PROCEEDINGS
OF THE
FIRST, SECOND, THIRD & FOURTH
HIGHWAY GEOLOGY SYMPOSIUM

VOL. I

First Annual Symposium on

"GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING"

April 14, 1950

Under the Sponsorship of

DEPARTMENT OF HIGHWAYS

RICHMOND, VIRGINIA

SYNOPSIS OF FIRST HIGHWAY GEOLOGY SYMPOSIUM

By

W. T. Parrott, Engineering Geologist

Virginia Department of Highways

To foster the exhange of ideas between highway engineers and geologists on problems relating to highway construction, the Virginia Department of Highways sponsored a symposium on "Geology as Applied to Highway Engineering" in Richmond on April 14, 1950.

Attending were representatives of the highway departments of Georgia, South Carolina, North Carolina, Virginia, Kentucky, West Virginia, Maryland and Pennsylvania. Other organizations represented included the United States Geological Survey, the Virginia Geological Survey, the North Carolina Commission of Conservation and Development, the United States Army Engineers, the National Park Service, the United States Department of Agriculture (Soils Engineers), the Bureau of Public Roads, the Engineer School at Fort Belvoir, and faculty members and students from the University of Virginia, Virginia Military Institute, Virginia Polytechnical Institute, and Washington and Lee University.

The meeting was opened with an address of welcome by C. S. Mullen, Chief Engineer of the Virginia Department of Highways. James A. Anderson, Commissioner, Virginia Department of Highways, then outlined the general purpose of the meeting.

The first paper was presented by A. Stinnott of the Ground Water Division, United States Geological Survey. He discussed the various methods of the control of ground water in unconsolidated settlements.

He briefly outlined the gological factors involving the collection and dispersion of ground water with various means of controlling the same in order to prevent damage to highways under construction as well as the elimination of costly maintenance work due to the action of ground water.

Mr. Stinnott's paper was followed by one by Dr. Jasper L. Stuckey, State Geologist of North Carolina. Dr. Stuckey discussed the importance of geological surveys in highway location and design. He gave a brief review of the principals of geology and the formation of various rocks. These rocks were treated from the standpoint of genesis, structure, and weathering, and their direct relationship to the importance in highway engineering. He discussed the importance of geological mapping, bringing out that this particular branch of mapping would enable the highway engineer to visualize the type of material over which his road would pass, enabling him to design the road in order to take care of unfavorable geologic conditions which might otherwise pass unnoticed. Dr. Stuckey emphasized the importance of soils, soil sampling, and soil mapping.

"Roads are founded on geologic materials and built of geologic materials," he concluded. "When the engineer has done his very best construction job, the road is no better than the foundation on which it is laid and the materials of which it is built."

Dr. Stuckey was followed by Mr. D. D. Woodson, Soils Engineer of the Virginia Department of Highways. Mr. Woodson spoke on soils as correlated with parent material. He discussed the various physiographic provinces of Virginia and the principal rock types from which the soils in each of these sections were derived. He gave a brief account of the action of the resulting soil, its mineralogical content, and the engineering qualitites of each of the soils so encountered. He stressed the importance of geological maps which enable the soils engineer to better forecast or correlate the probable action of the soil which overlies its parent rock. He concluded by stating that a close liaison of the work done by the geologist and the soils engineer would go far toward the solution of many problems which confront the soils engineer.

In a lively discussion following Mr. Woodson's paper, the group considered the various aspects of how the types of behavior of the soils in other States reacted in comparison with similar soils found in Virginia.

Robert A. Laurence, Regional Geologist of the United States Geological Survey, presented a paper on geologic factors involving land slides and rock falls. Mr. Laurence classified types of land movement in accordance with the classification given by C. F. S. Sharpe: (1) slump, (2) debris fall, (3) debris slide, (4) rock slides, and (5) rock falls. Each type of land movement and geologic conditions which caused it was illustrated. He concluded that, while many slides and rock falls cannot be avoided, a thorough geologic study will often indicate either a way to prevent a slide by changing the location or alignment of the cut or will indicate a method of stabilizing the slide.

Dr. L. W. Currier of the United States Geological Survey gave a paper on Federal participation in geologic materials surveys. He outlined the various types of surveys in which the United States Geologica Survey is participating. These surveys, he said, had been made in Kansas, Wyoming, and Montana. At the present time, he said his organization is cooperating with the Bureau of Public Works in Massachusetts in publishing a geologic map of this State. He emphasized that while the survey is not in the business to map for individuals or consultants, it quite often sends out field parties to work with various State agencies in publishing geologic construction material maps. The various types of maps ranging from the simple spot map to a complete geologic map of an area were illustrated and the value of each shown.

J. C. Stevens of the Virginia Council of Highway Investigation and Research gave a paper on the use of aerial photographs in engineering geology. He pointed out that various land forms could be recognized by their different characteristics. He added that various soil patterns could be interpreted due to their drainage characteristics. In addition to the above mentioned guides, it was pointed out that highway location could be speeded up by the use of these photographs in laying out preliminary base lines. The drainage pattern of the various streams which to some degree give the types of land forms were also discussed. All of the points mentioned in Mr. Stevens' paper were illustrated by slides.

Dr. R. W. Moore of the United States Bureau of Public Roads presented the subject of geophysical methods for subsurface exploration in highway construction. He explained the basic differences between the seismic method and the resistivity method. Each of these methods will give an accurate profile of any proposed road down to solid rock if they are used in a country which is subjected to leaching such as limestone and dolomite; they will not pick out cavities or solution channels, thus they do not take the place of borings for structures such as dams and bridges.

The final paper was written by Mr. S. E. Horner, chief geologist of the State Highway Commission of Kansas, on "Engineering Geology as Applied to Kansas Highway Problems." Mr. Horner was unable to be present and his paper was read by Dr. Raymond S. Edmundson of the University of Virginia. Subjects discussed by previous speakers were covered and the paper served as an excellent resume of the entire meeting.

Before adjournment, a tour of the Virginia Department of Highways laboratories was conducted.

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PROGRAM

Symposium on Geology as Applied to Highway Engineering

Department of Highways - Auditorium

Richmond, Virginia - April 14, 1950

9:00	A.M.	Registration - Lobby - Central Office Virginia Department of Highways

Presiding - Mr. W. T. Parrott, Engineering Geologist

9:30	A.M.	Address of Welcome - C. S. Mullen, Chief Engineer - Virginia Department of Highways
9:40	A.M.	Purpose of the Meeting - Gen. James A. Anderson, Commissioner Virginia Department of Highways
9:50	A.M.	Groundwater Control as Applied to Consolidated Rocks - William M. McGill, Virginia State Geologist
10:20	A.M.	Groundwater Control as Applied to Unconsolidated Sediments - A. Sinnot, United States Geological Survey, Groundwater Division
10:50	A.M.	The Importance of Geological Surveys in Highway Location and Design - Dr. Jasper A. Stuckey, North Carolina State Geologist
11:20	A.M.	The Correlation of Soils and Their Parent Material - D. D. Woodson, Soils Engineer - Virginia Department of Highways
11:50	A.M.	Adjournment for Lunch
1:00	P.M.	Geologic Conditions Affecting Landslides and Rockfalls - Robert A. Laurence - Regional Geologist - United States Geological Survey
1:30	P.M.	Federal Participation in Geologic Material Surveys - Dr. L. W. Currier, United States Geological Survey
2:00	P.M.	The Use of Aerial Photographs in Engineering Geology - J. C. Stevens, Virginia Council of Highway Investigation and Research

2:30 P.M. Geophysical Methods for Sub-Surface Exploration in Highway Construction- R. W. Moore, United States Bureau of Public Roads

3:00 P.M. Engineering Geology as Applied to Kansas Highway Problems - S. E. Horner, Chief Geologist, State Highway Commission of Kansas - to be presented by Dr. R. S. Edmundson, University of Virginia

3:30 P.M. Business Meeting

4:00 P.M. Tour of Laboratories

4:45 P.M. Adjournment

Proceedings

Second Symposium of

"GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING"

