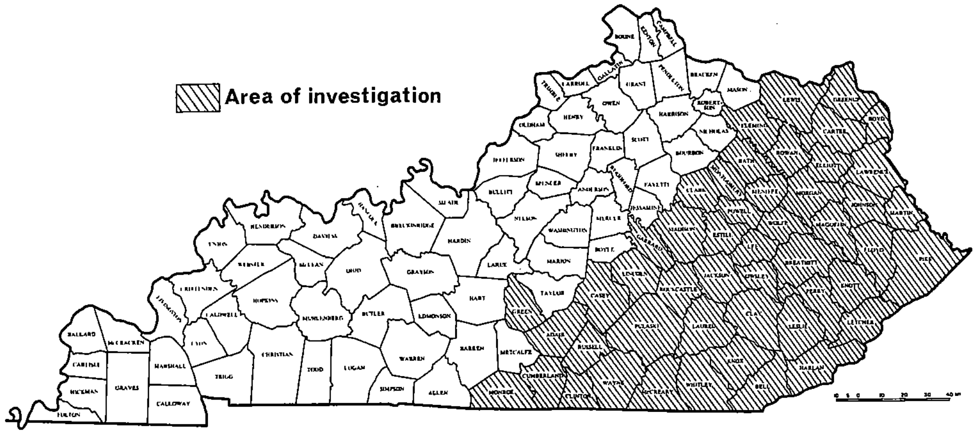

Fig. 1. Map of Kentucky showing counties included in the Appalachian region as defined by Public Law 89-4.

stone area are the narrow belts along the western edge of the Eastern Kentucky Coal Field and the front of the Pine Mountain overthrust block, the Kentucky River area of the Inner Blue Grass, and the Mississippian Plateau of the south-central part of the Commonwealth.

At the present time there are more than 40 active quarries, pits, and mines located in 28 of the Appalachian area counties. Stratigraphically, they are limited to rocks of the Mississippian and Ordovician, with the former predominating. Almost 50 percent of these active operations are partially or entirely in the St. Genevieve Limestone (Fig. 3). Limestones of the Lower Chester appear equally important.

The distribution of limestones offering the greatest potential for concrete and road aggregates is shown in Figure 4.

Kentucky River Area of the Inner Blue Grass

The combination of the Jessamine dome and the deep entrenching of the Kentucky River has exposed the Middle Ordovician Tyrone and Camp Nelson Limestones along the incised valleys of the river and its tributaries. These limestones are hard and dense and produce good aggregate. With the main valley

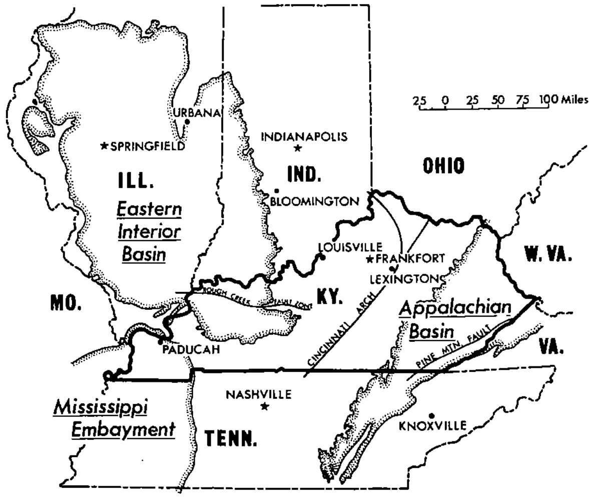

Fig. 2. Regional setting of Kentucky showing geologic features controlling the occurrences and distribution of limestone resources.

attaining depths of 300 to 400 feet below the Lexington Plain, underground operations are frequently the most satisfactory methods of recovering this rock. The locating of a quarry site can be further complicated by the presence of the Kentucky River fault system, which roughly parallels part of the course of the river and may limit the size of an operation. The meandering course of the Kentucky River crosses the fault system several times, resulting in a series of discontinuous exposures of the potential aggregate rock. An individual exposure may represent a large deposit or only a small, isolated block on the tip of a meander bend (Fig. 5).

Younger Ordovician limestones are quarried locally elsewhere in the region. In general, these rocks, being somewhat argillaceous and containing numerous shale partings, must be beneficiated before passing state and federal specifications for use in concrete aggregate for highways.

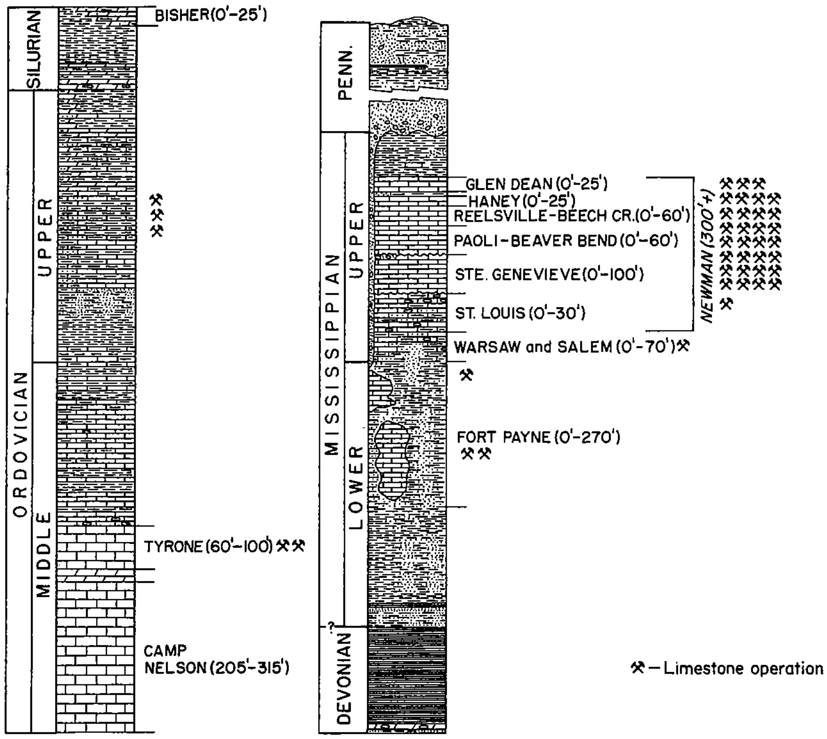

Fig. 3. Generalized geologic section of the Appalachian area of Kentucky showing the stratigraphic positions of limestone operations and thicknesses of commercial stone.

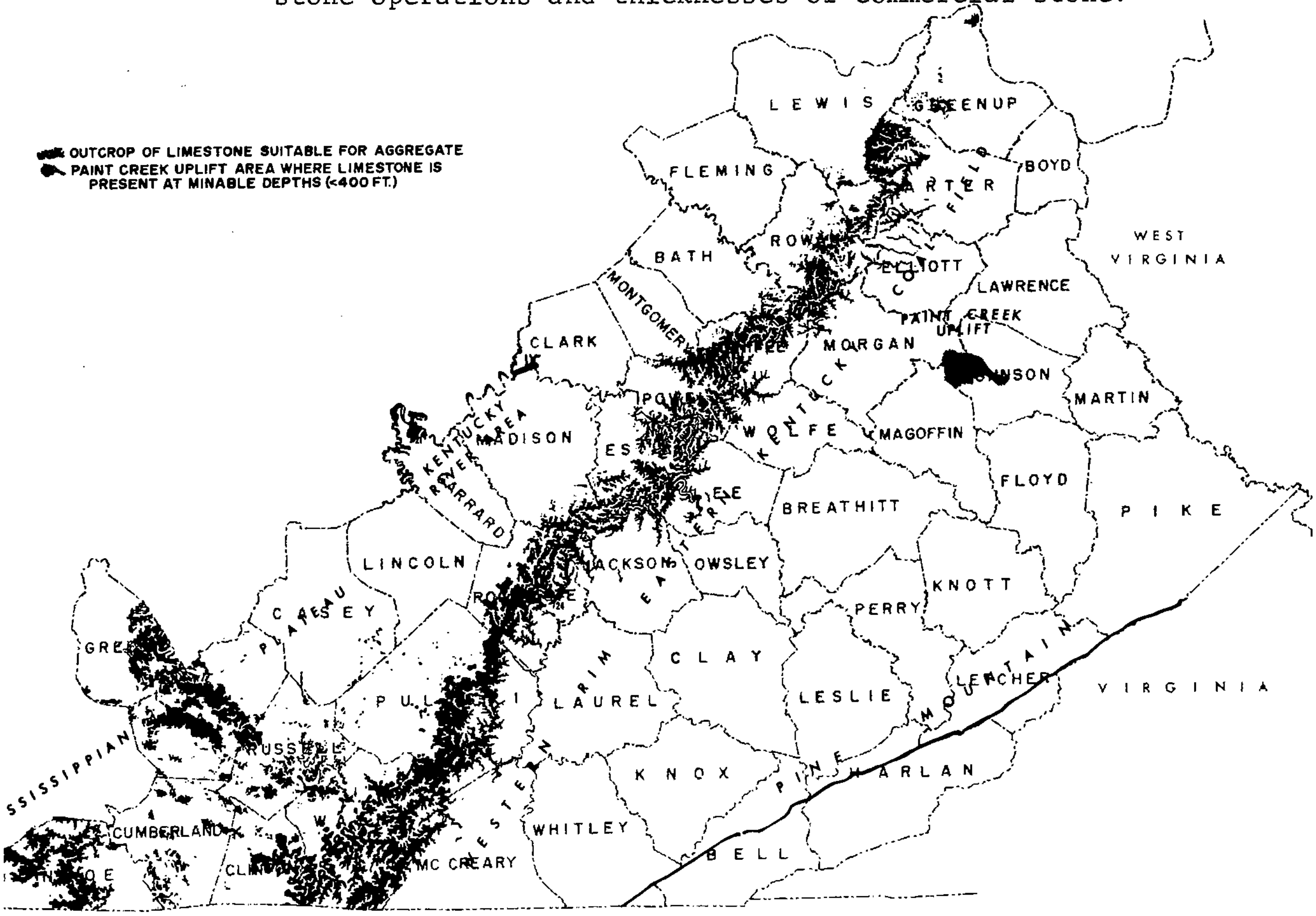

Fig. 4. Map of Appalachian region of Kentucky showing principal areas of occurrence of limestones for concrete and road aggregates (adapted from McGrain and Dever, 1967, Pl. 1).

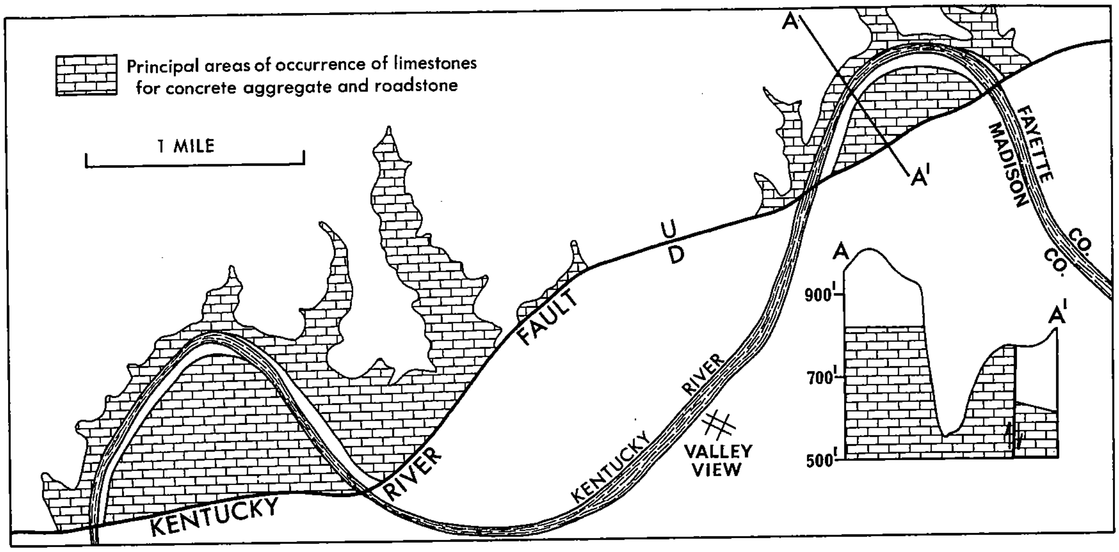

Fig. 5. Portion of Kentucky River valley in central Kentucky showing relation of limestone aggregate sources to Kentucky River fault. Adapted from geologic map of Valley View quadrangle (Greene, 1966).

Mississippian Plateau of South-Central Kentucky

This area is largely on the Cumberland saddle of the Cincinnati arch. :he exposed rocks range in age from Late Ordovician through Pennsylvanian, ,ith the Early Mississippian predominating. The older rocks are exposed ılong the valleys of the Cumberland River and its major tributaries, and successively younger rocks are exposed to the east and west.

The Fort Payne Formation, though primarily a clastic, is of particular :nterest because of its varied lithologies and abrupt facies changes.)perable thicknesses of limestone in the Fort Payne have been brought to Light by the cooperative geologic mapping program of the Kentucky and United States geological surveys (Fig. 6). Lenses and tongues of reeflike limestone vith irregular thicknesses and horizontal dimensions are present in several counties. The geometry of the larger bodies, whose areal extent is measured

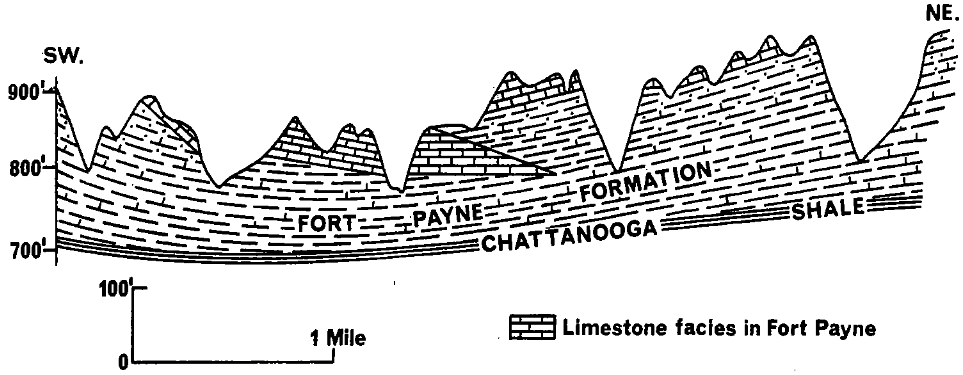

Fig. 6. Geologic section showing limestone bodies in the Fort Payne Formation. Vertical scale exaggerated. Adapted from geologic map of Montpelier quadrangle, Adair and Russell Counties (Lewis and Thaden, 1964).

in terms of miles, is that of an elongated lens, with the long axis trending northwest-southeast.

Locally, the St. Louis and Warsaw formations may contain strata satisfactory for quarrying. In general, however, these formations are argillaceous and the stone fails soundness tests. The Upper Ordovician limestones are similar lithologically to their central Kentucky counterparts and should be beneficiated before use in concrete aggregate. Upper Mississippian limestones may occur as small deposits on isolated knobs.

Western Rim of the Eastern Kentucky Coal Field

Upper Mississippian carbonates constitute the most important source of aggregate in Kentucky's Appalachian region. Their principal exposures are in and near the Pottsville Escarpment, extending northeastward across the state, along the western edge of the Cumberland Plateau. The Ste. Genevieve Limestone is the most important quarry rock and a majority of the quarries in this area are partially or entirely in this formation. The superjacent Chester limestone edges may be quarried with the Ste. Genevieve or operated separately, epending upon availability and the topographic situation. In their broad-r aspects, the limestone units form a huge, wedge-shaped body along the ength of the outcrop belt (Fig. 7). These aggregate-producing formations ttain their greatest thickness in the south, in the Lake Cumberland area f Pulaski, Wayne, and Clinton Counties, and decrease northward. This is due n part to the thinning of individual units and in part to pre-Pennsylvanian nd pre-Chester erosion. Numerous variations in thickness occur from Rowan 'ounty northward where a part or all of the carbonate sequence may be missing with basal Pennsylvanian clastics resting on Lower Mississippian siltstones), hereas a few miles away limestone thicknesses as great as 80 to 100 feet ay be measured. Detailed field investigations together with core drilling re necessary in this area to make a quantitative determination of the lime-tone resources in any specific locality.