February 16, 1951

Sponsored by

VIRGINIA DEPARTMENT OF HIGHWAYS

RICHMOND, VIRGINIA

SYMPOSIUM ON GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING

Department of Highways - Auditorium

Richmond, Virginia - February 16, 1951

The Adhesion of Bituminous Materials to Highway Aggregates - A. B. Cornthwaite, Assistant Testing Engineer, Virginia Department of Highways

Geological Enterprise in Virginia - Present and Future - Dr. B. N. Cooper, Professor of Geology - Virginia Polytechnic Institute

The Importance of Geology in Military Highway Construction - Frank C. Whitmore, Jr. Chief, Military Geology Section, United States Geological Survey

Injurious Minerals in Highway Aggregate Dr. Duncan McConnell - Professor of Mineralogy Ohio State University

What Does the Engineer Expect of the Geologist Professor A. T. Granger - Professor of Civil Engineering - University of Tennessee

The Use of Plate Bearing Tests in the Thickness Design of Flexible Pavements - L. D. Hicks, Assistant Engineer Materials and Tests North Carolina Department of Highways and Public Works

The Control of Groundwater in Consolidated Sediments - George D. DeBuchananne - Geologist in Charge - United States Geological Survey Groundwater Division

The Identification of Rock Types - D. O. Woolf Senior Materials Engineer - Bureau of Public Roads

The Construction of Highway Bridges and Separation Structures in Unconsolidated Sediments - Professor Frank W. Wheeler Professor of Civil Engineering University of Virginia

THE ADHESION OF BITUMINOUS FILMS TO HIGHWAY AGGREGATES

BY
A. B. CORNTHWAITE

It is needless, perhaps, to mention the part that mineral aggregates play in the building of our highways. When it is remembered, however, that in our bituminous or black-top roads, aggregates comprise approximately 95% by weight of the system, and that in plain portland cement concrete roads they account for approximately 80% by weight of the structure, it is readily seen that for the tens of thousands of miles of highways in the United States the quantities involved runs into astronomical figures. Added to this amount should be the thousands of tons used in the stabilizing and building up of thousands of miles of unsurfaced county or secondary roads.

For our discussion today, we are not so much interested in the quantity of the mineral aggregate used, but in the relationship of these aggregates to the bituminous materials.

Virginia is geologically blessed by having so many different aggregates of such good quality and quantity for use in building highways. Our aggregates vary from sand and gravel in the Tidewater and Piedmont areas to granites, limestones, gneisses, trap rocks, etc., in the Piedmont and mountainous regions. For each of the different types of aggregates you geologists recognize that there are many different geological formations of different ages which identify rocks of the same general classification.

In our laboratories we classify them by their hardness, or resistance to abrasion, their soundness, or resistance to freezing and thawing, and by their behavior with bituminous materials.

The behavior of these aggregates in highways sometimes leads the highway engineer to believe that they have not been classified into a sufficient number of catagories since it often seems that each individual particle behaves differently from the one next to it.

In combining aggregates and bituminous materials to build road surfaces, use is made of three different phases of the bituminous material: solid, semi-solid, and liquid. The solid and semi-solid materials require the use of considerable heat for their proper manipulation, and for that reason they are not readily adaptable to field conditions. They are, however, widely used in central mixing plants where both the aggregate and the bituminous material can be heated to the proper mixing temperatures, mixed, and the resultant mixture then applied to the prepared roadway, rolled into place and the job is complete.

The liquid materials naturally lend themselves to a much easier application and manipulation in the field than either the semi-solid or solid materials and for that reason are more widely used.

It has long been recognized that the presence of water in and around bituminous pavement structures causes more damage than perhaps any other one factor. This is the reason highway departments build such elaborate drainage structures, and periodically clean and maintain their drainage ditches.

This damage from water may exhibit itself in a number of different ways but of primary interest to us is the fact that the water causes a definite lack of adhesion of the bituminous film to the aggregate surface. Ultimately this lose of adhesion means that under traffic the aggregate particles will be displaced and the pavement begin to deteriorate.

All of you know how difficult it is to wet an oily surface with water - it is practically impossible. This same condition is encountered when trying to coat a wet aggregate surface with bituminous material. Without using a material that has been especially treated, this can't be done either. On the other hand, aggregates which have been coated with bituminous materials and later subjected to excessive moisture will in time tend to lose their bituminous coating which then permits the rapid destruction of the pavememt. It is not only the adhesion of the bituminous material to the aggregates that is affected, but the mechanical stability of the road surfaces is also weakened by the loss of cohesion between particles and under traffic the surface is destroyed.

The seriousness of this adhesion problem to highway departments was well demonstrated in Virginia about fifteen years ago when approximately fifty miles of road surfaces were lost due to the entrance of moisture into the pavement structure. This was rather a severe blow and very careful attention has been paid to the problem since that time in order to prevent its recurrence or to keep this type of damage to a minimum.

It is not intended to give you the impression that Virginia is the only state in the nation faced with the problem of obtaining satisfactory adhesion of bituminous materials to mineral aggregates under all possible climatic conditions. The problem has received nation-wide attention in recent years and has also been studied in many road research laboratories in Great Britain and Europe. The American Society for Testing Materials and the Highway Research Board both have committees actively working on the problem attempting to determine methods of evaluating the resistance of bituminous films to the effect of water, and the relationship between the bituminous films and the aggregate whether it be a question of surface tension, interfacial tension, or a combination of the two. The Bureau of Public Roads has also contributed very materially to this study.

Among highway engineers it is common to express this lack of adhesion by the word "stripping" and the degree of stripping in laboratory studies is considered to be a measure of the adhesive qualities existing between the bitumen and the aggregate.

To improve the adhesion of bituminous films to aggregates certain chemicals have been developed which we call additives and which chemically are closely related in action to the detergents in common use today. By their use the surface tension of the bituminous materials is altered to the extent that it is possible to not only cover and coat wet aggregates but also to enable them to retain that coating under adverse weather conditions. With the liquid bitumens being most widely used, it is in this field that the use of additives has been most pronounced.

As mentioned before, in order to secure proper coating of the aggregate when using solid or semi-solid bitumens it is generally necessary to heat not only the bituminous material to such a temperature that it will flow readily, but also to heat and dry the mineral aggregate. Both of these conditions are conducive to good adherence of the bituminous film and proper wetting of the aggregate surfaces. This does not mean that the solid materials are immune to the detrimental effects of water but their performance in this respect is generally superior to the liquid bitumens. Since the additives used to improve adhesion are organic compounds and as many are readily decomposed at the mixing temperatures required, the use of these additives in this class of products has been rather limited.

Not to slight the aggregate side of the story, it is also possible to improve the characteristics of the mineral aggregates by treating them with certain chemicals so that better adhesion is obtained. It may be somewhat surprising to the geologist to learn that any material that has existed some few millions of years could possibly be improved, but a great many patents have been issued for these processes.

However, the treatment of aggregates has always seemed to be doing things in the hard way for the reason that we would be treating 95% of the road structure, whereas by the use of additives in the bituminous material it is necessary to treat only 5% of the structure. Also, economically it has been found to be more feasible to treat the bituminous material than to treat the aggregate. For example, it we assume the additives to increase the cost of the bituminous material by two cents per gallon then for an application rate of 0.25 gallons per square yard the additive would cost 1/2 cent. For treating aggregates the cost may vary from $.60 to $1.00 per ton, and if the aggregate is applied at the rate of 25 pounds per square yard the increased cost would be from 3/4 cents to 1-1/4 cents.

There have been a great variety of methods developed whereby the stripping of bituminous materials from aggregates can be studied. These methods can be divided into two rather broad and comprehensive groups. The first group would include those methods in which the coated aggregate particles are immersed in water as individual particles, and in the second group the effect of water on the bitumen coated aggregate is studied by determining the compressive strength of laboratory prepared cylinders.

One of the original water immersion procedures was developed by Victor Nicholson (1) and reported in the Proceedings of the Association of Asphalt Paving Technologists in January, 1932.

In this method the coated aggregate is allowed to cure under certain specified conditions after which the mixture is immersed in water and agitated for fifteen minute periods at temperatures of 77°F., 100°F., and 120°F. The percentage of stripping taking place during each test period is estimated visually. This method received wide acceptance for a number of years but has now been

(1) Adhesive Tension in Asphalt Pavements, its Significance and Methods Applicable to its Determination. Victor Nicholson. Proc., Association of Asphalt Paving Technologist, Jan. 1932.

largely superseded by other methods.

In 1933 Riedel and Weber (2) reported on the results of their studies on the "Adhesiveness of Bituminous Binders on Aggregates" in which the coated aggregate was subjected to the action of boiling water. The primary objection to this procedure has been that this is a condition which would never be obtained in pavements and therefore could not clearly represent what could be expected to happen under normal circumstances. In addition, the aggregate particles used were of small size, minus 10 mesh, and not representative of the sizes commonly used in road building. It does point out, however, the vulnerability of bituminous films of low viscosity, or of those materials that have been heated above their softening points to the action of water.

In recent years the most emphasis has been placed upon static immersion stripping test in which the coated aggregates are immersed in water at room temperature or at slightly elevated temperatures for a period of from 18 to 24 hours. At the end of this time the particles are rated as to the amount of stripping that has taken place during the immersion period. This is the type of test that Virginia follows in determining the compliance of bituminous materials purchased under our adhesion test requirements. As a matter of fact, we have two adhesion tests - in the first test we use a dry dolomite aggregate, which represents a basic type of aggregate, and in the second test we use wet silica

(2) On the Adhesion of Bituminous Binders on Aggregates. W. Riedel and H. Weber. Asphalt und Teer, Vol. 33, September 13, 1933.

gravel to represent an acid type of aggregate. It is required that the aggregates retain 90% of their bituminous coating after 18 hours immersion.

In 1945, J. T. Pauls and H. M. Rex of the Bureau of Public Roads(3) reported on a study they had made in which mixtures of aggregate and bituminous materials were compacted into 4" x 4" cylinders and then tested for compressive strength in the dry state. Other duplicate compacted specimens were immersed for periods up to one week at different temperatures after which their compressive strength was determined. The compressive strength of the immersed specimens expressed as a per cent of the dry compressive strength of duplicate samples is considered to be a measure of the resistance of the mixture to the effect of water. This test has been commonly known as the Immersion-Compression Test and is representative of the second group of methods developed for studying adhesion.

This method has the advantage of permitting many variations by which the effect of different aggregates, sands, mineral fillers, gradation of the aggregates, and types and grades of bituminous material can be studied.

It has the additional advantage of permitting the determination of actual values in pounds of compressive strength of the dry and wet specimens and thus eliminating personal estimations.

The American Society for Testing Materials has recently

(3) A Test for Determining the Effect of Water on Bituminous Mixtures. J. T. Pauls and H. M. Rex. Public Roads, Vol. 24, No. 5, July, August, September 1945.

published this method as a Tentative "Method of Test for Effect of Water on Cohesion of Compacted Bituminous Mixtures", D-1075-49T.

Two methods of evaluating the amount of stripping have commonly been used. One is a visual estimation of the per cent of aggregate remaining coated after the immersion period giving total coverage a value of 100%. The second method is a hand separation of the particles showing any stripping from those remaining completely coated, and determining the per cent of the particles remaining completely coated based on the total weight of the sample under test.

In an effort to find a more suitable method of estimating the amount of stripping some attention is being given at the present time to the evaluation of stripped aggregates by means of comparing them with a series of standard area black and white photographs.

The absorption of water-soluble dyes into the uncoated areas of the stripped aggregate, followed by photo-electric light reflectance measurements, is another possible means of eliminating the human-equation in rating stripping.

In the study of adhesion of bituminous materials to mineral aggregates, two descriptive terms have been applied to the aggregates. Those aggregates which tend to retain their bituminous coating in the presence of water have been called hydrophobic, which means 'water-hating'. It has generally been assumed that the hydrophobic aggregates are the more basic type of rocks such as limestones and dolomites.

Those aggregates which readily lose their coating of bituminous materials in the presence of water have become known as hydrophyllic, which means 'water-loving'. In general rocks of this type are the acid rocks such as quartzites and granites.

In connection with the Department's State Wide Aggregate Survey, which was started in 1947, the stripping characteristics of some 722 samples of aggregates have been determined. That there is a difference in behavior of the acid and basic type of rocks with bituminous materials is shown in the following table.

STRIPPING RESISTANCE

	Per Cent by Wt.	Per Cent Estimated	No. of Samples	No. of Counties
Limestone	65	89	312	29
Granite	25	64	37	18
Gneiss and Granite Gneiss	32	68	78	22

Note: The stripping resistance has been calculated as an average of the average from each county.

Here it will be seen that for 312 samples of limestone from 29 counties an average value of 65% adhesion was determined when figured by weight, and 89% adhesion when estimated visually, Contrasted to this, for 37 samples of granite from 18 counties and average value of only 25% adhesion by weight and 64% by visual estimation was determined. Also, for 78 samples of gneisses and granite gneisses from 22 counties average values of 32% and 68% adhesion was determined. In this study, the same asphalt and the same procedure was followed throughout.

There are other characteristics besides the mineralogical composition of aggregates which appears to have a great effect upon the degree of adhesiveness of bituminous films to their surfaces. For example, those aggregates which have a rough texture tend to retain the bituminous film much better than one whose surface is plane or glassy. Aggregates which are porous or highly absorptive also improve the adhesiveness of bitumens.

Mention has been made of the importance of the consistency of the bituminous film in resisting the entrance of water. In addition to consistency the temperature of both the bituminous material and the aggregate at the time of mixing seems to be quite important.

Road surfaces which are laid under the favorable conditions of hot and dry summer weather are much less apt to fail when the new treatment is followed by rainy weather than those surfaces laid during the cool damp days of early spring and late fall and then rained upon. In the Highway Department we recognize this condition by generally specifying that bituminous materials for use in winter or cold weather must meet our Type II Adhesion Test - i.e. - have the ability to coat wet silica gravel and retain that coating during 18 hours continuous immersion. The aggregate in this case is an "acid" type.

The period of time required for the curing or "setting-up" of the liquid bituminous materials is much shorter in summer than it is during the spring and fall. This, of course, means that the loss by evaporation of the solvent or thinner in the bituminous material is greatly accelerated by the high temperatures of summer, resulting in a much more viscous bituminous film that is highly resistant to moisture.

The age of the bituminous film seems to play an important part also in improving its adhesion to aggregates. Most coated aggregates that have been stored for a considerable length of time will show better adhesion than a freshly mixed sample. Apparently, the adhesive bond between the bitumen and the aggregate becomes more firmly established.

There is another thing not so widely recognized which appears to play a part in obtaining satisfactory adhesion which has been described as a surface effect. In this case bituminous films apparently become set and are highly resistant to the effects of water until the surface has been disturbed through some mechanical action when it is found that the underlying material is as vulnerable to the action of water as the original mixture would have been.

The effects of traffic on a road surface of this nature can be readily visualized during a rain storm. The disruption of the film by traffic permits the ready entrance of moisture into the system, resulting in the displacement of the bituminous material, loss of stability, and finally displacement of the aggregate and loss of the road surface.

In conclusion, I would like to point out that Virginia has been one of the pioneers in focusing attention to the urgent need of improving the adhesion characteristics of our bitumen-aggregate mixtures. We have worked very closely with many of the promoters of the "additives", and, certainly for some of them, have been instrumental in their developing better products. Our laboratories for a long time were the proving grounds of the early products.

If nothing else, we have been instrumental in focusing the attention of the refineries to the necessity of properly selecting the crudes from which the bituminous materials are obtained, and the selection of proper solvents to obtain the best curing properties possible with a resultant increase in resistance to stripping.

Contrary to our solution of the stripping problem where we use only two aggregates to evaluate all of our bituminous materials: - whether they are to meet either Type I or Type II Adhesion Test - our work, and that of all other investigators, has conclusively shown that each combination of bitumen and aggregate is a problem unto itself. Unfortunately, under competitive bidding practices, it is practically impossible to take advantage of this fact and always use the best possible combination.

Definite progress has been made in the solving of the adhesion problem. We have probably overlooked a good bet in not acquainting the geologist with our problem and enlisting his aid. After all, it may be easier to take the mountain to Mohammed than to take Mohammed to the Mountain.