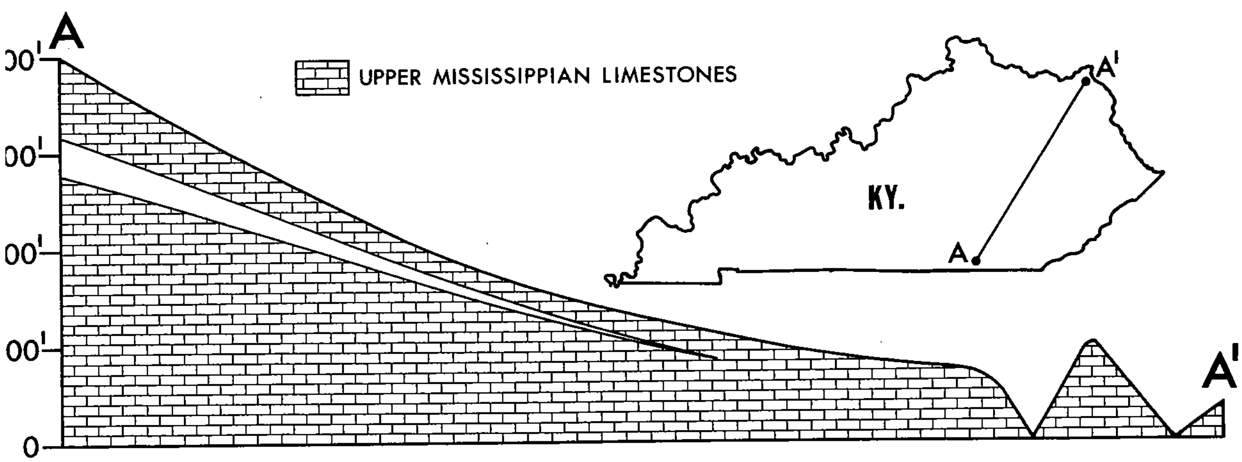

Fig. 7. Sketch showing variation in thickness of the Upper Mississippian limestone section from Clinton County on the Tennessee border to Greenup County on the Ohio River.

Two other conditions that may affect the size and location of quarry operations along the Pottsville Escarpment should be noted. Displacement along the Irvine-Paint Creek fault system may restrict the size of a potential operation in or near the fault zone which extends through Wolfe, Powell, Estill, and Madison Counties. And, generally, quartz sand and chert granules are found in increasing amounts in the Ste. Genevieve formation northward from Wolfe County.

Pine Mountain

The Upper Mississippian limestones are exposed again to the southeast by the Pine Mountain overthrust. They are expressed as a steeply inclined block of carbonate rocks more than 300 feet thick (Fig. 8) that extends along the fault scarp for a distance of approximately 100 miles from near Jellico, Tennessee, northeastward to the vicinity of Elkhorn City, Kentucky.. Except at the water gap at Pineville, the limestone unit is uninterrupted along its entire extent.

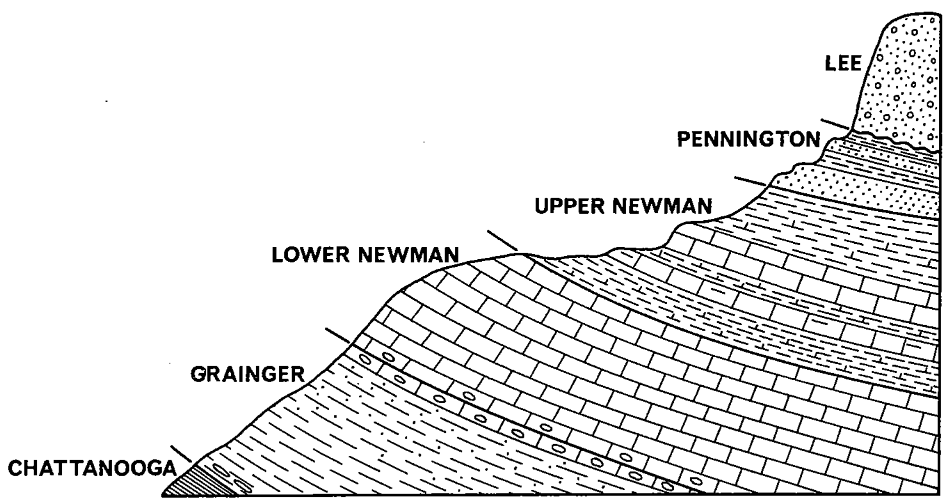

Fig. 8. Generalized geologic section of Pine Mountain.

This sequence of carbonate rocks has been referred to as Lower lewman and as Greenbrier. For the most part these rocks appear to be :quivalent to the Ste. Genevieve and Chester limestones of the western :dge of the Cumberland Plateau, and represent a large reserve of potential .ggretate material. Lithologies similar to the St. Louis have also been ıbserved. The rugged topography, thick overburden, and steeply dipping ·eds pose the greatest problems in operating these deposits.

Paint Creek Uplift

In parts of Johnson, Magoffin, and Morgan Counties, in the middle ·f eastern Kentucky's coal field, the Upper Mississippian limestones may e found within less than 400 feet of the surface (Stokley, 1949; Hauser, 953; McGrain and Dever, 1967). Limestone has been brought near the surace in an area generally devoid of aggregate sources by the Paint Creek nticline, a faulted uplift. Near the crest of the uplift the limestone eposit ranges in thickness from 30 to 150 feet and represents a substanial reserve of aggregate material for the area. Depths to this stone ncrease sharply away from the uplift. Core drilling is necessary to etermine the depth, overburden, quality of stone, and subsurface water onditions.

Conclusions

It is reasonable to assume that the construction of roads, dams, nd other structures will accelerate in the Appalachian region in the years ahead. Large supplies of limestone suitable for many industrial purposes are available in a number of Kentucky's Appalachian counties, but they are not evenly distributed over the whole area. Knowledge of the geographic distribution and dimension and attitude of limestone deposits will provide a basis for fuller development of the resource. Proper utilization of this resource should provide considerable savings to major construction projects.

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THE GEOLOGY OF CONSTRUCTION SAND AND GRAVEL RESOURCES OF WEST VIRGINIA

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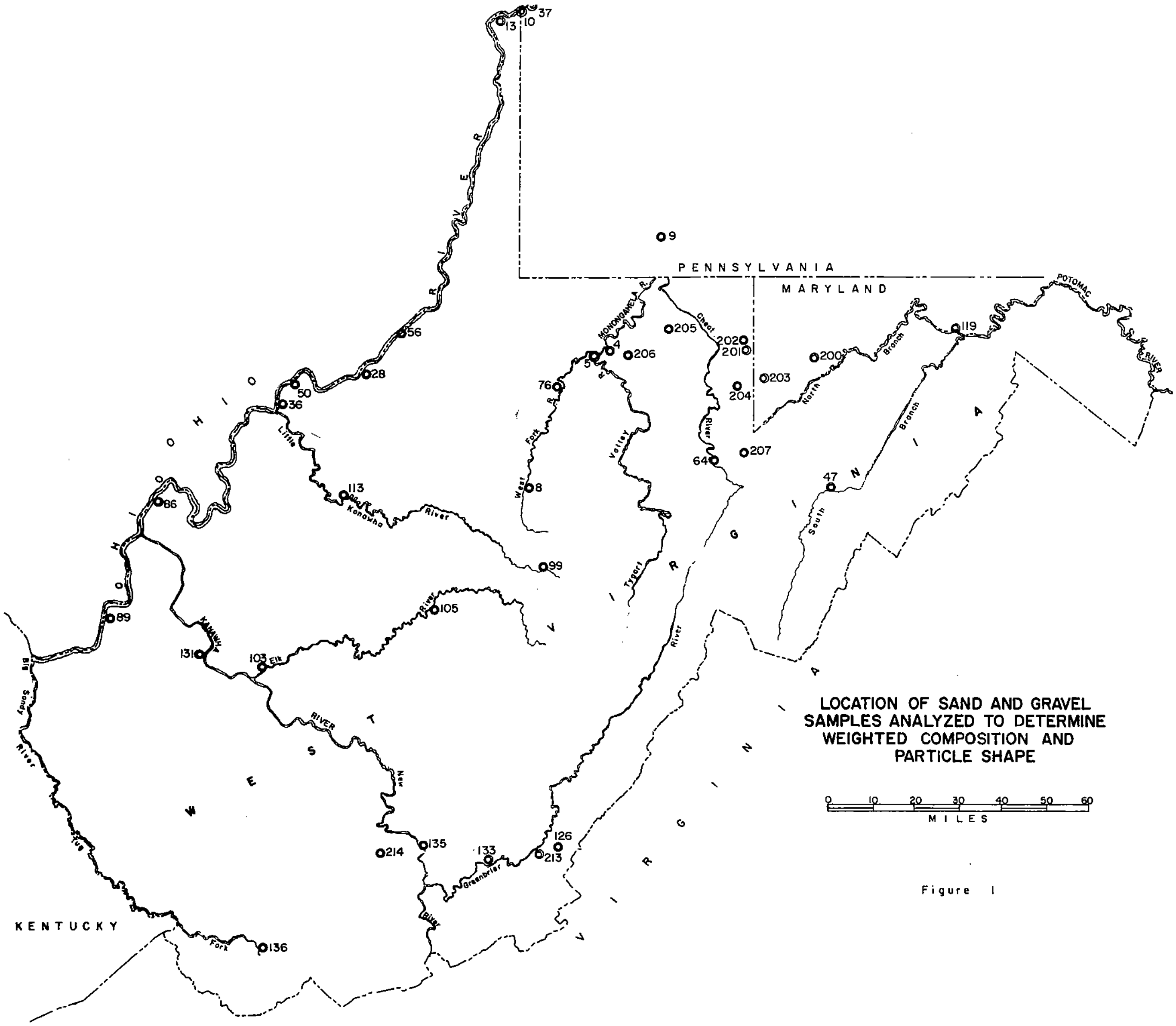
Introduction

In this study, 191 samples were collected from alluvium, residuum, and bedrock at 178 sampling sites. In addition, test results of 72 samples from The Inventory of Highway Aggregates by Seeger and Kent were incorporated in the study. The conclusions of the study are based on 246 alluvial samples, 2 residual samples, 1 sample of talus and colluvium and 14 samples from ortho- and proto-quartzites (sandstone). Seventy-three percent of alluvial samples were of fluvial origin; the remainder were of glaciofluvial origin. The quality of 207 samples was determined and petrographic studies of 13 glacio-fluvial, 19 fluvial and 13 sand (crushed from sandstone) samples were made (Figure 1).

Samples were collected from terraces and bedloads of all major streams of West Virginia and from clean-washed sandstones, believed to be suitable for the manufacture of construction sand. The State Road Commission conducted quality tests on the samples as received and the staff of the West Virginia Geological Survey performed petrographic analyses to determine the mineral content, condition of the constituents, viz., good, fair or poor, and the roundness or angularity of flat, equidimensional and elongated shapes.



LOCATION OF SAND AND GRAVEL SAMPLES ANALYZED TO DETERMINE WEIGHTED COMPOSITION AND PARTICLE SHAPE

Figure 1

In this report, alluvial sand and gravel are unconsolidated materials resulting from the natural disintegration of rocks and sand and gravel processed from friable sandstones or weakly bound conglomeratic sandstones (ASTM designation C125-66). Sand (fine aggregate) is $<$ 3/16-inches in diameter (-4 mesh, 4.76 mm.) and gravel (coarse aggregate) is $>$ 3/16 inches in diameter. Although the upper limit of gravel sizes for most commercial purposes is less than $3\frac{1}{2}$-inches in diameter, deposits of gravel with sizes up to 6-inches in diameter were representatively sampled. No attempt was made to sample deposits with sizes greater than 6-inch in diameter. Since it would be necessary to crush and prepare these coarse materials for use in construction, a study of materials coarser than 6-inches in diameter would more logically be included in a study of aggregates crushed from sandstones than in a sand and gravel study.

The stratigraphic sections of alluvium, containing samples, were measured, the areal extent of the deposits (inferred and possible) were measured on aerial photographs and topographic maps with a planimeter, and the possible reserves of the deposits computed in cubic yards.

Geology

Alluvial sand and gravel are being or have been transported by water and are classified in West Virginia as either (1) glaciofluvial, or (2) fluvial deposits on stream bottoms, floodplains or terraces. In addition to these principal sources of sand and gravel in West Virginia there are (3) residual sand on sandstone and friable sandstones, and (4) talus, rock slides and colluvium containing materials $<$ 6-inches in diameter along steep hillslopes recognized as prospective sources.

Alluvial sand and gravel occur together and are variably proportioned from deposit to deposit. Since deposits range from sand to entirely gravel sizes, it is impossible to give empirical physical and chemical definitions. The diversely mixed materials represent the resistant residues of various rock types encountered in entire drainage basins. They are subjected to a combination of disintegrating forces, chiefly the mechanical action of streams, rain, freezing and thawing, the dissolving action of acid and alkaline waters and, as in the case of the Ohio River deposits, an additional factor, the influence of glaciation.

The topography of West Virginia reflects the differential resistance of massive sandstones to the softer rocks and the geographic arrangement of the rocks by local structures. An additional more subtle topographic influence is: the Pleistocene glaciation which extended south as far as central Ohio and Pennsylvania but left West Virginia entirely unglaciated. The realignment of the ancestral drainage to the present drainage system and the periglacial influences profoundly affected the stage of stream and surface development of West Virginia. These diverse geological activities have contributed to the development of several indistinct cycles of erosion and deposition of alluvial deposits (clay, sand and gravel) during the long erosional period since the final deposition of Paleozoic sediments.

The petrographic and physical characteristics of sand and gravel samples from streams reflect the differences in the stratigraphy, structure and relief -- the geology -- traversed by the various streams. Since the general geology of the State has been known for nearly a century, the petrographic characteristics when taken in conjunction with quality tests pinpoint deficiencies in deposits for use as aggregates in construction and make possible a more realistic evaluation of the alluvial samples. There are decided petrographic differences, if glacio-flucial material from the Ohio River Valley and fluvial materials from the streams of the Valley and Ridge and the streams of the Appalachian Plateau Provinces are compared.

The study of the character and reserves of sand and gravel deposits of West Virginia is timely because of the limited reserves of Ohio River sand and gravel adjacent to a large central belt of sandstones associated with the coal producing section of central and western West Virginia. This belt of sandstones is bounded on the east by belts of limestone with unlimited reserves of rock materials suitable for aggregates and on the west by the Ohio River which has a limited reserve of aggregates of sufficiently coarse gradation confined to its northern reaches. Fortunately, the sand and gravel industry of the Ohio River is served by navigable streams, the Ohio, Monongahela, and the Kanawha Rivers, which traverse the broad sandstone aggregate belt upstream to Fairmont and Charleston in West Virginia. The Ohio River traverses similar areas in eastern Ohio, southwestern Pennsylvania and northern Kentucky.

All streams of West Virginia with the exception of those forming the Potomac Drainage and Potts Creek, a headwater tributary of the James River Drainage in easternmost Monroe County are tributaries of the Ohio River. The deposits of all streams are indigenous to the Appalachian Plateau and Valley and Ridge Provinces of West Virginia with the exception of the Ohio and New Rivers, through flowing streams in north-south and east-west directions. The deposits of sand and gravel of all streams are composed largely of sandstone debris, the rock most resistant to physical and chemical disintegration. The Ohio River's deposits are dominated by debris from extrabasinal sources eminating from the ablating glaciers during the Pleistocene and the New River's deposits were derived in part from debris from the Valley and Ridge and Blue Ridge Provinces of West Virginia, Virginia and North Carolina. Deposits with similar petrologic characteristics and of similar quality are grouped together and discussed under deposits of the Ohio River, deposits of streams of the Valley and Ridge and Blue Ridge Provinces and deposits of streams of the Appalachian Plateau Province (Figure 2).