GEOLOGIC ENTERPRISE IN VIRGINIA---PRESENT AND FUTURE

Byron N. Cooper
Virginia Polytechnic Institute

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For a few minutes, I would like to consider with you some conditions and situations prevailing in Virginia, which should be the concern of all thinking persons whose enterprise involves the use of earth materials.

Primarily, this conference is concerned with geology as applied to highway engineering. As we all know, geologic science properly applied to engineering procedures is demonstrating its true worth every day. All of us attending this meeting are keenly interested in the particular applications of geological techniques to highway engineering practice. I hope that you will bear with me while I explore with you certain general and, I believe, more fundamental matters involving the potential usefulness of geologic research to all types of enterprise in Virginia.

The daily application of geological principles to engineering practice has doubtlessly convinced all of you of the need for more and more useful information on the rocks, minerals, soils, and topography of Virginia. The engineering geologist does not have the time to do much in the way of basic geological research; he is hired for more immediately urgent purposes.

Where does the engineering geologist get his basic information on earth materials? He gets it largely from publications of the State Geological Survey. Of course, he also gets help from the Federal Geological Survey reports and maps. However, the basic responsibility for investigating the rocks, minerals, groundwater, and topography of Virginia is assigned by law to the State Geological Survey, a division of the Department of Conservation and Development. Most of us know we have a State Geological Survey, and some of us kmow its personnel and something of what the Survey is doing. How are we in Virginia progressing with our survey of the rocks and mineral resources of the Commonwealth? Are we nearly through with the survey? How long will it take to finish it? Naturally you expect me to answer these questions since I have raised them. Probably also you are expecting to be surprised one way or the other by my answers. Grab onto your seats - every one of you - because you are in for a severe jolt.

First, let us look at topographic mapping. It scarcely needs saying that geological field investigations are largely dependent on availability of topographic base maps. I understand also that topographic maps are still considered helpful to highway planning engineers. With certain exceptions, the making of topographic maps is a 50-50 cooperative arrangement, with the Federal Government matching State appropriations for this special purpose. It scarcely needs saying that a topographic map on a scale smaller than 1 inch equal 1 mile is so generalized as to be virtually useless for modern field work. Since about 1915, the U. S. Geological Survey has sought to map the civilized parts of this country on an inch-to-the-mile scale with contour intervals ranging from 5 to 50 feet, depending on the local relief. After 35 years mapping on a scale of an inch-to-the-mile, it might be supposed that Virginia would have been more or less completely mapped. Actually, this is far from true. Indeed, parts of Virginia have not been mapped topographically at all - some 3,000 square miles of it. Have you ever tried to find out anything about the topography, bedrocks, or mineral resources of Floyd County? If you have you know what a complete vacuum is.

Parts of Virginia are well mapped. The TVA has graciously taken over the mapping of the Tennessee River drainage area, which includes Scott, Washington, Smyth, Russell, Tazewell, Lee, and parts of adjacent counties in Virginia. Within a few years, this area embracing some 3,500 square miles or 8 per cent of the area of the State will be covered with $7\frac{1}{2}$-minute contour maps on a scale of 1:24,000. These maps have already proved of inestimable value in geologic mapping. It is fortunate that so large an area of the State is being mapped in detail without direct cost to Virginia. This is not the only part of the State that is being mapped without direct outlay of State funds. The U. S. Coast and Geodetic Survey naturally has to map the coastal areas of Virginia. The United States Army Corps of Engineers has for deadly serious reasons found it necessary to make 1:24,000 and 1:25,000 contour maps of a wide strip of the Eastern Seaboard. Taken together, the Coast and Geodetic Survey and Corps of Engineers maps cover an area of about 17,750 square miles or 41 per cent of the total area of Virginia. As a result of these mapping programs, Virginia in 50-50 co-operation with the United States Geological Survey carries on a mapping program that now needs to apply to only half of the total area of the State. But this is not as good as it sounds. Considering that Virginia currently appropriates $25,000 for topographic mapping, which is matched by Federal funds, we are spending annually just about enough to survey and publish an engraved topographic map of an area of 200 square miles---or about three-fourths of one 15-minute inch-to-the-mile quadrangle. Since there are 38 fifteen-minute quadrangles yet to be mapped on a scale of one inch to one mile, it follows that it will be the year 2,000 before Virginia will have completed her basic mapping program - unless some revolutionary improvement takes place to accelerate this work. We cannot wait that long; the rest of the world moves at a faster pace. I need not remind you that the United States Geological Survey has already decided as a matter of future policy that topographic map coverage on a scale smaller than 1:24,000 is not suited to the present-day needs of thickly settled areas. Now if we consider that we should have 1:24,000 contour maps of all of Virginia to keep pace with future developments, our current budget for topographic mapping projected into the future could not provide 1:24,000 contour maps for all of Virginia before the year 2,100. Virginia ought to be spending $100,000 a year on topographic mapping to provide for the basic and inescapable needs of 10 and 15 years hence. A mapping program such as the present one---which results in the completion of two 15-minute quadrangle maps every three years is just too slow. Geologists and engineers should be concerned about this situation.

How about basic geologic research in Virginia? Of course, that has been retarded to. Taken together, detailed geologic maps in colors, so far published, cover less than 10 per cent of the State's total area. The geology of a considerable section of the State is very poorly known and has never been studied except by the most generalized of reconnaissance methods such as were in vogue 50 years or more ago. In the rugged Appalachian Mountains west of the Blue Ridge, just one single fifteen-minute quadrangle has been geologically surveyed in detail, written up in conventional bulletin form with an accompanying geologic map published in colors. There are 52 other 15-minute quadrangles and an additional 60 $7\frac{1}{2}$-minute quadrangles left to study in this way in the Appalachian area of Virginia; 55 fifteen-minute quadrangles to map in the Blue Ridge and Piedmont regions; and approximately 300 $7\frac{1}{2}$-minute quadrangles lying east of the 78th Meridian in Virginia. Compare this progress with that of some of our neighboring states and you will see that we are moving very slowly.

What sort of work is being done that provides real quantitative information on the chemical and physical properties of bedrocks and surficial sediments of Virginia? In other words, what work is being done that will provide the engineer with the kind of detailed information that he can so readily use. Very little--painfully little.

Significant geologic details are seldom discovered without detailed geologic mapping. Consider the fact that a great many of our geologic mapping units are many hundreds, and some even thousands of feet thick. Geologists are supposed to map what the Federal Geological Survey geologists lovingly call a geologic formation---which for all essential purposes of mapping is supposed to be a unit.

Have any of you ever seen this formation we call the Wissahickon schist? If you have, you will appreciate the ironic humor I gleaned from a field trip report prepared by one of my students a year or so ago. He commented that the trip covered excellent exposures of all the principal types of rock--igneous, sedimentary, metamorphic, and Wissahickon. Just what good do we geologists do when we map units as generalized as the Wissahickon? Very little good, I think. In the Appalachian Valley there are more than two dozen named stratigraphic "units" each with a thickness of more than 1,000 feet. None of these are geologic formations in any sense of the word. Refining and splitting up of the old mapping divisions of strata has barely started in Virginia. Geological enterprise in Virginia is still in the "lumping"---not the "splitting" stage.

Every Appalachian Valley geologist has had something to do with a widespread mapping unit known as the Rome formation. Recently I examined 20 geological publications containing descriptions of the Rome formation, and every one of the twenty reports described the Rome as "very heterogeneous". Some spell out heterogeneous in lengthy detail, for example (and I am quoting now from a learned report by my favorite author) "The Rome consists of maroon and green variegated shales and siltstones, ocher-yellow crumbly siltstones, brown arkosic sandstones, salmon-pink to dark-bluish gray limestones, black dolomites, and sharpstone conglomerates." Isn't that a helpful description? Little wonder that one geologist summing up the characteristically diverse features of the Rome formation concluded that "it is homogeneous in its heterogeneity." If an engineer sets out to determine the physical properties of the various geologic "units" delineated on the Geologic Map of the Appalachian Valley he gets into trouble right away. Most of the units the geologist is still mapping embody a variety of rock types. Moreover the engineer is naturally confused by the fact that the same name is applied to different types of rock from place to place. For example, the mapped unit known as the Athens formation is variously a massive, dense, fine-grained limestone just west of Harrisonburg; a heterogeneous succession about one-third black shale and two-thirds black silty limestone near Lexington; at Catawba Sanitorium, a black calcareous silty shale; a thin paper shale at Raphine; a hard arkosic flagstone north of Wytheville, and a fine-grained sandstone in the Great Knobs country south of Abingdon. Many of our formational units we delineate on geologic maps tell us very little about the types of rock and their spatial relations one to another.