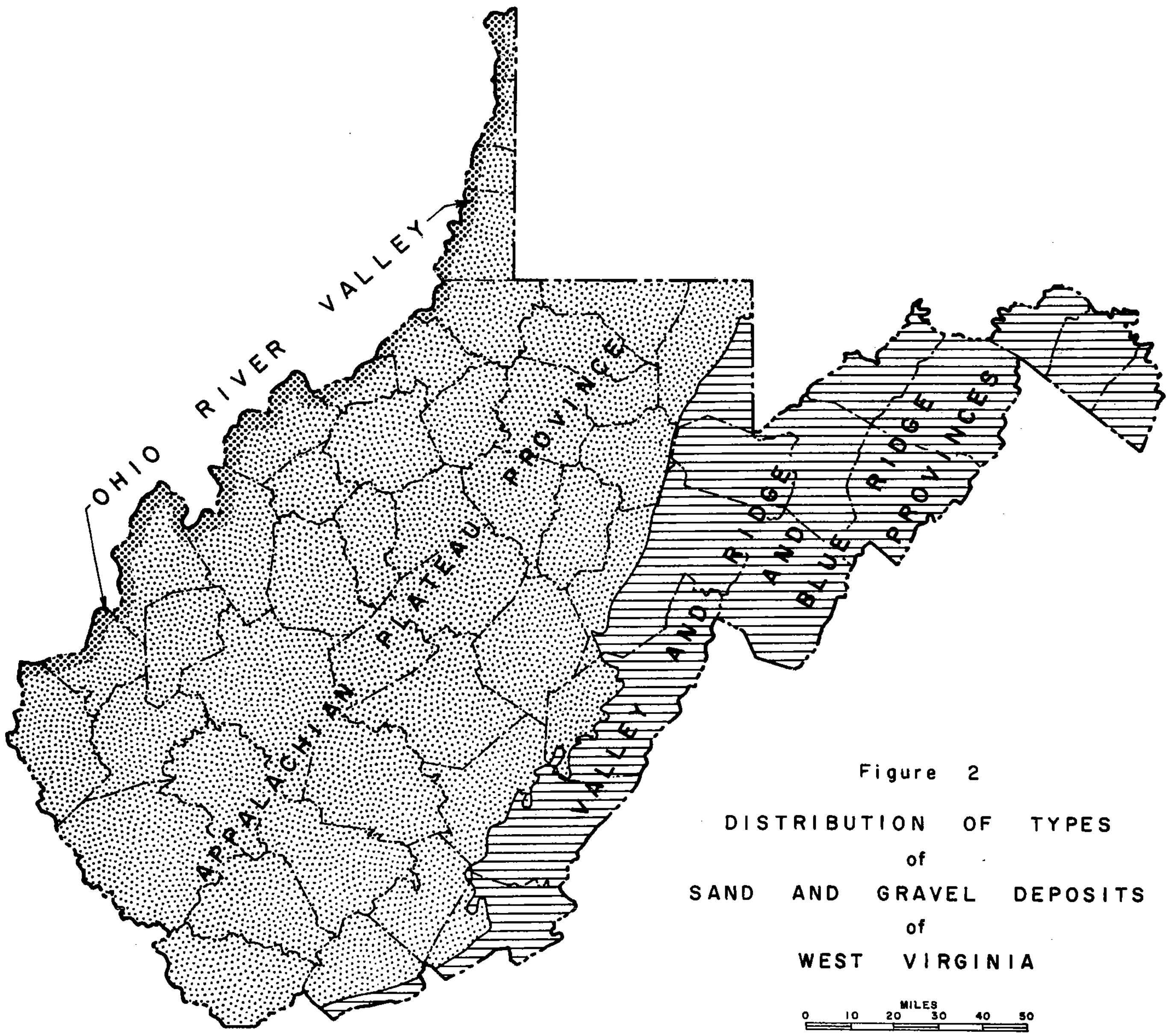


Figure 2

DISTRIBUTION OF TYPES of SAND AND GRAVEL DEPOSITS of WEST VIRGINIA

Ohio River Deposits. -- The development and deposits of the Ohio River Valley were influenced by its proximity to continental glaciation during the Pleistocene. The valley trench, cut earlier, was aggraded with deposits over 100 feet thick by glacial debris from extrabasinal sources entering the headwater streams in the Ohio Watershed. The filling of the Ohio Valley dammed the tributary streams flowing from unglaciated areas into the Ohio River. During the damming, clays, silts, sands and some gravels, derived from intrabasinal sources, filled the Monongahela Valley upstream to Morgantown; the Little Kanawha Valley upstream to Grantsville; the Kanawha Valley upstream to Charleston; and other smaller tributaries.

The erosion of bedrock along the Ohio Valley has not progressed far enough for development of a well-defined valley flat on both sides of the river. As the Ohio River lowered its valley in fill material, it left a series of terraces clustered at various levels on the slip-off slope of meanders. On the opposite side or on the nip of meanders, the river is presently removing narrow terraces and is impinging on steep bedrock hill-slopes. At the present time, the river is flowing on a thin veneer of alluvium above bedrock over most of its course. At a few places, the river is flowing on bedrock forming rapids or falls as at Letart Falls.

A large reserve of fine- to coarse-pebble and fine-cobble gravel is present in terraces, islands, banks and bedload in the bottom of the Ohio River. The cobble gravel sizes are restricted generally to the northern reaches of the river. The gradation of gravel sizes diminishes perceptibly downstream although gravels of boulder size are observed in terraces as far south as Clifton, West Virginia. The boulders are usually of local derivation but igneous and metamorphic varieties are occasionally observed. The sand

and gravel industry is dredging long stretches of the river bottom, banks and islands of the river. Smaller tonnages of sand and gravel are contributed annually by operators mining in small pits, located on the terraces. The sand and gravel on terraces above recorded high water-level, delineated generally by railroads, improved roads and plant sites, have been largely preempted by residential and industrial sprawl. The future prospective reserves of sand and gravel are restricted to the terraces in the zone between high-water level and the pool stage. The pool stage of the Ohio River is being raised with a general plan for replacement and modernization of existing navigational structures by the U. S. Corps of Engineers.

Sedimentary (fine- to medium-grained sandstone and limestone) plutonic (granite and quartz) and metamorphic (gneiss and quartzite) rocks from extrabasinal sources contribute greatly to the usual quality of the immature sand and gravel of the Ohio River. Chert, a mineral of dubious value in construction is derived glaciofluvially along with the other minerals and rocks. In addition, the local rock section contributes smaller although varying percentages of rock material such as sandstone, shale, clay, coal, ironstone and silt and fine- to medium-grained quartz. Generally the quartz is indistinguishable from similar materials of extrabasinal origin. The gravel fraction contains over 60 percent rounded, flat and equidimensional shapes and the sand fraction is principally angular equidimensional shapes.

The quality of sand and gravel dredged from the Ohio River is satisfactory after processing for most construction uses as long as the gravel is sufficiently coarse to meet grading specifications. Sand (fine aggregate), normally a co-product or by-product of gravel production, is in plentiful supply in the dredging operations on the Ohio River. Much of the sand in sand deposits is more finely graded than the sand fraction associated with gravel and will not meet the size specifications for concrete sand.

Deposits of the Streams of the Valley and Ridge Province. - The streams of the Potomac Drainage, with the exception of the headwaters of the North Branch upstream from Keyser, the Greenbrier River and eastern tributaties, drain and derive sand and gravel from the rocks of the Valley and Ridge Province of West Virginia and Virginia. The deposits in the upper section of the New River in West Virginia are derived principally from the Valley and Ridge and Blue Ridge Provinces of West Virginia, Virginia and North Carolina.

The drainage of the streams of the Valley and Ridge Province is arranged in an angular or trellis pattern owing to the systematic folding and the orderly development of the valleys in softer shales and limestones and ridges of resistant sandstones. The main streams generally follow the structures and meander in fairly wide valleys 10 to 20 feet below well developed floodplains. Terraces are developed at various levels above floodplains and serve as sites for the location of residential settlements and small municipalities. At several points the streams leave the valleys and cut through narrow water gaps across anticlinal structures and linear ridges of resistant sandstones so characteristic of the Valley and Ridge Province. These sandstones are the source of the pebble to boulder gravel deposited as bedload on the slip-off slopes of meanders, in islands and at the confluence of major tributaries with master streams. Small deposits of pebble to boulder gravel in clay and scattered gravel are present on terraces.

Practically all the hillslopes in West Virginia with the exception of limestone terrain have some sandstone debris strewn along steep slopes. The slopes of the Valley and Ridge Province are strewn with a great abundance of pebble to cobble gravel derived from the steeply dipping overturned resistant sandstones. The name 'rock streams' aptly describes the manner in which the material in thick accumulations moves along undulating surfaces or coves at the heads of ravines and streams and finally debouches as fairly well-rounded gravel in alluvial fans at confluences with master streams. Some residual and colluvial sand, clay and rock debris also accumulates on the more gently dipping sandstones of the east limbs of the anticlines.

The areal extent of gravel in any deposit is small but the total reserve of bedload gravel in long stretches of streams is large. Little use has been made of the gravel deposits of the Valley and Ridge Province because of the abundant reserve of limestone brought to the surface along the sharp anticlines of the area. A small amount of pebble to cobble gravel has been crushed for road construction along the South Fork and the South Branch of the Potomac River. Some construction sand has been separated from residual and colluvial clay and rock debris by washing and screening. Residual sand and colluvial clay, sand and gravel have been blended and used locally in the construction of soil cement or other type of stabilized base material.

The sand and gravel of the streams of the Valley and Ridge Province are of fluvial origin, are derived entirely from intrabasinal sources and are largely the resistant sandstones of the area. The gravels approach 50 percent flat and equidimensional rounded shapes. The predominance of flat over equidimensional shapes depends on the bedding characteristics of the source rocks. Gravel from the Oriskany, Tuscarora and upper part of the Juniata sandstones are equidimensional shapes and gravel derived from the thin-bedded sandstones of the lower part of the Juniata, Clinton and Upper Devonian are flat shapes. Many gravel deposits in West Virginia have at least a trace of elongated shapes but the gravel deposits of the Valley and Ridge Province have measurable quantities of elongated shapes. Deposits of quartz sand are not common along the streams of the Valley and Ridge Province. Finely graded quartz sand deposits were observed along the New River. The sand fraction, associated with gravel deposits, is usually small and contains a high percentage of shale sand. The deposits on terraces are composed of gravel embedded in a matrix of fine sand, silt and clay.

Soundness and Los Angeles tests show that the gravels of the Valley and Ridge Province are of fair to good quality. They are coarsely graded and would require crushing and screening to meet the grading specifications for coarse aggregates. The gravel deposits on terraces would require crushing and screening to size and extensive washing to remove the coating of clay and silt. After processing, it would be necessary to blend fine aggregates, possibly silica sand, crushed from sandstone, with the gravel fraction to compensate for the poor quality and paucity of natural fine aggregates associated with the gravel.

Deposits of Streams of the Appalachian Plateau Province. - A complex dendritic drainage has developed on the flat-lying rocks, composed of alternating lenticular beds of sandstone and shale associated with coals, clays and thin limestones of Pennsylvanian and Permian age of the Appalachian Plateau Province. The streams meander in entrenched valleys and have developed picturesque gorges across the resistant sandstones of the Pottsville Group of northern West Virginia and the sandstones of the New River Formation, Pottsville Group, of southern West Virginia. The streams are narrow and are fringed with narrow floodplains of fine- to medium-grained sand, silt and clay on the slip-off slopes of meanders and at the confluences of major tributaries. Limited and thin deposits of coarsely graded angular to rounded pebble to cobble gravel, becoming more finely graded downstream and some fine- to coarse-grained sand are present as bedload in the channels and in small islands. In the downstream portions of the streams, the gradients decrease to around 2 feet per mile, the channels widen and streams meander broadly. Bedloads of sand and gravel give way to deposits of fine-to medium-grained sand. The sand content in floodplains increase in the lower reaches of the southern streams such as the Kanawha, Elk, Coal, Guyandotte and Big Sandy-Tug Fork Rivers. The floodplains of the lower reaches of the Little Kanawha River and the Monongahela River in West Virginia are composed of fine sand, silt and clay. Gravel deposits of limited areal extent were observed on terraces a few feet above the floodplains on the eastern tributaries of the Monongahela Drainage of the Allegheny Mountain Section (subdued Valley and Ridge type structures) of the Appalachian Plateau Province. These streams, the Cheat and Tygart Rivers, have a bedload of sand and gravel and low-lying terraces with gravel of limited areal extent embedded in a matrix of fine sand, silt and clay. High-level terraces (> 100 feet above pool stage), cut during earlier drainage changes of the ancestral Monongahela, Little Kanawha and Kanawha Rivers, are the sites of alluvial clay, silt, sand and some gravel deposition of intrabasinal origin. These deposits resulted from the aggradation and subsequent damming of the main lines of drainage during continental glaciation in Central Ohio and Pennsylvania during the Pleistocene.

The sand and gravel deposits of the streams of the Appalachian Plateau Province are derived entirely from intrabasinal sources and reflect the more resistant rocks, principally varieties of sandstone, traversed by the streams. An ample source of materials from numerous nearly flat-lying sandstones containing shales, thin coals, clays and minor quantities of limestone provides thin discontinuous bedload deposits laid temporarily on the slip-off slopes of meanders, in islands, and at the confluence of major tributaries with the master stream and are awaiting transport farther downstream. At present, sand and gravel deposits are used in the maintenance of roads, secondary roads and streets and for fill material. Selected materials are used for the construction of road base along the Cheat River, Numerous small sand plants have sporadically pumped sand from the bottom of the Elk, Coal, Guyandotte and Big Sandy-Tug Fork Rivers. Some plants recover coal sand and gravel for säle. Co-product or by-product fine- to medium-grained sand is sold for traction sand and for local mortar and concrete.

The resistance of the exposed rock along with local stream characteristics influence the quality of the gravel which ranges from entirely poor to less than 50 percent in the good condition. The gravel deposits are coarse and poorly graded in the upstream portions of higher gradient and the shapes of the gravels are influenced somewhat by the distance of transport and the bedding characteristics and weathered condition of the source sandstone. The sand fractions, associated with gravel, are a small percentage of the coarser gravel deposits and are coarser grained than the sand in sand deposits. The coarseness of the sand fraction of gravel deposits is attributed to aggregates of sand in a fair to poor condition in the northern streams and coal, carbonaceous shale and aggregates of sand in the poor condition in the southern streams. As sand deposits become finer grained, the quartz content increases and the sand improves in quality. In the lower reaches of southern streams large quantities of sand derived from masses of fine-to medium-grained sandstones, cemented with siderite and other carbonates, are deposited as bedload and underlie the floodplains. With the exception of coal, the sand deposits lack the +8, +16 and +30 mesh fractions required for the gradation of concrete sand. Coal gravel and coal sand is a ubiquitous constituent in the sand and gravel deposits of the southern streams, viz., Elk, Coal, Guyandotte, and Big Sandy-Tug Fork rivers.

Although the reserves of sand and gravel at any deposit are small, there is a large total reserve of sand and gravel along the streams of the Appalachian Plateau Province. Some deposits are suitable for use as fill material and in the construction of road base. Sand deposits may find use in mortar.

Construction Sand from Sandstone. - High-silica sandstones of Lower Pennsylvanian to Silurian ages are prospective sources of silica sand (fine-aggregate) for use in concrete and mortar in West Virginia provided the sandstones are friable enough for separation to discrete grains.