About 20 years ago, the Bureau of Reclamation, largely at the behest of the eminent engineering geologist, Charles P. Berkey, delved in a big way into the microscopic petrography of rocks to be used in great engineering works. The laboratories set up by this agency were able to demonstrate the close relationships existing between rock-forming minerals present, their grain-size, and packing of particles on the one hand with the durability characteristics of various rock formations occurring in the huge reclamation areas of the West. The petrographic laboratories of this Bureau sought, and in many instances, found the answers to many basic problems. What determines the abrasion characteristics of crushed stone? What determines the durability and bonding characteristics of rock used for aggregate? How will a rock stand up under different conditions in tunnels and cuts? How expensive will it be to drill, shoot, and cut through different kinds of rock at various attitudes? What are the weathering characteristics of different rocks in and out of concrete? The answers the petrographers found to these and other questions came the hard way. Needless to say "lumpers" did not figure in the solution of these problems. The answers were found by geologists with strongly developed "hair-splitting" attitudes.

Why has Virginia been so slow in doing vitally necessary detailed geologic work? The Virginia Geological Survey is charged by law with performing the following public services:

Surveying the rock and mineral resources of the State;

Determining geological materials suitable for roadbuilding and methods for utilizing them;

Examination and classification of soils, and investigation of their particular adaptability to various crops;

Preparation and publication of economic and geologic maps of the State;

Preparation of various types of reports on the mineral resources of various districts;

Working out of cooperative arrangements with the Federal Geological Survey for topographic mapping and certain types of geological projects.

Down through the years, the Virginia Geological Survey has never received sufficient funds to perform these duties. The State Geological Survey, I assure you, has done wonders with what funds have been given it to carry on scientific work---but against the rising tide of present-day demands, the Geological Survey is now fighting a loosing battle in spite of recent increases in technical personnel. The Geological Survey now needs the help and support of every civic-minded Virginian. Let us not blame our State Geologist or his staff for the prevailing conditions; they have done a good job down through the years. In spite of their efforts, a log jam of unfilled requests for information and of completed and edited reports has built up at the Geological Survey office in Charlottesville. Some completed manuscripts have been awaiting publication for 10 years--for want of printing funds. One Survey bulletin containing vitally important information sought by industry was taken to the printer in September, 1945, and is in February, 1951, still at the printers. Geological work that is done but not published is indeed a poor investment. The Geological Survey probably has sufficient number of completed reports to use up five times the amount now allowed for printing of its reports.

If Virginia is to get its topographic-mapping program into second gear and if the Geological Survey is to be enabled to carry out the work assigned to it, funds for its operation must be materially increased. How much is necessary? Let us look at what other states are doing.

Michigan, whose annual mineral production, like that of Virginia is hitting close to 170 million dollars, spends about 250,000 dollars for the operation of its geological survey. Tennessee, with an annual mineral production of just about half that of Virginia spends about 30 thousand dollars more per year for its Division of Geology than does Virginia for her Geological Survey, and this in spite of the fact that both the United States Geological Survey and Tennessee Valley Authority carry on rather extensive geological research in Tennessee. Oregon, with a mineral production of slightly less than one-tenth that of Virginia spends almost exactly the sum Virginia spends investigating her mineral resources. Even in Missouri--the "Show Me State"--twice as much is expended for the investigation of mineral resources as is spent in Virginia---but Missouri produces annually about two-thirds the total value of mineral products produced in Virginia. What needs to be done for the Virginia Geological Survey will not break the State treasury. An adequate appropriation would simply be an investment from which the State would draw immediate, as well as long-range, returns. The scientific work done by the State Geological Survey uncovers sought-after raw materials which lead to new mineral industries. Let us see how this works.

In 1942, the State Geological Survey inaugurated a detailed State-wide study of two common types of rock---limestone and dolomite. One unit of this study was completed as Geological Survey Bulletin 62 and published in 1944. The total cost for all staff salaries on this project, costs of chemical analyses, costs of editing, and charges for printing the report amounted to 7 thousand dollars. The information contained in this little bulletin of about 100 pages was responsible for the development of new plant installations valued at more than 15 million dollars! Three brand new industries in a brief period of five years!

Kimballton---back in 1945 a forgotten little hamlet in Giles County---suddenly became transformed into a beehive of industrial activity. It is now the largest single shipping point for high-calcium chemical lime in the eastern United States. The information provided in the same little inexpensive report was directly responsible for the location of the new Lone Star Cement plant now being built at the northeast end of Tinker Mountain near Roanoke. Numerous other examples could be cited which would similarly demonstrate that investment of funds in geological work pay big dividends.

Less tangible in terms of dollars and cents worth, but nonetheless real are the many useful services provided by a geological survey to engineering enterprises. The wider and straighter our future highways become, the more and more detailed information on earth materials will be needed to build them as economically and as soundly as possible.

During 1950, agricultural enterprise in Virginia did a 450 million dollar business. To stimulate this industry, Virginia invested during the fiscal year ending June 30, 1950, a total of $1,591,629 for scientific research and for dissemination of the results of these investigations. The mineral industries of Virginia, producing commodities worth more than one-third the value of all agricultural products sold in 1950, receive nowise comparable support in the way of geological research. The sound practicality of making substantial investments in the future of Virginia agriculture by supporting a strong and vigorous research program in agriculture has been amply demonstrated down through the years. Considering the diversity of Virginia's mineral wealth, the making of substantial investments in the future of Virginia's mineral economy would likewise prove to be sound and fruitful.

The reason why I felt impelled to present this picture to you today is that you individually and as a group are in a position to know how useful geologic science can be. I am hoping that you will see fit to use all the influence you have to secure the public support and financial appropriations needed by our State Geological Survey for its successful performance. Remember that geological enterprise has no pressure group to champion its accomplishments and to explain fully what is needed in the way of State funds for geological research.

Personally, I believe that it is up to those of us engaged in various enterprises concerned with the utilization of earth materials to publicize the urgent need for more funds for geological research and more funds for geologic and topographic mapping.

Support your Geological Survey. Demand more of it. It can be of terrific service to you, if it is provided with the funds necessary to carry on the work it is supposed to do.

THE IMPORTANCE OF GEOLOGY IN MILITARY HIGHWAY CONSTRUCTION *

Frank C. Whitmore, Jr.

The keynote of military highway construction, as of most military engineering operations, is expediency. Routes of communication must be developed as quickly as possible, and the communication network must be maintained no matter what operational conditions develop. Obviously, therefore, a military road is never built unless there is absolutely no road or trail, however rudimentary, that can be developed for military use. Furthermore, if a road must be built, construction procedure is kept as simple and quick as possible. First, the best possible subgrade is located; second, this subgrade is protected by proper drainage; and, third, the absolute minimum thickness of stable courses is added above the subgrade.

The operations of the military highway engineer are further complicated by the enormous traffic fluctuations that occur under military operational conditions. Location and distribution of the miximum traffic load often bears no relation at all to the preexisting road net; therefore many roads must bear a burden many times greater than that for which they were designed, and, consequently, maintenance and repair requirements are extraordinarily heavy. To avoid increasing the great volume of traffic passing over the roads and to avoid tying up valuable trucks in long hauls of construction materials, local materials must be used.

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* Publication authorized by the Director, U. S. Geological Survey.

As a result of all these considerations, the approach taken by both the engineer and the geologist must be quite different from that taken by them in civil highway construction: they must use substandard materials as well as makeshift methods. With this necessary difference in viewpoint and the resulting difference in methods of operation, however, the geologist's major contribution does not differ greatly in its basic items from the contribution of the geologist in civil highway construction and maintenance.

He can aid in locating gravel or rock for crushing, to be used for aggregate, surface and base course, and fill. He can determine or predict the drainage properties of bedrock and soil, the depth of overburden and the depth of weathering in bedrock, the probable stability of hillside cuts, the type of banks and bottom of streams (with especial emphasis on their suitability
pontoon - p. 2
for anchoring ponton and other temporary bridges), the susceptibility of ground to frost heaving and landslides, and the location of material suitable for binder.