Minable thicknesses of the Pottsville of Pennsylvanian and the Berea of Mississippian ages are medium- to coarse-grained with lenses of fine pebble gravel in Preston, Tucker, Randolph, and Pocahontas counties. Selected units of these sandstones are suitable for concrete sand after crushing and processing. The Pottsville sandstones become medium-grained and lose pebble gravel as they pass below drainage in Monongalia, Taylor, Barbour, Upshur, Nicholas, Fayette, Greenbrier and Raleigh counties. Friable units of sandstones in these counties are suitable for use as mortar sand after crushing and processing.

The Stony Gap and Droop sandstones of Mississippian age of southern West Virginia and the older Devonian and Silurian sandstones, the Oriskany, Healing Springs, Keefer and Tuscarora, of the Valley and Ridge Province are fine- to medium-grained tightly cemented sandstones with thin zones of fine pebble and gravel. Local deposits of residual sand on these sandstones and friable zones are suitable for use as mortar sand.

The Pennsylvanian and Mississippian quartzose sandstones are considered mature mineralogically but their angularity and fair sorting classify them as immature fluvial deposits. The high-silica sandstones are composed mostly of angular equidimensional grains of quartz and quartzite which are cemented to various degrees of induration with quartz. Occasional fragments of other rock types are present, e.g., a variety of the more resistant sedimentary rocks such as sandstone and chert, a minor quantity of clay and micaceous minerals, and trace amounts of a ubiquitous suite of fine grained heavy minerals, viz., zircon, leucoxene and tourmaline. The coarser fractions of sand are more highly rounded than the finer fractions. Occasional dust rings, outlining grains in thin section, indicate that a larger percentage of grains were rounded originally than are evident. Etching of coarser grains and secondary crystal growths in optical continuity with original grains are common.

Locally the sandstones vary in induration from friable to orthoquartzitic. The efficiency of breaking the weakly bonded aggregate is an important technical problem in the preparation of sandstone for use as mortar and concrete sand. The difficulty of crushing high-silica sandstone into discrete grains condemns deposits as prospective sand sources. The fineness moduli of many of the samples are slightly lower than the fineness moduli recorded in the physical tests because the size distribution of the latter more nearly represents the discrete sand sizes before induration. The fineness moduli of the sandstone samples, reduced to discrete grains, range from 1.42 to 2.05 and the conglomeratic sandstones from 2.61 to 4.36. The percentage loss of fine aggregates, recorded in the soundness test, increases in proportion to the quantity and friability of the aggregates of sandstone in the sample.

Conclusion

Petrographic examination and quality tests of sand and gravel samples verify that sand and gravel from the bedload and terraces of the Ohio River are suitable for a variety of construction uses after preparation. Similar studies of the gravel of streams of the Valley and Ridge and the Allegheny Mountain Section (Cheat and Tygart Rivers) of the Appalachian Plateau Provinces indicate that the gravels could be processed and blended with coarser processed sand from sandstones for many Portland Cement concrete and asphaltic concrete uses. The sand and gravel deposits of the remainder of the Appalachian Plateau Province are available in many instances for use in construction of shoulder and base for roads and in secondary road and street maintenance. The coarser bedload and high-level terrace sand deposits appear suitable for use in mortar and as engine sand and fine-grained sand for use as filler.

At the present time, however, specifications for aggregates are becoming increasingly stringent. The need for continuing study of the methods of processing, agglomerating and utilizing the little used marginal deposits of sand and gravel along the streams of West Virginia should continue as a means of supplementing the Ohio River deposits which are being depleted or preempted by residential and industrial sprawl.

Acknowledgments

I would like to acknowledge Dr. Paul H. Price, Director of the West Virginia Geological and Economic Survey and State Geologist, for assigning this fascinating study to me and for his patience and encouragement during the study over a period of three years. I would like to call attention to and say thanks to two faithful workers, Mr. Charles E. Hozdic, formerly employed by the eological Survey and to Mr. Samuel M. Brock, Jr., the co-author, formerly ith the State Road Commission and presently employed by the Geological ırvey. Mr. Richard G. Hunter, Spectroscopist, analyzed the clean-washed ındstone samples chemically and prepared slides for this paper and Mr. Paul Queen, Cartographer, drafted the figures for the report[1] and for this paper. ı important contribution in the study was the quality testing of all samples the Staff of the Morgantown Testing Laboratory, West Virginia State Road mmission, Mr. John Judy, Engineer-in-Charge. Appreciation is extended to e Dravo Corporation and the Ohio River Sand and Gravel Company, large sand d gravel producers in West Virginia, numerous other sand and gravel pro- cers, and curious landowners. All accepted inconvenience graciously and tended every courtesy during the sampling.

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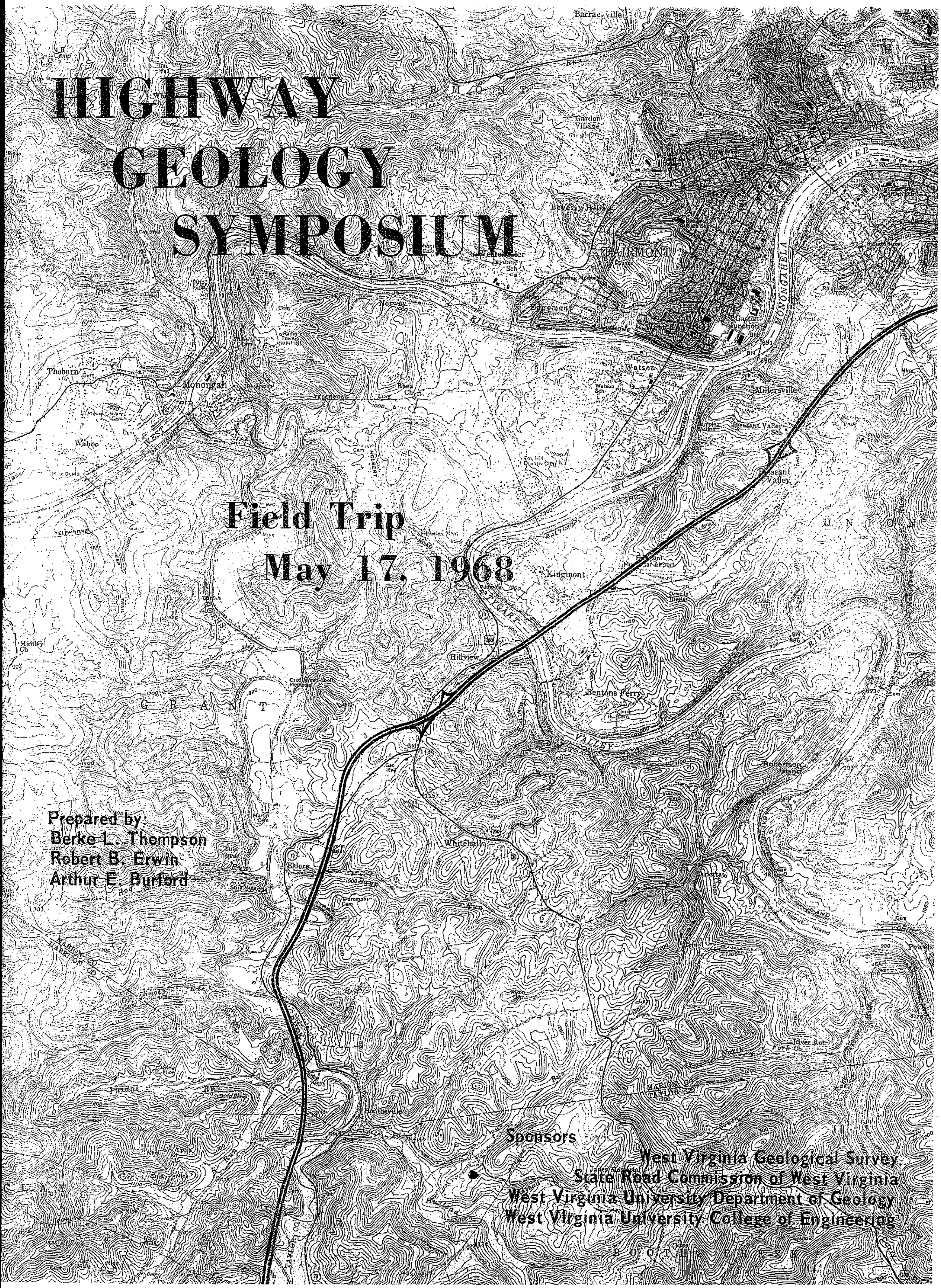
[1]Sand and Gravel Resources of West Virginia: W. Va. G.S., Manu- :ipt, 1968.



HIGHWAY GEOLOGY SYMPOSIUM

Field Trip
May 17, 1968

Prepared by:
Berke L. Thompson
Robert B. Erwin
Arthur E. Burford

Sponsors

West Virginia Geological Survey
State Road Commission of West Virginia
West Virginia University Department of Geology
West Virginia University College of Engineering

FOREWORD

On behalf of the National Steering Committee the sponsors of the 19th Annual Highway Geology Symposium welcome you to West Virginia. We hope this field trip will prove to be stimulating and interesting. Our problems in highway construction are not unique but in turn provide a spectrum of situations inherent with our varied geology and terrane. The trip deals with construction problems in areas of incompetent bed rock, unconsolidated deep glacial lake fill, and mined and unmined coal seams lying above, below, and at or near road grade. The State Road Commission was fortunately, for our purposes, able to pinpoint all of these adverse conditions along a short stretch of Interstate 79 now under construction south and southeast of Fairmont, West Virginia.

The field trip was planned principally by the State Road Commission represented by Berke L. Thompson, Assistant Director, Materials Control, Soil and Testing Division. The geologic setting was provided by Alan C. Donaldson, Associate Professor, Department of Geology. A committee under the chairmanship of Robert B. Erwin, Assistant State Geologist, developed the meeting program.

Please enjoy and profit from your stay with us.

Paul H. Price, Chairman
1968 Highway Geology Symposium
Morgantown, West Virginia
May 17, 1968

INTRODUCTION

The Morgantown-Fairmont area is characterized mainly by Pennsylvanian-aged terrestrial sandstones, siltstones, shales, and coals and very minor marine shales and limestones of the Allegheny, Conemaugh, and Monongahela Groups. (Figure 1). The shales are especially weak and susceptible to failure. The sandstones and siltstones are lenticular and discontinuous and are inconsistent in thickness. Mine voids especially below road grade but also above or near grade present special problems. Thick Pleistocene lake deposits, mainly clays, silts, and sands, present special fill and cut slope problems.

The trip begins with a detailed resumé of local stratigraphy (STOP 1) and continues with a look at an active landslide (STOP 2), both in Morgantown. Then, along a short stretch of Interstate 79 near Fairmont (approximately 16 miles to the southeast) stability of thick Pleistocene lake deposits (STOP 3 and 5) is considered along with the treatment of cut slopes where coal is encountered above grade (STOP 4) and design methods when mine voids are encountered below grade (STOP 6) and at or near grade (STOP 7).

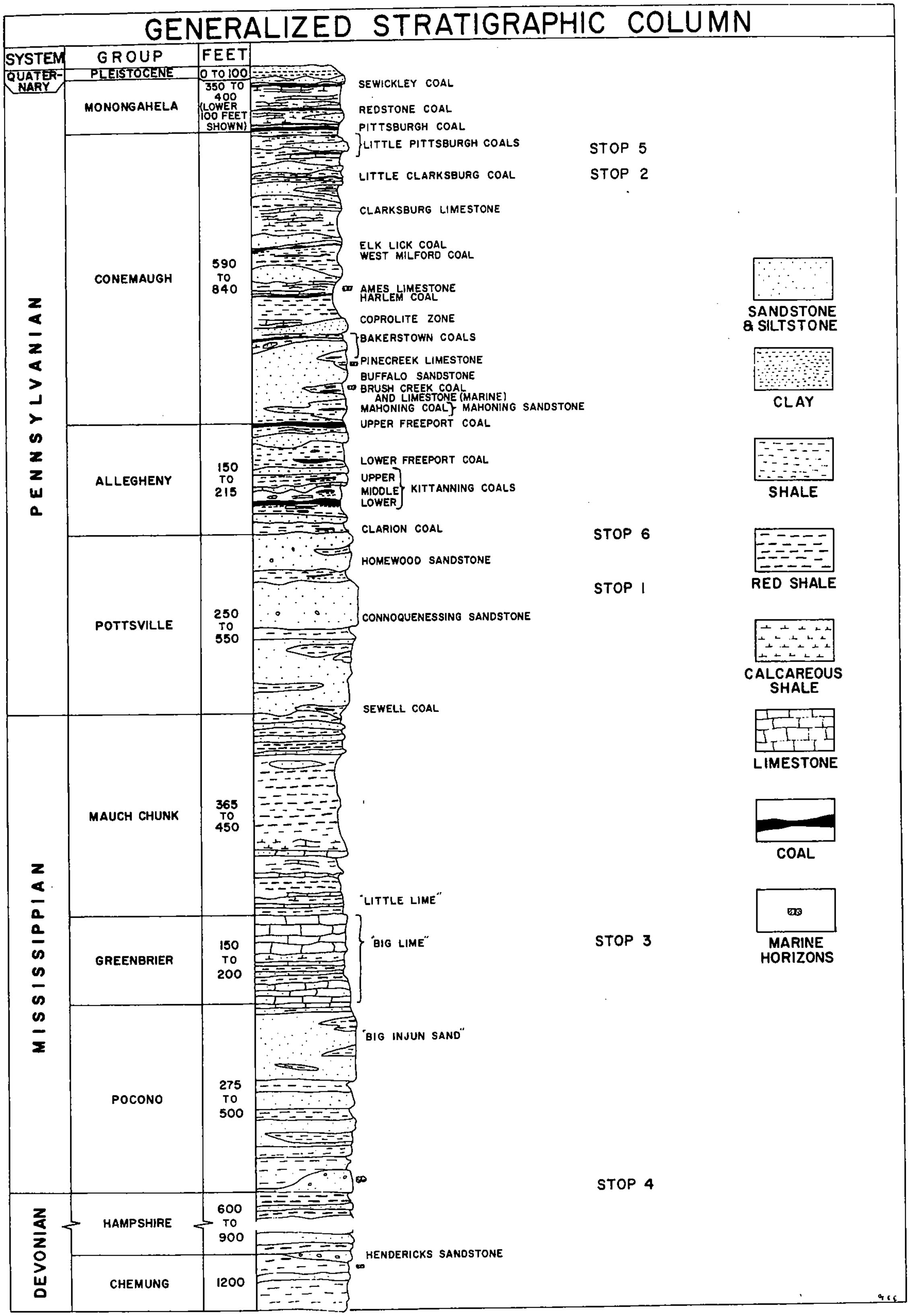


FIGURE 1

Generalized Section of Rocks in Appalachian Plateau.

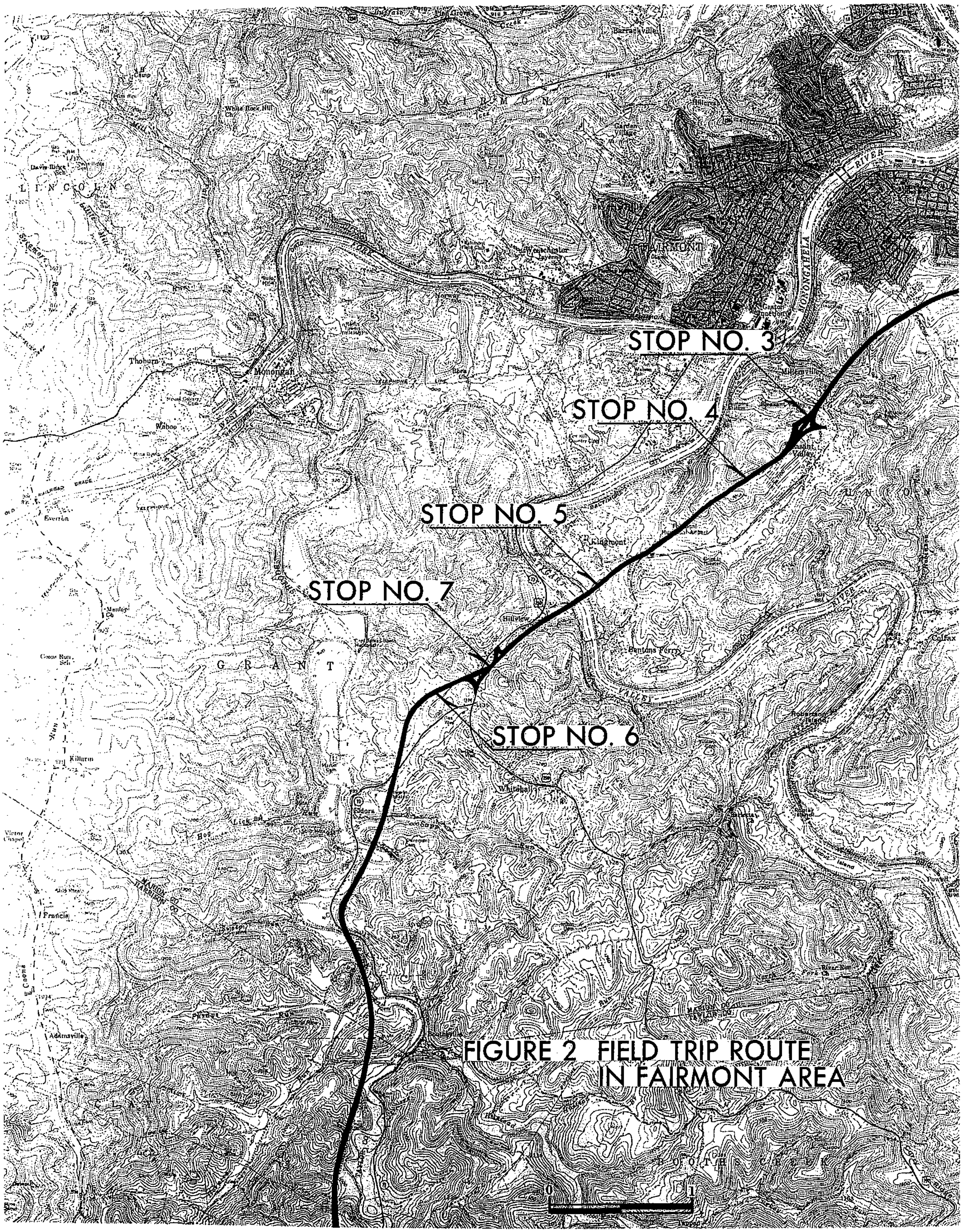


FIGURE 2 FIELD TRIP ROUTE IN FAIRMONT AREA

ITINERARY

Mileage	Distance	
0	0	Depart TWIN TOWERS, EVANSDALE CAMPUS.
.9	.9	STOP #1: BOULEVARD SECTION, MORGANTOWN, CONEMAUGH EXPOSURE.
		Follow State Route 73 and Green Bag Road.
7.0	6.1	STOP #2: TYPICAL SLOPE FAILURE IN A GLACIAL LAKE DEPOSIT IN WEST VIRGINIA (Field Trip Party Will Not Debus)
		Follow Green Bag Road, State Route 73 and local Service Route 31 to Fairmont, then to Interstate 79.
26.3	19.3	STOP #3: FOUNDATION TREATMENT AND CONTROLS UTILIZED FOR A FILL OVER DEEP ANCIENT GLACIAL LAKE DEPOSITS.
		Follow I-79.
27.2	0.9	STOP #4: A TYPICAL TREATMENT OF CUT SLOPES WHERE COAL IS ABOVE GRADE.
		Follow I-79.
28.1	0.9	STOP #5: A CUT SLOPE DESIGN UTILIZED IN SOIL DEPOSITED BY GLACIAL LAKES.
		Follow I-79.
29.9	1.8	STOP #6: A DESIGN METHOD FOR ROADWAYS WHERE COAL HAS BEEN MINED BELOW GRADE.
35.1	5.2	Follow I-79 to end.
		Follow State Route 73.
41.1	6.0	Site of old mine fire and settlement of Route 73.
		Continue on State Route 73 to Ramp of I-79.
41.6	0.5	STOP #7: A DESIGN METHOD FOR ROADWAYS WHERE COAL HAS BEEN STRIP-MINED AND DEEP MINED AT GRADE.
61.6	20.0	Return to TWIN TOWERS, MORGANTOWN, via I-79, Local Service Route 31 and State Route 73.

STOP 1: BOULEVARD SECTION, MORGANTOWN, CONEMAUGH EXPOSURE

An excellent exposure of approximately 300 feet of sedimentary rock representing the middle part of the Conemaugh Group (Pennsylvanian age) can be observed here. The Section occurs along the steep valley slope bordering the Monongahela River between the railroad track and the Engineering Science building of West Virginia University. A generalized profile and description of the rocks in the Boulevard Section are presented in Figure 3. The rocks strike approximately N 30 E and dip 3° NW.

The beds exposed in this section were deposited 325 million years ago as deltaic and shallow marine sediments. The sediments were transported seaward from an eastern land source. The sediments were transformed into rocks when they were buried by additional sediments and subsequently uplifted and slightly flexed during mountain building time about 300 to 200 million years ago. Streams flowing across this newly developed land removed tons of rock and carved out our present hilly landscape. In relatively recent times, less than 1 million years ago, glaciers blocked the northward flow of the Monongahela River near Beaver Falls, Pennsylvania, causing a natural lake to develop. Varved clays and sands accumulated in this lake. Removal of the ice dam and diversion of the Monongahela River into the present Ohio River resulted in deepening of the river valley. A river terrace

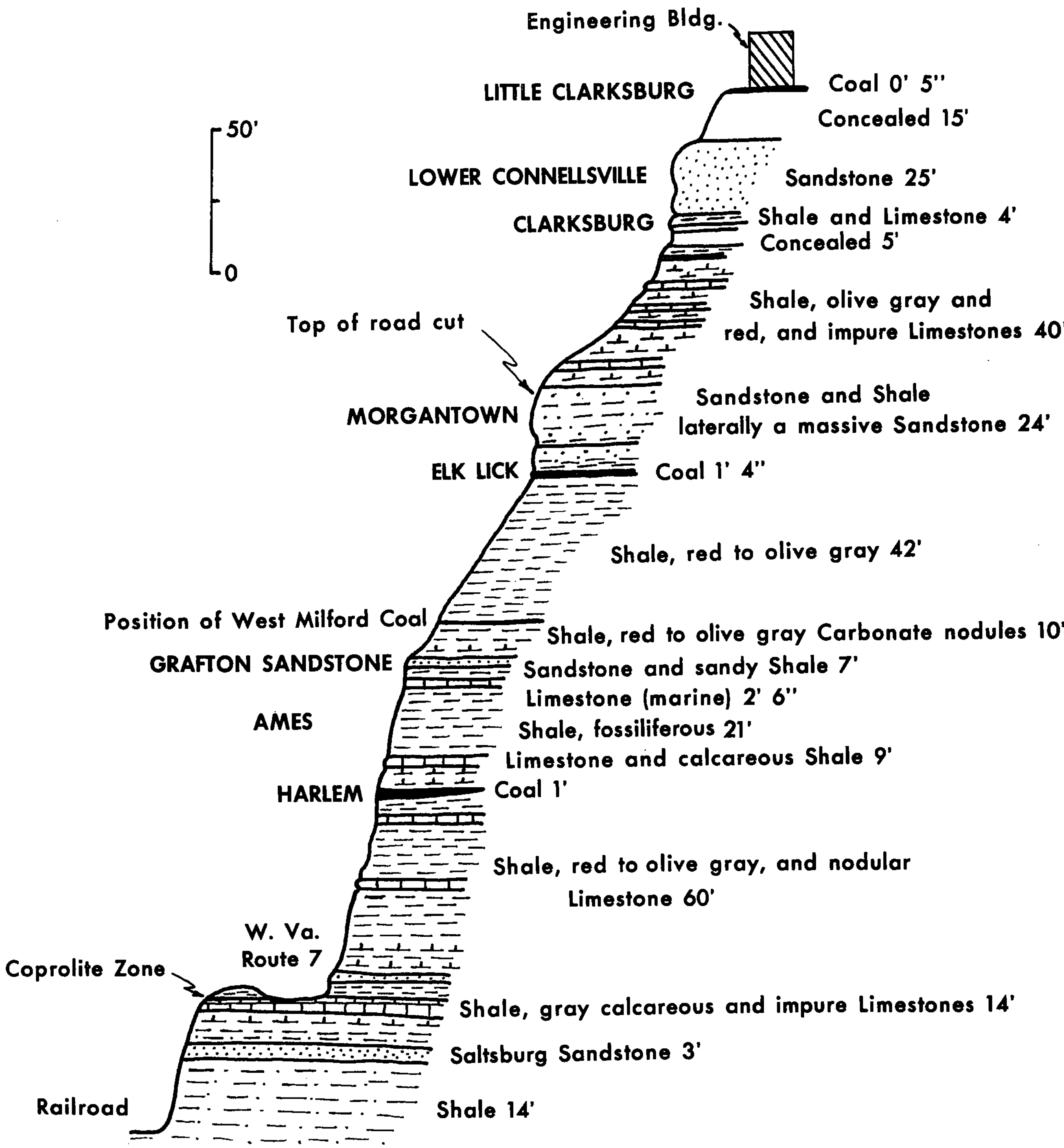


FIGURE 3 BOULEVARD SECTION, MORGANTOWN

(after Arkle 1963)

and flood plain are relatively modern additions to the area by the river.

Differential weathering results in overhangs of sandstones which eventually fail along joint planes. The shales and underclays are the most plastic rocks in the sequence. Ground-water movement is predominantly joint controlled because of the low permeability of the argillaceous sandstones, limestones, and shales. The clays deposited in Lake Monongahela and the more recent river terrace are the least stable earth material in the area. At the southern end of the road cut near the Esso Station (perhaps within view of some observers here) evidence of multiple slides of colluvium, principally composed of red shale, is apparently related to the removal of the terrace deposits and colluvium at the toe of the hill at road level.

STOP 2: TYPICAL SLOPE FAILURE IN A GLACIAL LAKE DEPOSIT IN WEST VIRGINIA. (Field Trip Party will not debus)

Green Bag Road was originally constructed by a local firm as a short access to one of their plants. The road became a part of the State Road System in 1962.

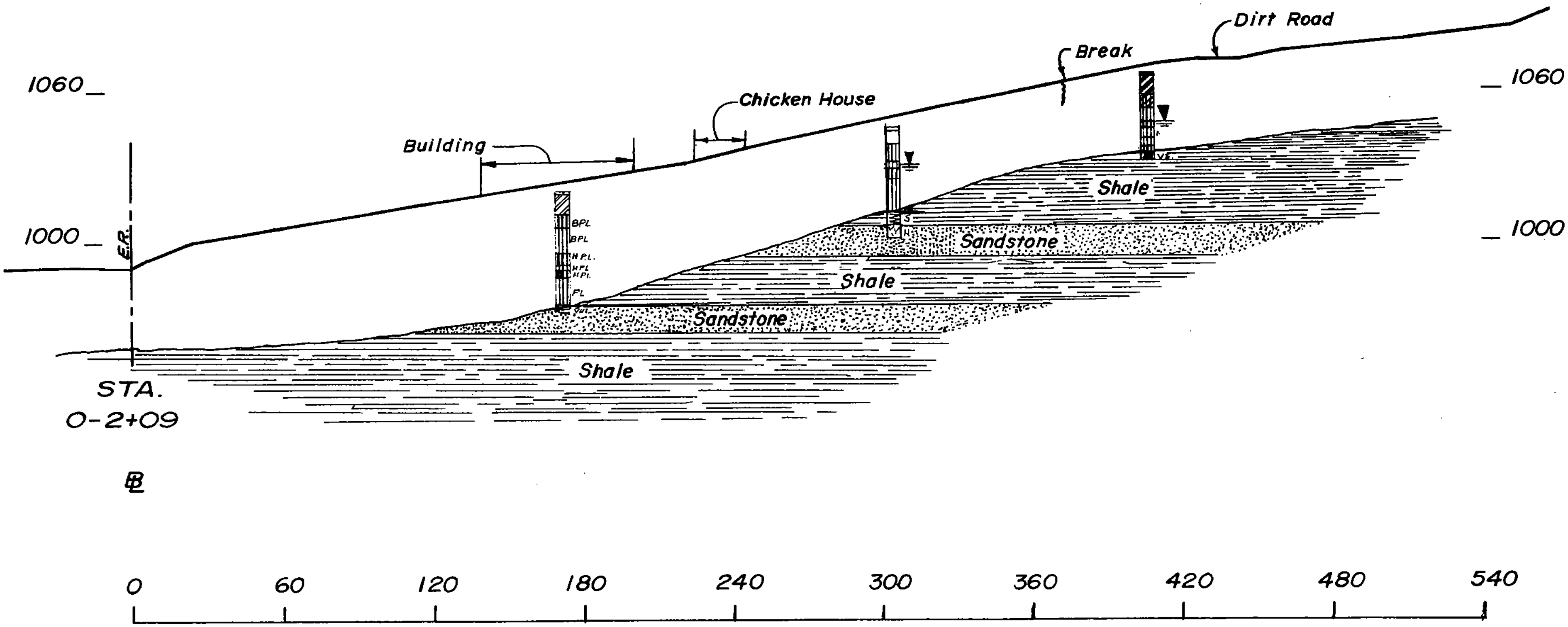
The shallow cut near the intersections of Local Service Routes 64 and 81 exhibited instability in 1963 and was corrected by placing a rock buttress in the cut slope. During the investigation for this correction the boring program revealed the existence of a deep soil overburden believed to be one of the terrace deposits of ancient Lake Monongahela which existed during the Pleistocene Epoch.

The original correction apparently stabilized the shallow failure, however, additional movement of a much larger magnitude was observed in 1967 adjacent to this area. Tension cracks appeared some 370 feet from the edge of the roadway and the land owner reported distress in his farm buildings.

In order to fully study the slide, two observation wells which would accomodate a "slope indicator", an instrument for measuring horizontal ground movements to depths up to several hundred feet, were installed to locate the slip plane. Additional borings were performed along with a seismic and electrical resistivity survey. One of the wells was located some 300 feet from the edge of the roadway in the upper mid-portions of the slide. Movement was recorded at all depths indicating that the slip plane is apparently near the rock line.

The study found that a much larger soil mass was moving than that observed in 1963. In fact, it was apparent that the soil mass which extended some 700 feet from the edge of the roadway up the hillside was also potentially susceptible to movement.

The accompanying cross section (Figure 4) at station 2+09 shows that the natural ground lays on an approximate



SLIDE ON GREEN BAG ROAD NEAR MORGANTOWN W.VA.

FIGURE 4

five horizontal to one vertical. The soil first exhibited its inability to resist the local stresses that were produced within the cut slope when the roadway was originally constructed. These stresses were acute but merely indicated that the constructed slope was too steep for the material. The more recent failure is attributed to lateral support and to ground water conditions.

Several methods of correction were studied such as, drainage, unloading the head of the slope, construction of a counter balance, chemical stabilization, etc. The estimated cost of correction is of such magnitude that it is difficult to recommend any one or a combination of several as a means of re-establishing stability due to the type of public facility involved.

STOP 3: FOUNDATION TREATMENT AND CONTROLS UTILIZED FOR A FILL OVER DEEP ANCIENT GLACIAL LAKE DEPOSITS.

The decision to locate I-79 as shown in Figures 2 and 5, over this alignment was the result of extensive soil and geologic studies and a comprehensive boring program both in this area and in the adjacent valley to the east known as Pleasant Valley.