This type of information is developed by geologists at three general levels in the armed services. These are, in the order of increasingly detailed treatment, the intelligence level, the operational planning level, and the field consulting level. In the first of these an attempt is made to present, by means of maps with accompanying brief texts, the characteristics of soil, rock, and landforms of the area that will affect military construction. In operational planning (usually carried on in the theater of operations), the area being considered is much more limited in scope, and the problems to be solved are more definite. Accordingly more detailed information is needed, for which reason aerial photographs are likely to be used to a greater extent in construction planning than in studies designed for high-level planning use. At the field consulting level, the geologist works directly with the military engineer in a construction battalion or similar unit; here the problems faced are, of course, very specific indeed. The method of reasoning used by the geologist at all three levels of operation is essentially the same. Legget (1939, p.69) says, "The geologist analyzes conditions as he finds them; the engineer considers how he can change existing conditions so that they will suit his plans."

Besides being a student of things as they are, the geologist is also a student of things as they were, and this greatly increases his value to the engineer. By reconstructing the events that caused the present landscape, it is possible to increase the accuracy of predictions of both surface and subsurface conditions in inaccessible areas: obviously a technique of importance to the military geologist.

Such inductive analysis as this is particularly valuable when the military geologist is dealing with modern environments different from the temperate-climate landscape, which he himself has been trained to regard as the norm. In this connection, we immediately think, of course, of the arctic and tropical environments, many aspects of which have caused the military engineer much trouble in the past. In the tropical Pacific, for example, an extremely widespread soil is friable "lateritic" clay (Figure 1).

When undisturbed it is well drained; when extensively worked without careful moisture control its structure breaks down and it "puddles" and is likely to turn into a morass. An understanding of the processes that formed such a soil, and how they differ from those that formed our temperate soils, leads to the knowledge necessary to insure proper treatment of the soil in construction.

As an example of the use of geologic reasoning in field consultation we may cite the work of James Gilluly of the United States Geological Survey during the Leyte campaign. Gilluly, who was assigned to the Office of the Chief Engineer, Southwest Pacific Area, was asked to develop new sources of road gravel in an area covered by tropical vegetation: a flood plain across which a stream meandered. All convenient sources of gravel in the stream bed had been exhausted; therefore Gilluly decided to search for abandoned stream channels elsewhere on the flood plain, knowing that these must exist and must contain gravel of approximately the same type as that already utilized from the present stream bed. In "prospecting" these abandoned stream meanders which, of course, were covered with vegetation, the area of search was further reduced by using the principle that streams erode on the outside of a meander and deposit on the inside. This reasoning resulted in the location of suitable supplies of gravel buried under vegetation and 2 to 3 feet of soil.

More complex than field consultation is the preparatinn of engineering studies based upon geologic and soils data. The preparation of a study of one of the Pacific islands in 1944, exemplifies the chain of reasoning followed in the preparation of such a report dealing with an area that the authors have never seen.

Information on this island was very scanty in 1944. There were a few brief descriptions in German and Japanese of the geology of the island group as a whole; the only available maps were on a small scale, with form lines instead of contours, and of dubious accuracy; there were no geology or soils maps. During the preparation of the report, there became available a series of low-level oblique aerial photographs (Figures 2 and 3). This set became by far the most productive source of geologic information for the island.

The geologists were, of course, familiar with the general outline of the geologic history of the Western Pacific Area and with the types of rock and soil that would result from such a history. With this background, much in the way of detailed information could be deduced from close study of the aerial photographs. In the northern part of the island, for instance (Figure 2), three raised limestone terraces could be distinguished, as well as a broader, lower terrace only a few yards above sea level (foreground of Figure 2). Knowledge of the volcanic activity that accompanied the building of these islands in the geologic past, as well as scattered descriptions, led to the conclusion that the limestone of the three raised terraces was tuffaceous. The lower, younger terrace was concluded to be more nearly pure limestone. Both types of limestone, it was predicted, would be suitable for use as fill, riprap, road metal, and concrete aggregate. The tuffaceous limestone would be less satisfactory for road surfacing then the "pure" limestone, because it would be more dusty under traffic. Limestone bonds well when used as a surface course and, in its natural state, is porous and drains quickly.

A glance at the aerial photographs shows that the terrace faces are potentially excellent quarry sites.

The residual soils developed on these limestones were estimated to reach a maximum depth of about 5 feet, at the inner margin of the bench in the foreground of Figure 2. Generally, however, the soil would be much shallower than this, and could be easily stripped by dozers, exposing a relatively smooth coral limestone surface for use as an emergency road.

All the facts cited above are obviously useful to the engineer; all were developed from a knowledge of the processes that formed the island.

Figure 3 illustrates two of the soil types defined for the island on the basis of study of the aerial photographs. One of these forms the margin of the island, between the water's edge and the outer edge of the first terrace. It is a coarse-textured soil that has apparently been transported only a short distance. It appears to be well drained and to consist of gravelly sand, sandy loam, or sand.

The soil of the terrace surfaces, on the other hand, is residual: a friable, "lateritic" stony clay or clay loam, such as is mentioned above. These soil types were defined on the basis of knowledge of soils developed elsewhere from similar parent rock and under similar conditons.

Turning for contrast to arctic and subarctic areas, it is again obvious that the military geologist as well as the engineer must cope with physical processes with which he is almost completely unfamiliar, whether he is working on the ground or preparing a research report of the area.

Figure 4 (tundra, northern Alaska) is a scene typical of hundreds of thousands of square miles in the Arctic. Drainage is impeded by perennially frozen ground not far below the surface and, as a result, there is a tendency in many places to build roads on the flanks of ridges, where drainage is somewhat better. This expedient does not avoid all problems either, because of the tendency, in the arctic and subarctic, for surficial materials to move constantly downhill on even the gentlest slope as a result of frost heaving and of the lubrication resulting from the presence in spring and summer of melt water on the surface of the permafrost table, a few feet below the surface of the ground. Study of the processes at work on the constantly changing Arctic landscape has demonstrated that certain landforms are more stable than others (Hopkins and Sigafoos, 1951). Identification in air photographs or otherwise of relatively slow-moving parts of the landscape can be of great help in laying out new routes for road construction.

In the field of military construction, it is not enough for the geologist or soil scientist to develop such pertinent facts as have been briefly quoted above, or even to demonstrate their application to the solution of military construction problems. There still remains the problem of presenting the necessary information succinctly and without extraneous detail. It is the practice in military geology to present as much information as possible on a map or series of maps (Figures 5, 6, and 7), with a very brief accompanying text, on the theory that the user of such reports has no time to spend in close study of a voluminous publication.

The first step in preparation of a military geology report should be the compilation of basic-data maps, such as the geologic and soils maps illustrated in Figures 5 and 6. The information thus presented is then used in the preparation of interpretive maps, of which Figure 7 is an example. It will be noted, of course, that Figure 5 is not a geologic map in the classical sense. It is much simpler: its map units are not formations in the ordinary geologic sense, and they are not defined in terms of geologic age. In preparing a map of this type, map units are defined in terms of common physical properties that have an important effect upon military construction problems; this usually involves combining a number of geologic formations in a single map unit.

The same procedure is followed in preparation of the basic soil map for a military construction study (Figure 6). As in the maps used here as examples, the basic soil map is practically always more complex than the basic geologic map.

Figure 7, showing suitability for airfield construction, is essentially the type of map that would be used for planning road construction and maintenance. Its chief sources were Figures 5 and 6, from which were drawn information concerning soil drainage, slope, and availability of construction materials. The classification reached after consideration of these factors is, in this case, expressed primarily in terms of suitability for airfield construction. In this particular example it happens that the map-unit classification, ranging from "good" to "very poor", also ranges from the areas of best drainage to those of poorest drainage. It is possible that the presence of extremely uneven terrain, or the lack of construction materials in some areas, would necessitate a less orderly progression in terms of drainage. This illustrates a cardinal principle of military geology: the final answer to a problem of military application of geologic information must be stated as simply as possible, but the geologist must remember that this answer can be reached only by analysis of a multiplicity of factors. No single factor will alone give the answer.