The geologic occurrence of ancient lake deposits was discovered by the boring program for an alignment which had been proposed on the eastern hillside of Pleasant Valley. In

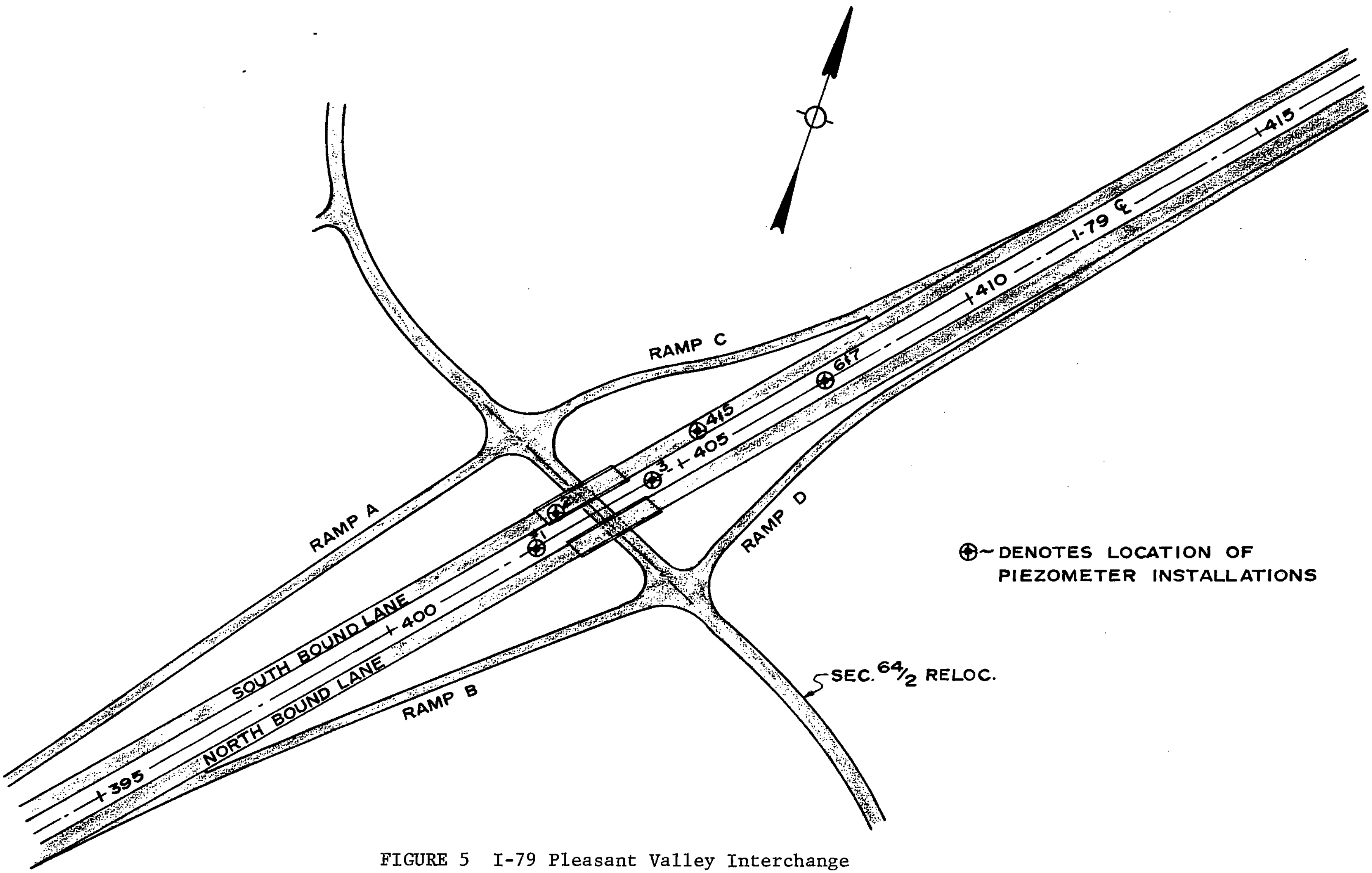


FIGURE 5 I-79 Pleasant Valley Interchange

PLAN

SCALE 1" = 200'

I-79 MARION CO.

order to determine the rock soil contact and stability of two large fills the proposed foundations were saturated with auger borings. Two cross sections (Figure 6) and a plan view containing contours on both the ground surface and top of rock (Figure 7) were prepared which indicated the probable deposition of the lake deposits in these locations. It was soon apparent that to cross the deposits in any other alignment than the one adopted would require extreme remedial measures to satisfy stable cut slopes and fills. The boring and geologic program produced evidence that thick deposits consisting of wet clays, sands, and residual soil blanketed the hillside and valley floor (Figure 8). These sediments were found between elevations 945 and 1125 feet. The sediments were attributed to ancient Lake Monongahela.

All alignments studied that were apparently possible had to cross the lake deposits. The one chosen crosses these sediments in the most advantageous manner as it crosses normal to the valley rather than at an angle which would create sidehill fills and cuts.

The design of the fill followed normal design for fills constructed on compressible materials in West Virginia in that it required that the bottom five feet of the embankment be constructed of permeable rock fill.

The following requirements which were recommended by the soils consultant retained by the prime design consultant were

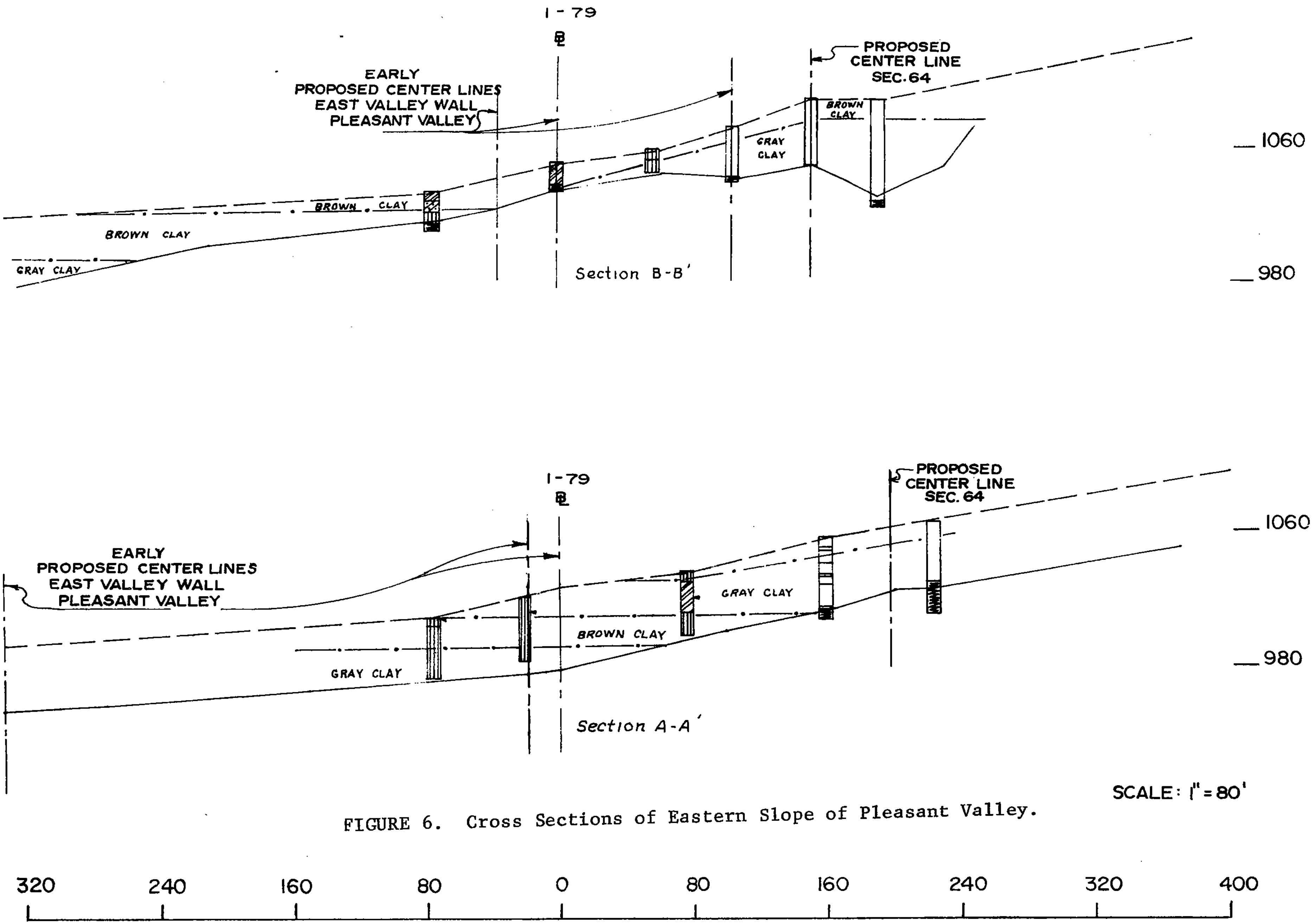


FIGURE 6. Cross Sections of Eastern Slope of Pleasant Valley.

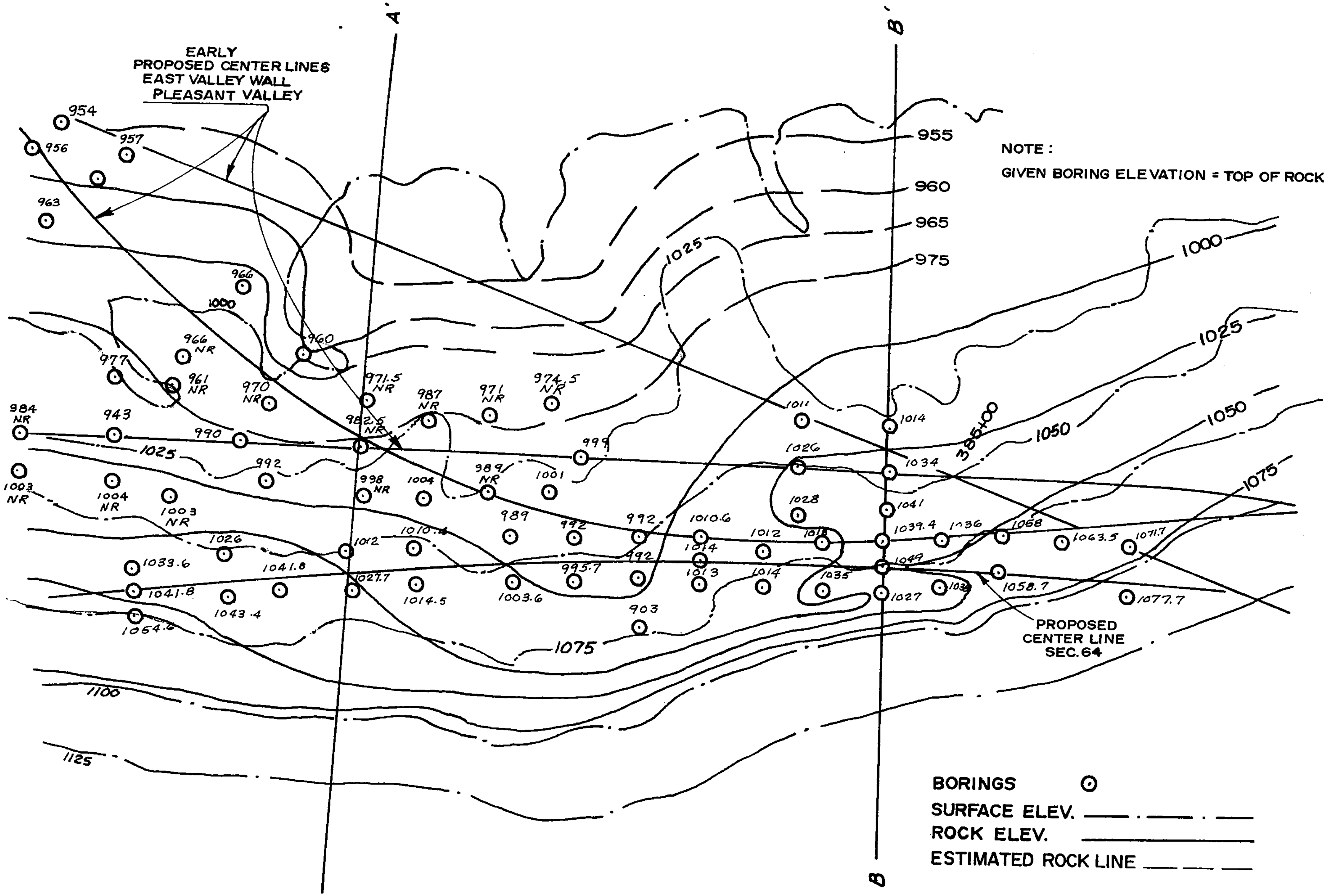
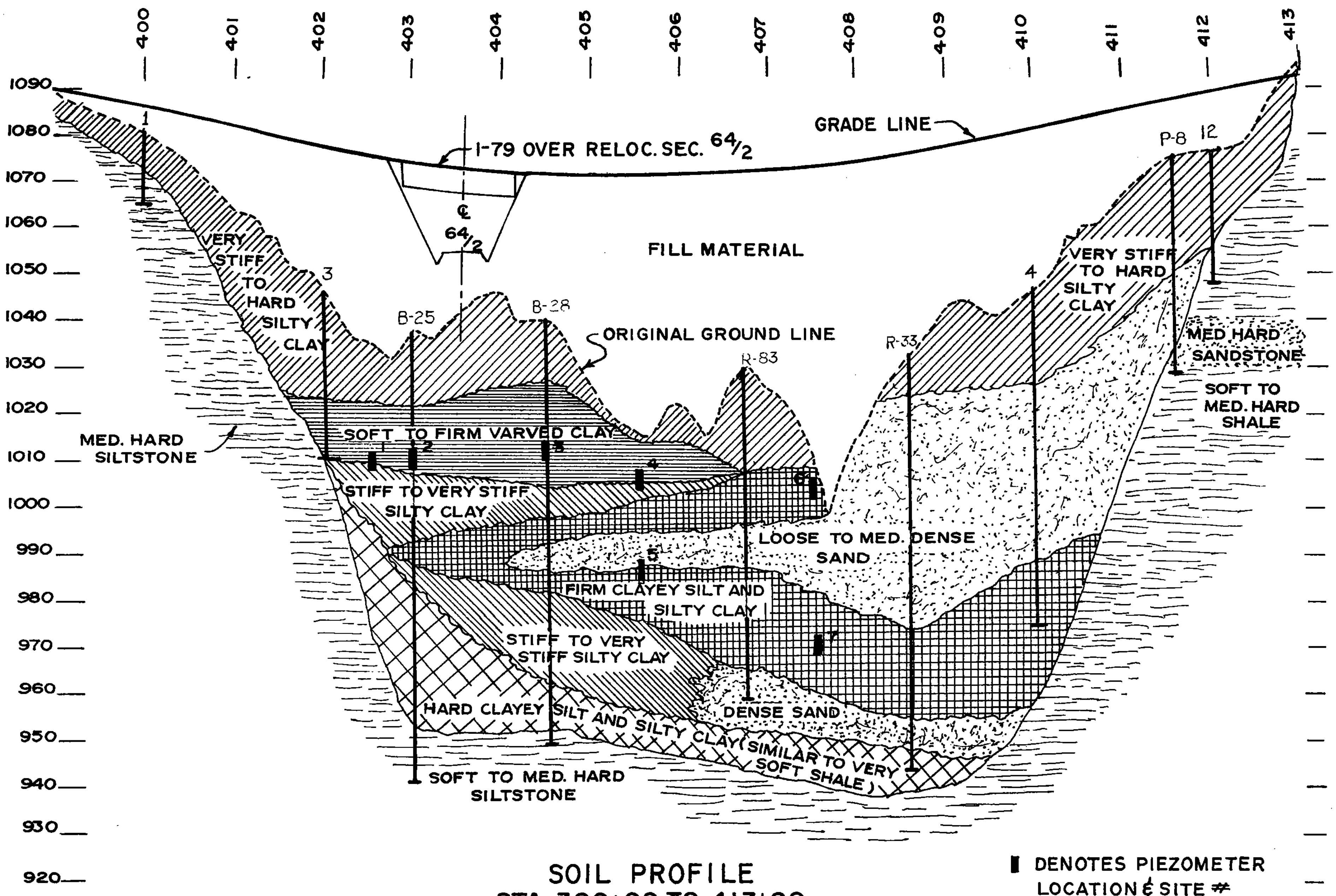


FIGURE 7. Sketch Map of Borings for I-79 in Pleasant Valley.

adopted. A rate of loading was specified for both the mainline and route 64/2 in this area. The construction should be halted when the embankment reached elevation 1055 feet. When reaching this elevation, a four months waiting period was specified after which construction of the embankment was limited to twelve (12) feet of fill per month. In order to determine if excessive pore pressure due to loading by the embankment was developing during construction on the underlying lake deposits seven (7) piezometer devices (Figure 9) were installed at various elevations in the lake deposits (See Figures 5 and 8). A pore pressure of two (2) tons per square foot was designated as the critical equivalent stress and all operations were to cease should this pressure be exceeded. Ten (10) settlement plate sites were designated in order to determine if the predicted settlement was actually taking place. Also slope stakes were recommended to the left of centerline to provide early warnings of loss of slope stability.

Piezometer readings never reached the critical value designated although one site, Number 7, increased up to 400 days before a pressure peak of .85 tons per square foot was reached. No distress had been noted in the slope stakes indicating that the design was apparently successful in preventing a base failure. Because the sand and clay layers apparently lens and pinch dut in a horizontal direction,



SOIL PROFILE
STA. 399+00 TO 413+00
I-79 MARION CO.

FIGURE 8

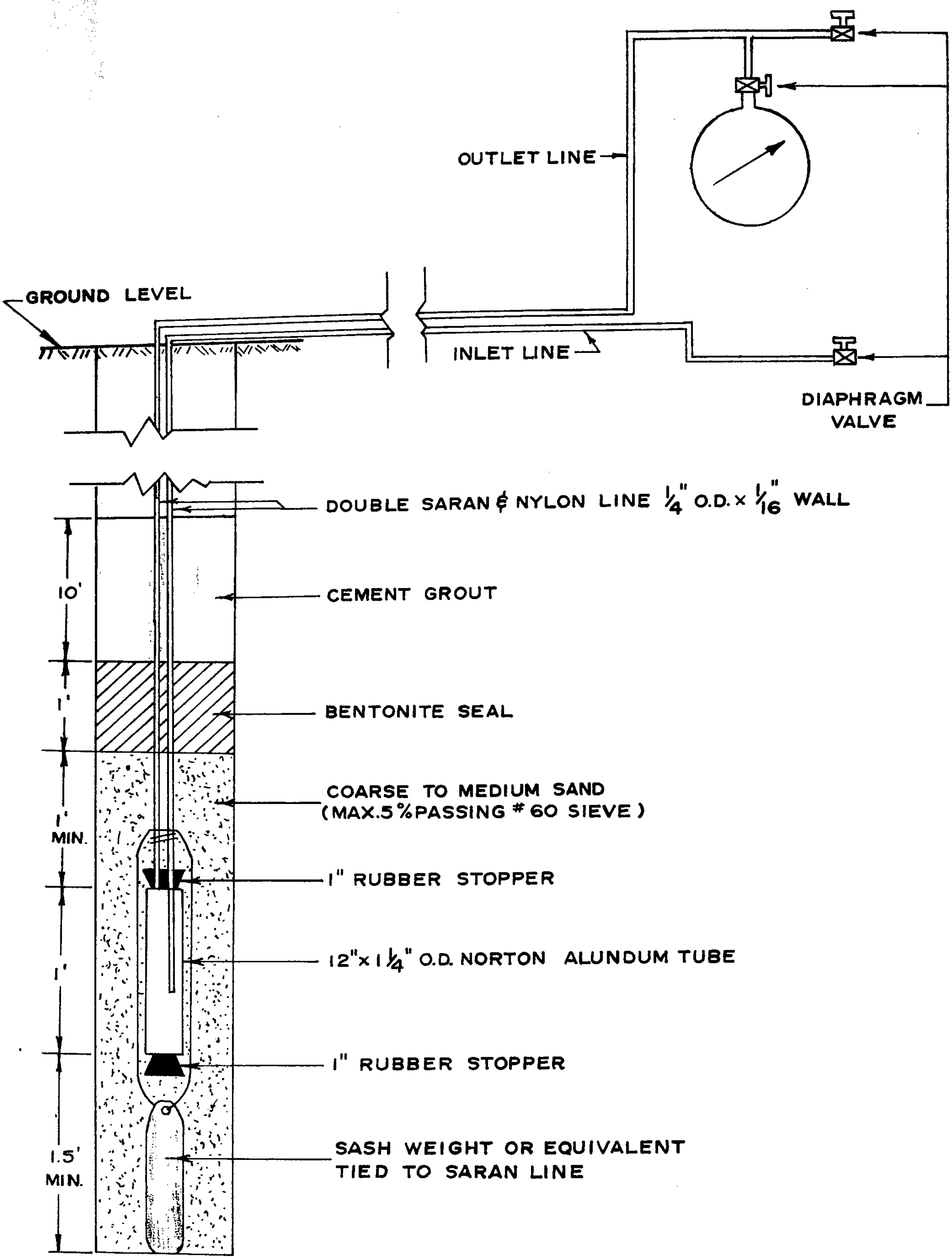


FIGURE 9

DETAILS OF PIEZOMETER SYSTEM

it was difficult to predict the time required for settlement; however, there has been 9.6 inches of settlement indicated by the settlement plate at station 405+50 which represents 32 percent of that predicted. There has been an overall average of 26 percent of settlement of that predicted by consolidation tests to date.

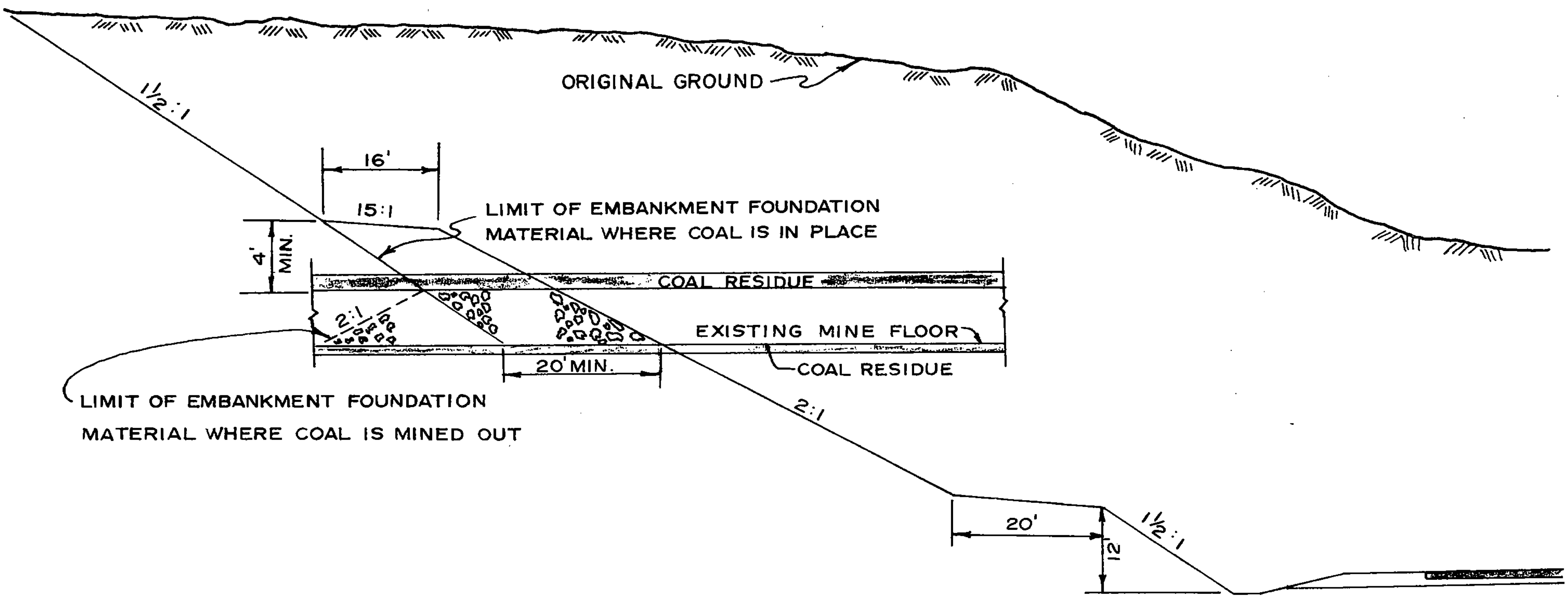
Two interesting points should be noted concerning the geology of this location; one is the depth of the sediments in the valley, they were found to be some twenty-seven (27) feet lower than any other lake sediments in the area and the upper limits appears to be some ten (10) feet higher than found in earlier studies of the lake deposits, and second, the valley which is filled was apparently the outlet of a stream or river running through Pleasant Valley. The major drainage for the valley presently runs in the opposite direction.

STOP 4: A TYPICAL TREATMENT OF CUT SLOPES WHERE COAL IS ABOVE GRADE.

During the design of the slopes for this project the problem arose as to a method of closing the openings left by deep mining of the Pittsburgh coal which is here above grade. It was imperative that an economical method be found to close and drain these openings because of the wide-spread nature of the problem here and on other projects.

Several methods were proposed such as sealing with block or concrete walls, filling the opening with rock and shooting the openings closed. It was difficult in the design phase to determine the overall extent of the openings, the amount of collapse, the orientation of room and entries to the slope, and the accuracy of the available records of some of the mines to be encountered. None of the above proposed methods were adopted.

The method adopted as indicated by Figure 10 proved to be the most economical, required little maintenance and produced a very pleasing and aesthetic design. The design incorporated a bench at the bottom of the coal normally twenty (20) feet wide and required back filling to a minimum of four feet above the top of the opening with random material. Normal compaction specifications were required for the placing of the material on the bench. The material was placed similar to the other benches with an outer slope of two (2) horizontal to one vertical and the top was sloped fifteen (15) horizontal to one vertical toward the roadway so that run off would not enter the opening. Soil slopes were used to attain adequate compaction, to deter settlement, and to insure strength. Underdrain was also incorporated in the openings in those areas where the dip of the coal was toward the roadway. The extra footage above the opening was provided in order to combat settlement and the reappearance of an opening.



TYPICAL SECTION
TREATMENT OF MINE VOID AREA
STA. 2166+50 TO STA. 2172+00
FIGURE 10

There are several areas where this method has been utilized on this project including the one on the right slope, all have performed satisfactorily. On some of the later designs Select Rock Fill has been designated. This material which is normally the best rock from the excavation but does not require acceptance testing, provides a drainable material and thus alleviates the danger of problems that might arise should the drainage conditions behind the slope change at some future date.

STOP 5: A CUT SLOPE DESIGN UTILIZED IN SOIL DEPOSITED BY GLACIAL LAKES

One of the early problems encountered both in design and construction of this project was the slope design through the ancient Lake Monongahela deposits. The original construction through this area was for two lanes of future four lane roadway and it was soon apparent that the silty deposits became quick due to frost, ground water and pore pressure. Fortunately, the original design of the roadway never reached completion because of the addition of this section into the Interstate System. The subsequent additional time allowed for a more detailed study of the problem and the design which can be seen. The deposits were laid down on an irregular erosional surface making it difficult to predict the soil-rock contact. Numerous auger borings were taken (Figure 11) for the purpose of defining the soil-rock contact moisture

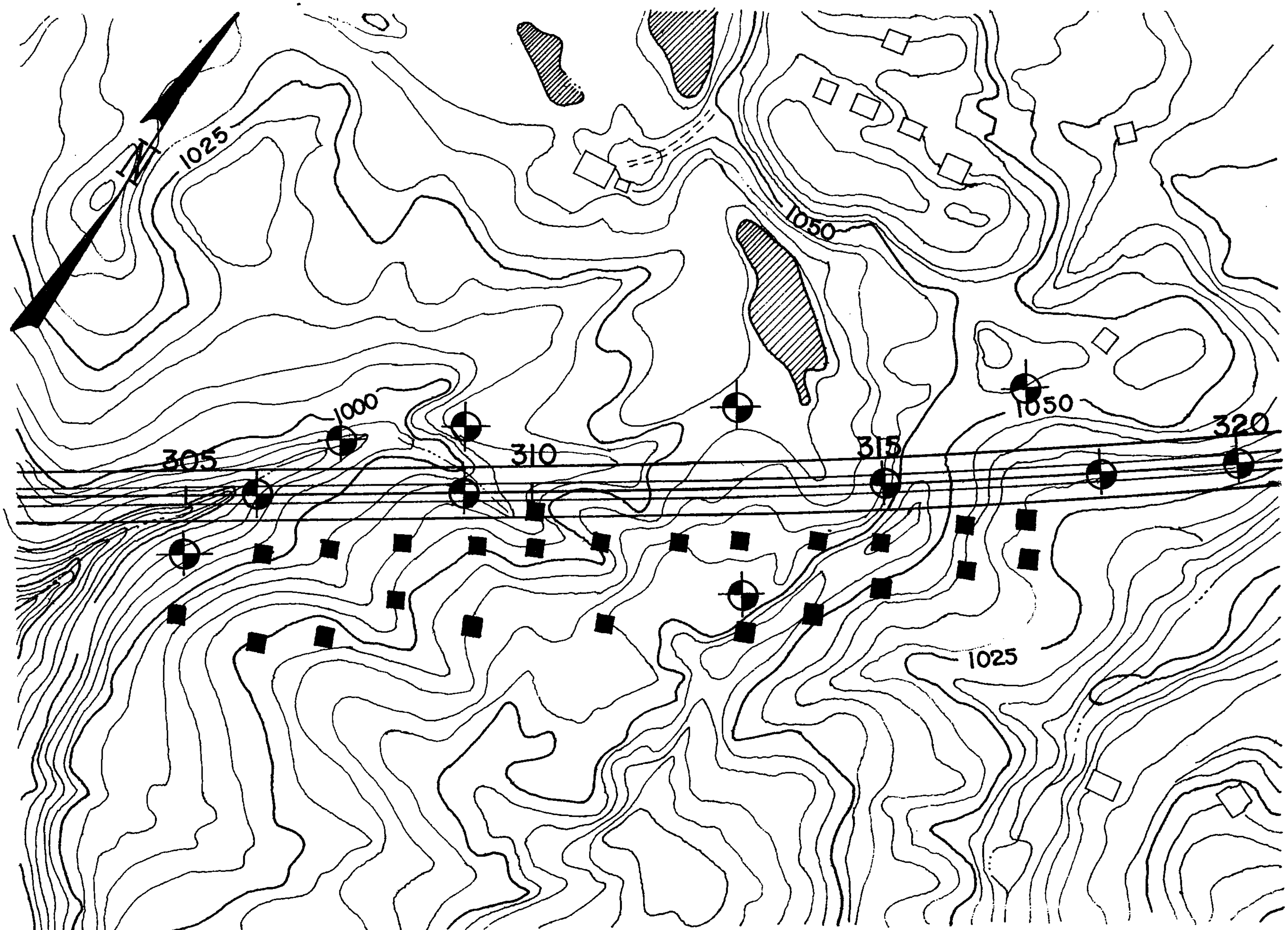


FIGURE 11 Plan View Showing Core Boring At Stop 5.

content, and strength and type and distribution of the material during the investigation for the four lane facility. Additional borings were required due to alignment and grade changes. These borings along with those taken during the initial study provided adequate information for the stable design (Figure 12).