In conclusion, it is the responsibility of the military geologist to predict as closely as possible what the ground conditions are, with particular reference to drainage and suitability for stabilization; where construction materials may be found within a suitable radius; the properties of these materials, and the problems that may be met in extracting them. He does these things on the ground as a consultant working closely with the engineer, and also at a great distance from the area of study, as a contributor to intelligence studies or to pre-operational planning.

Due to the inability to reproduce the photographs, they were not included.

DETRIMENTAL MINERALS IN CONCRETE AGGREGATE

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I am honored by the invitation to address this group of engineers on this subject. Your interest in this matter probably arises from your desire to produce low cost pavements - low in original cost but also low in maintenance and repairs. You probably like to think about a truly permanent pavement which would withstand heavy traffic, rain and sun, freezing and thawing, and all of the other things that tend to cause destruction of pavements. Being practical, however, you realize that such ideal construction materials do not exist, so you build pavements or highways of various materials, one of which is concrete.

Some people regard concrete as a lifeless material. I like to think of it as something dynamic, rather than static - something which is continually changing. Hydration proceeds rapidly during the first few weeks but continues at a decelerating rate for months if not years. When destructive forces have once begun, their acceleration can become quite pronounced and a good pavement can become a bad pavement in a surprisingly short time. Autogenous healing, presence of unhydrated clinker grains in very old concretes, and other similar evidences seem to suggest that concrete continues to grow (that is, hydrate) during its entire useful life. Nevertheless, concrete has a limited ability to withstand the ravages of time and part of this limitation is related to the aggregate employed.

I wish it were possible to bring to you a more complete picture of the role of aggregate in the life of concrete. Unfortunately only a portion of the picture has been painted to date. Concrete aggregate has been defined as an inert material, usually sand, gravel, crushed stone, etc., which makes up the bulk of concrete when combined with portland cement and water in suitable proportions. This definition contains a fundamental falacy. In fact if aggregates were strictly inert we would have no occasion for discussing the matter today. Some aggregate materials are highly reactive toward the physical-chemical processes which accompany the hydration of portland cement. On this occasion we can only consider a few typical sources of difficulty in our search for ideal aggregates. Probably no aggregate is completely inert. In fact, if such a completely inert aggregate were to exist, it might be anything but ideal; it might have zero adhesive strength with the cement paste.

We are therefore searching for good aggregates, aggregates that react in favorable ways. The **easiest** approach to this subject is a negative one, i.e. a consideration of some typical bad aggregates in order to see what makes them bad. Facetiously, I might comment that there is another approach to this problem which has been applied with diminishing success in recent years: namely, the assumption that all aggregates are inherently good and any distress which appears in the concrete can be directly attributed to the poor quality of the portland cement, or contamination of the mix water, or the honesty of the contractor, or various other "excuses." Please do not gain the impression that I imply that all diseases of concrete are directly attributable to the aggregate. This is not true and this over-simplification would be as erroneous as blaming all faulty concrete on the cement.

All of you are well aware that sulfate solutions are "bad medicine" for concrete, whether the sulfate comes from chemical wastes, ground water or whatever the source. When I was with the Bureau of Reclamation, we were fabricating mortar bars of various different rocks and minerals in order to obtain data on their behavior with respect to high-alkali cements. I decided to throw in a ringer - so to speak - and requested that alunite ($KAl_3(SO_4)_2(OH)_6$) be used in a set of mortar bars. Alunite is not ordinarily considered a soluble mineral, but what happened to those mortar bars should never happen to any concrete structure. Although the bars containing the high-alkali cement showed severe distress at very early ages, the low-alkali bars soon were so badly cracked and expanded that we made no further measurements. Alunite is not likely to occur in many potential aggregate deposits but I can assure you that a little of this mineral would go a long way in causing the deterioration of the concrete.

Most of you have heard of Stanton's history-making discoveries concerning certain pavements in California. You realize, of course, that this was the original recognition of what is now loosely called alkali-aggregate reaction. More accurately, it should be called "reaction between certain types of aggregate materials and the alkalies of portland cement." Stanton deduced that there was something unusual about the aggregate of the faulty pavements. It was not long before the siliceous constituents were recognized as the source of the difficulty and various organizations became interested in attempting to learn more about reactive aggregates. Because Parker Dam had exhibited some of the same symptoms of disease that had been observed for the California highways, the Bureau of Reclamation became actively interested in learning the causes, symptoms, and cure or prevention of this disease of concrete. I was one of the "doctors" hired by the Bureau in 1941 to make clinical studies of this disease and to conduct research on its cure or prevention. R. F. Blanks, a rather progressive engineer, and C. P. Berkey (Geologist on the Bureau's Board of Consultants) had sold Chief Engineer Harper on the study of concrete deterioration by petrographic methods. In those days I might add that it took a bit of selling.

What we found out about the diseases of concrete is, in general, readily accessible in the literature. These studies were confined essentially to aggregate materials which react with portland cements containing more than 0.60 percent of soda equivalents. A considerable number of minerals and rocks were found to react with these cements, as is indicated in table 1. We did not confine our investigations to empirical tests but attempted to obtain a general

TABLE I. Rocks and Minerals Which are Deleteriously Reactive with High-Alkali Cements

Reactive Mineral	Chemical Composition	Physical Character
Opal	$SiO_2 \cdot n\ H_2O$	Amorphous
Chalcedony	SiO_2	Cryptocrystalline fibrous
Tridymite*	SiO_2	Crystalline

Reactive Rocks -1-	Reactive component
Siliceous Rocks:	
Opaline cherts	Opal
Chalcedonic cherts	Chalcedony
Siliceous limestones	Chalcedony and/or opal
Volcanic rocks**:	
Rhyolites and rhyolite tuffs	Volcanic glass, devitrified glass, and tridymite
Dacites and dacite tuffs	
Andesites and andesite tuffs	
Metamorphic rocks:	
Phyllites	Hydromicas (?)
Miscellaneous rocks:	
Any rocks containing veinlets, inclusions, or grains of the reactive minerals listed above	

* Hornibrook, Insley, and Shuman, 1943, p. 218

-1- Artificial silicate glasses, such as pyrex glass, are known to be reactive

** The volcanic types listed are known to be deleteriously reactive; basalts are known to be innocuous; data regarding trachytes, latites, and phonolites are lacking.

(This is the table which appears on p. 237 of the Engineering Geology (Berkey) Volume published by the Geol. Soc. Amer.)

picture of the physical-chemical conditions which contribute to the destruction of the concrete. In this latter connection, I shall merely state that we were able to demonstrate that the expansion, cracking and decline in strength of the concrete was caused by the attack upon certain aggregate materials by the caustic solutions within the hydrating cement which resulted in the formation of gelatinous materials. These gelatinous materials are capable of imbibing water from the concrete and, in doing so, produce osmotic pressures which exceed the tensile strength of concrete. We were successful in measuring these pressures and were also successful in calculating pressures of the same order of magnitude on the basis of well-established laws of physical chemistry. In addition, a chemical test to ascertain reactivity of aggregates with respect to high-alkali cements was devised by the Bureau's Petrographic Laboratory.

Suppose one has, however, a reliable source of low-alkali cement. What aggregates must one avoid if good concrete is to be assured? I can only give you a few helpful clews on this subject. Some aggregates are apparently capable of contributing alkalies to the hydrating cement. Possibly this was the case with alunite. We did not investigate the mechanism of deterioration caused by alunite because we had too many other irons on the fire. However, a recent Bureau memorandum indicates that analcite can release sodium ions and thereby cause adverse reaction if other deleterious materials are present. Analcite is a member of a mineral family known as Zeolites and its behavior is similar to that of artificial zeolites in treatment of water. Since the cement juice of concrete is saturated with respect to calcium ions, analcite can release sodium ions by base-exchange processes. The question of base exchange in concrete is not new; it may have been responsible for deterioration of cast stone and stucco, as described by Loughlin in 1923.