The deposits had layers of sand, silt and heavy clay. The clays acted as impermeable barriers for vertical flow of ground water thus the deposits contained almost a uniform forty percent moisture content. The clays were both very wet and weak in their natural condition.

The original design for the soil slopes for the two lane facility were on a two (2) horizontal to one (1) vertical and the slopes in the shale were on a three-quarter (3/4) horizontal to one (1) vertical. Due to the fact that the initial borings were for the four lane facility at a different grade some of the shale slopes were found to be in soil. After the slopes had lain through the first winter during construction, it was apparent that the angle of repose for the material even when on the two (2) horizontal to one (1) vertical was too steep for the inherent conditions. The final design incorporated a bench at the soil-rock contact with a three (3) horizontal to one (1) vertical through the deposits with immediate seeding to deter erosion (Figure 12).

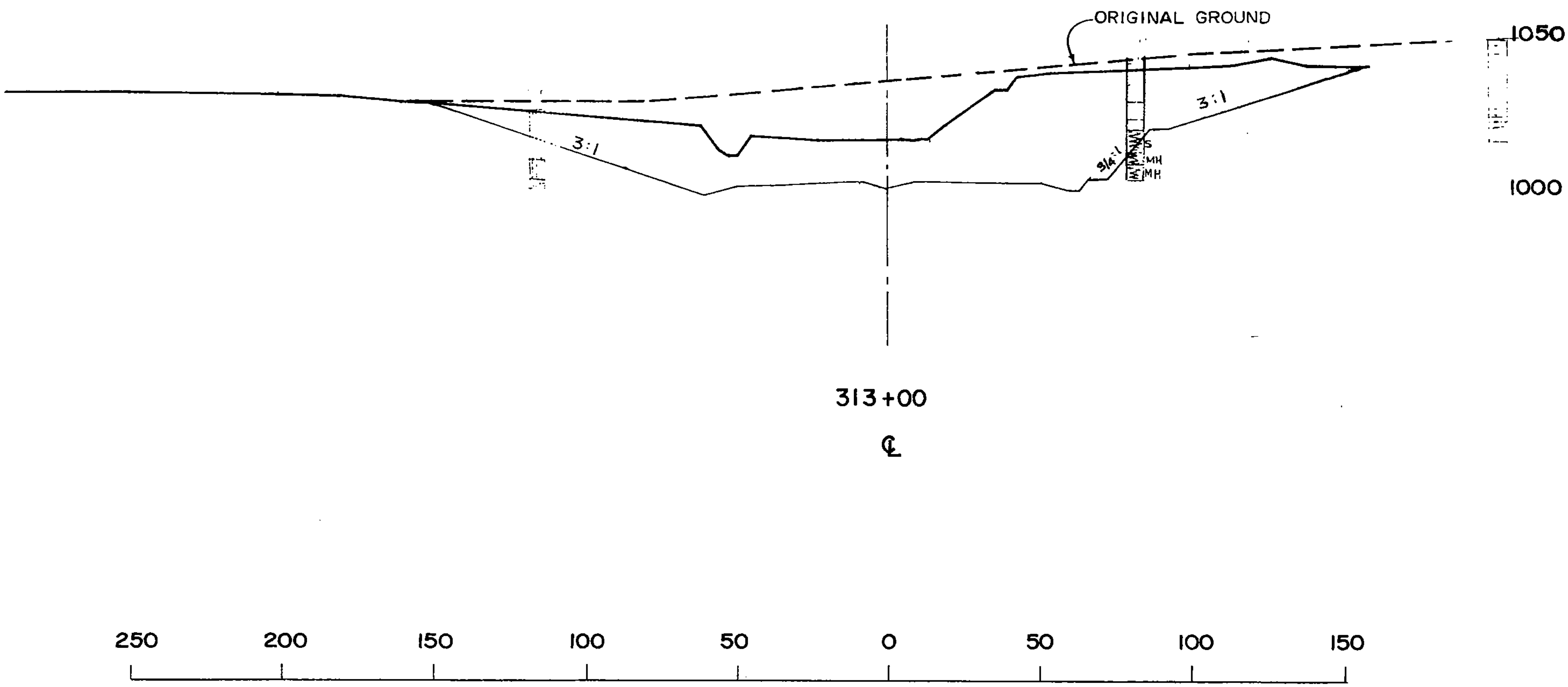


FIGURE 12 Cross Section and Design at Stop 5

The roadway across the river was at grade in a similar deposit. The depth of the unsuitable material below subgrade varied from 0 to 13 feet which was uneconomical to remove to the soil-rock contact. It was considered unsuitable due to the silt and moisture content. The deposit was so wet and quick that normal construction equipment could not operate and construction in the area had to cease until corrective measures could be designed and placed. Underdrain was installed along with a cement treated stone cover to bridge the material.

STOP 6: A DESIGN METHOD FOR ROADWAYS WHERE COAL HAS BEEN MINED BELOW GRADE.

The Pittsburgh coal at Station 2330+00 is within fifteen (15) feet of the grade line. The coal was deep mined and in the past has been on fire in adjacent areas but not within the past few years.

It has been the practice of various agencies to undercut and backfill where coal has been mined within thirty (30) feet of the natural ground or within thirty (30) feet of the grade line. The grade line for Interstate 79 is on a plus grade toward the northeast and the apparent dip of the coal along the grade also dips to the northeast. Thus, although the coal is within fifteen (15) feet of grade at Station 2330+00, the opposing grade and dip directions result in a gradual vertical separation of the coal seam and the grade as stations increase.

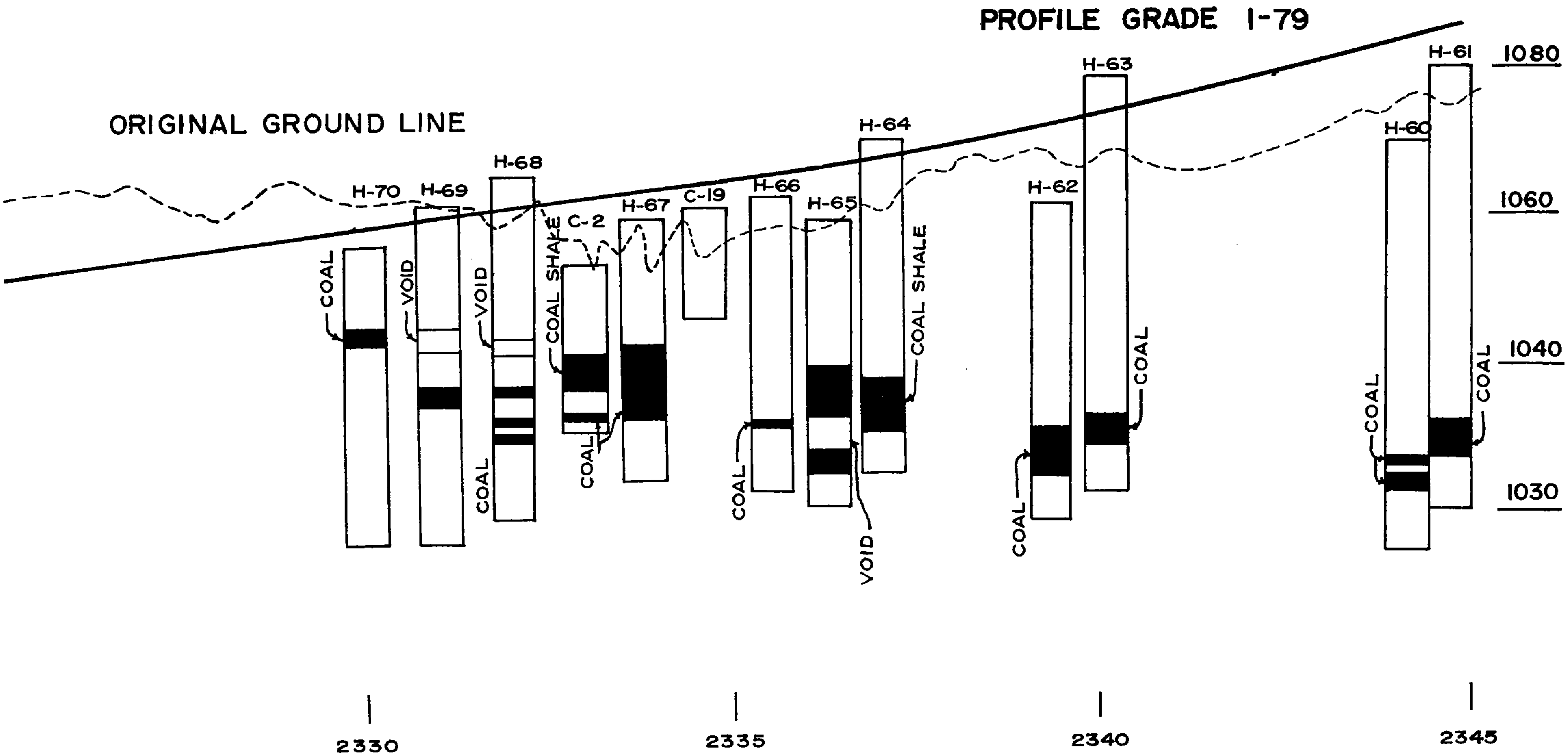
The profile of borings and grade, and the coal seam are shown in Figure 13.

Between Station 2330 and 2336, it was the recommendation of all who were involved in the design that the area be undercut as there was insufficient rock overburden to adequately support the roadway. It would have been desirable to continue the undercut further in order to provide a barrier should the coal be reignited and to insure against future differential settlement; however, due to the depth and consequential economics, only the area between Stations 2330 and 2336 was designated for this treatment. In addition an onsite inspection by a Road Commission Geologist was requested during construction to determine where the undercut should be terminated.

From the onsite inspection, it was determined by the Geologist that due to the presence of small voids and the degree of collapse, the originally proposed limits for the undercut be adopted.

A flexible pavement was recommended to compensate for anticipated future differential settlement. The design (shown in Figure 14) has produced very little distress to date.

NOTE: We now follow Interstate 79 south to the end of construction, approximately 5.2 miles. We then follow State Route 73 northward to the I 79 On Ramp for STOP 7. In the area just southeast of STOP 6 (See Figure 2), note the rolling nature of State Route 73 caused by subsidence. It should also be noted that in places here several inches of asphalt have been laid down as a remedial measure.



PROFILE I-79

STA. 2330+0 TO STA. 2345+0

FIGURE 13

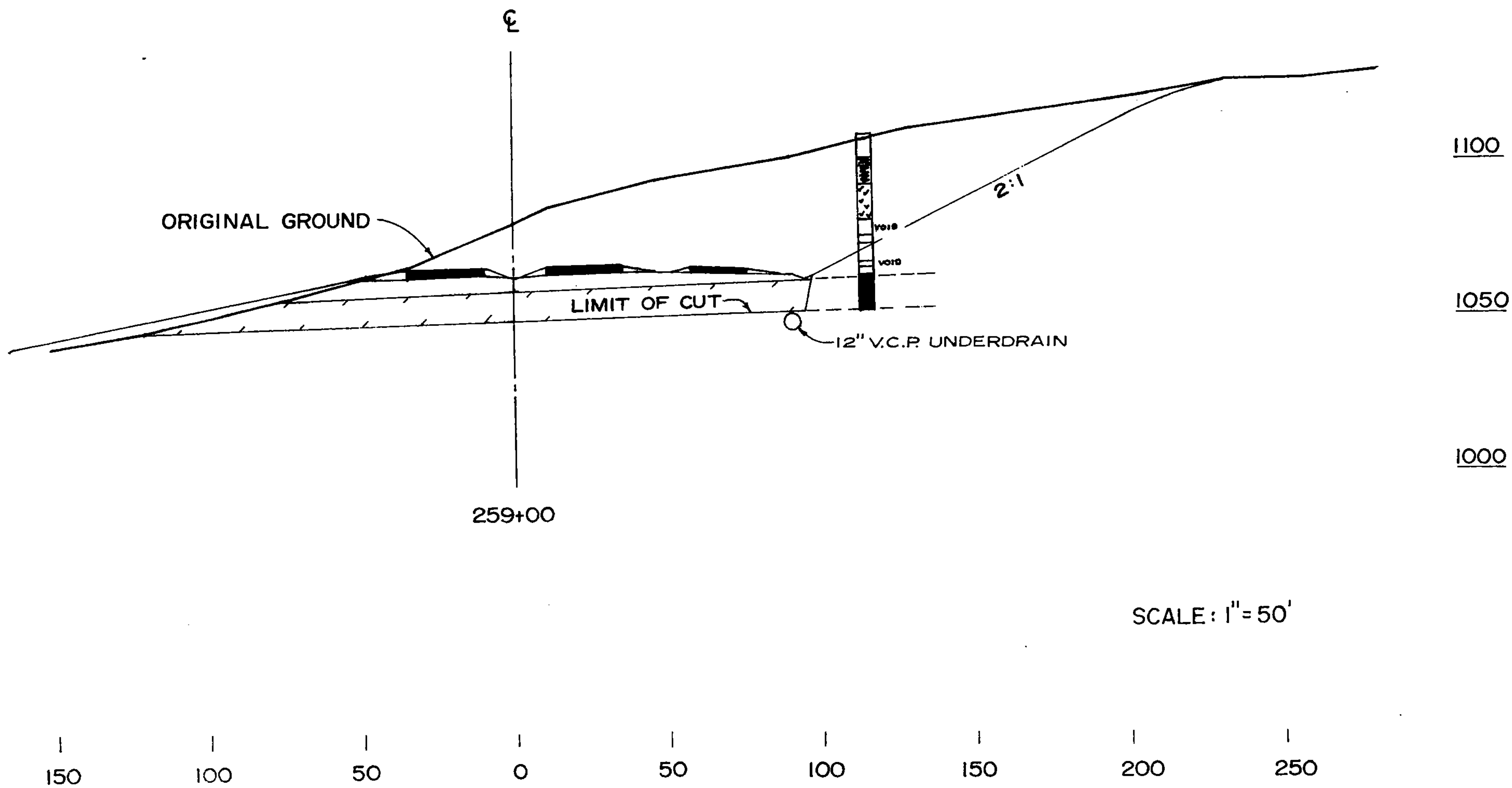
STOP 7: A DESIGN METHOD FOR ROADWAYS WHERE COAL HAS BEEN STRIP-MINED AND DEEP MINED AT GRADE.

During the Soil and Geological Data Investigation, the Pittsburgh coal was found to be either at grade or only a few feet below grade at this location. In addition deep mining and strip mining was conducted previously in the hillside adjacent to the roadway. The removal of the coal was sufficient to cause the overlying formations to lose some of their structural stability and to collapse.

The Pittsburgh coal dips toward the roadway in this area and the abandoned mine had an undetermined amount of water trapped in it. It was necessary to drain the mine and provide drainage of the coal. This was accomplished by carefully opening the original entrance and allowing the head of water to bleed off. Some of the coal was in place and the grading contract called for removal of the coal beneath the roadway and back filling with random material.

Underdrain was also installed behind the constructed slope and extended below the coal. The slope design adopted for the cut, as indicated on the included cross section at Station 259+00 for the NE Ramp (Figure 15) is a two horizontal to one vertical which provides a minimum safety factor of approximately 1.6. The slope was shot to insure as much total collapse as possible as borings indicated that voids up to six feet were still present behind the slope. In other areas, both total and partial

FIGURE 15 Cross Section of Northeast Ramp at Stop 7



collapse had occurred with voids dispersed throughout the upper strata. It was desired to have a uniform slope that would not have further distress and possibly cause a loss in stability of the slope above any surface settlement.

There were also openings in the area of the I 79 off ramp on the south side of US 250. A fill was designated to be placed over the area which had several abandoned mine entries some of which had partially collapsed. The area was drilled and shot on a nonuniform spacing to guard agsinst structural instability of the fill at some future date. Approximately 0 to 6 feet of fill was placed over the area prior to shooting to aid in the collapse. After shooting was complete there was little surface evidence that any extensive collapse had developed. There has been little or no distress evident in the roadway since it was completed in 1963.