Another interesting case came to our attention. A dolerite, i.e. a fine-grained basic igneous rock somewhat like basalt, was submitted to the laboratory as a possible aggregate source. Microscopic examination disclosed the presence of about 30 percent of clay, which had formed as the result of hydrothermal alteration of the primary igneous minerals. Wetting and drying tests indicated that this rock was capable of expanding and contracting as much as 0.06 percent and of absorbing and releasing almost 3 percent of water. Under comparable conditions of testing argillaceous (clayey) limestones have been found to show 0.1 percent linear fluctuations. Stratified rocks may show in excess of 5 percent variation in linear expansion depending upon the direction in which the expansion is measured. Expansion is usually greatest when measured perpendicular to the bedding planes.

Sweet and others have discussed the dangers associated with the use of certain weathered cherts and have described some special tests which can be applied to determine their suitability as aggregate. Pyrite and marcasite (FeS_2) can become oxidized, with the formation of sulfuric acid and iron oxides, and thus cause popouts and iron staining. Argillaceous constituents can decrease freezing and thawing resistance. Gypsum can liberate sulfate ions to the cement juices and probably cause formation of calcium sulfo-aluminate with consequent expansion of the concrete.

Coatings on aggregate are another potential source of difficulty. Coatings may consist of silt, clay, gypsum, impure carbonates of lime and magnesia, iron oxides, opal, manganese-containing substances, phosphates, or mixtures. In general, if a vigorous washing treatment will remove coatings, the coatings should be removed. Coatings should be viewed with suspicion until they have been given a clear bill-of-health, because of their possible chemical and/or physical affects on the concrete.

You fully realize that we have not discussed any of the standard tests for aggregate materials. We have not mentioned presence of coal lignite or other types of organic matter. No consideration has been given to so-called soundness tests, tests for porosity, abrasion, etc. In general we have confined our attention to factors which are likely to escape detection by these methods. These methods of testing probably should and probably will be continued until they are replaced by more adequate methods. In addition to these methods, service histories are a valuable aid in attempting to predict the behavior of aggregates that have been in use for several years.

When considering new, untried aggregate sources or when a new type of portland cement is to be used with older aggregates, it is probably advisable to obtain, if possible, a careful petrographic examination. This report should be made by a petrographer who is familiar with the causes of deterioration of concrete. Unfortunately, the number of petrographers who have interested themselves in this field is not large. Nevertheless, it seems appropriate to mention that the late Professor Holden of Virginia Polytechnic Institute did some fine pioneering work of this sort.

In closing I wish to acknowledge the source of most of the results which I have discussed. Except as otherwise noted these results were obtained by my former associates with the Bureau of Reclamation, amongst whom the names of Roger Rhoades and R. C. Mielenz deserve special mention.

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"WHAT DOES THE ENGINEER EXPECT OF THE GEOLOGIST?"

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(Paper delivered at Virginia Department of Highway's symposium on "Geology as Applied to Highway Engineering". Richmond, Virginia, February 16, 1951)

"What does the engineer expect of the geologist?"

This question might be answered with considerable truth by saying that many engineers expect many things of many geologists. Such a reply, however, would of course be more flippant than helpful. There appears to be, nevertheless, a slight tinge of flippancy in the question, with its implied presumption that the engineer has a right to expect something of the geologist, and that the geologist is somehow under obligation to do something for the engineer. If there are any relations of obligation between the two professions, they are certainly mutual, and it would therefore be much more appropriate to ask: "How can the geologist be of assistance to the engineer?" This discussion will attempt to give some reasonable answers to the latter question.

Geology covers vast areas - geographically, physically and chronologically - and to a very considerable extent engineering does likewise, The two professions also have large overlapping areas; many engineers use geology regularly in their own practice and probably most geologists do considerable engineering of one kind or another, Examples which come readily to mind are topographic surveying, foundation work and subsurface exploration, which are extensively engaged in by both civil engineers and geologists. Because of this kinship of activity, and perhaps for other reasons, it is the writer's opinion that civil engineers at least can see eye to eye and have a common viewpoint more nearly with geologists than with any other scientists. This is fortunate with respect to assistance which geologists can render engineers, because to be of much assistance to another it is necessary first to understand his problems and particularly his point of view.

It may be of interest here to consider one characteristic difference between the attitude of the engineer and that of the pure scientist. While definitely interested in scientific theory and knowledge and earnestly desirous of having his constructions soundly based thereon, the engineer is usually concerned primarily with the practical application of such theory and knowledge to the particular business at hand. The scientist, on the other hand, frequently and perhaps usually regards the discovery of scientific fast and the development of scientific theory as ends in themselves, and may be somewhat indifferent as to their practical applications, if any. While this generalization is somewhat faulty, as generalizations usually are, it does at least serve to indicate a real difference in viewpoint between many engineers and many scientists which it will be well to bear in mind in considering cooperation between them.

In order for the geologist to be of assistance to the engineer, therefore, it is necessary first for him to remember that the engineer is seeking a practical, usable and reliable answer to a specific problem. Since there is a great deal in common between the two professions, and since most geologists are fully as much concerned with practical solutions as are engineers, this should offer no particular difficulty to the geologist.

Since the engineer is always working within definite limits of time and money, he is inclined to be impatient of anything which may appear unnecessary and to contribute nothing to the orderly and rapid solution of his problem. This applies not only to over-lengthy and elaborate investigations, but to unnecessarily verbose and complicated reports. The engineer is not favorably impressed with an appearance of great learning, unless it is obviously of practical significance, and he is no more patient with abstract and complex scientific language in connection with his professional problems than he is with abstruse medical jargon when he is trying to learn from a doctor what is wrong with his health. He will be much more impressed with a simple, concise and precise statement which he clearly understands. This does not, of course, mean that the engineer can afford to be ignorant of all geologic facts or unacquainted with basic geologic terminology, but it does mean that the geologist should use highly specialized terms as little as possible and explain them where necessary.

When an engineer seeks help from a geologist, he expects that the geologist will discuss the problem with him until he understands it, and also understands what the engineer needs to know in order to have the answer; that if sufficient information is not available the geologist will say so in order that proper steps may be taken to obtain it; that the geologist will then give the problem careful and conscientious consideration, and report his conclusions in the light of his best judgment; and that the conclusions will be clearly stated and reliable. He expects further that any limitations or qualifications of these conclusions will be pointed out, as he certainly does not wish to be under any misapprehension as to the degree of precision of the geologist's findings. He does not expect the geologist to be perfect or omniscient, nor does he expect him to read the interior of the earth with the same precision that he would read a printed page; on the other hand, he does and should expect the geologist not to express his conclusions with any more precision or confidence than he actually feels, or pretend to any greater degree of knowledge than he actually has. It is evident that failure of the geologist to live up to these reasonable expectations may lead to expensive or diastrous results.

It is the responsibility of the geologist, when called into consultation by the engineer, to give information and conclusions of the character indicated above on matters within his knowledge. It is the responsibility of the engineer, and not of the geologist, to decide what use is to be made of such information and conclusions. If, for example, the matter relates to designing a structure in a region which may experience earthquakes, the engineer expects the geologist to inform him as to the past history of the region in that respect and to advise as to the probable frequency, duration and intensity of earthquakes in the locality. It is then the engineer's responsibility to decide what earthquake forces to design for and how to provide for them, although he may consult with the geologist in regard to these matters.

The foregoing remarks relate primarily to the general type of service which the geologist should give the engineer in order to be of greatest assistance to him. It now seems in order to consider some specific types of problems in which the advice of the geologist can be very helpful to the engineer. The list below represents no attempt to give a complete catalogue of all problems of this character; such a catalogue would be beyond the scope of this discussion and beyond the ability of the writer. Most of the problems listed, and all of the problems discussed, lie within the realm of civil engineering; both because this appears more appropriate to the present occasion and because the writer is more familiar with problems of this nature. Doubtless there are projects in all branches of engineering in which the geologist can be and is of great assistance to the engineer.

All of the matters enumerated below have both geologic and engineering aspects and therefore offer opportunities for effective and valuable cooperation between the two professions:

Explorations for petroleum

Explorations for minerals and ores

Mining

Tunneling

Subsurface exploration for foundation purposes (drilling, geophysical methods, etc.)

Foundation problems of dams

Foundation problems of bridges and buildings

Location of construction materials

Interpretation of aerial photographs for soil mapping purposes

Problems involving cuts and embankments (railroads, highways, levees, etc.)

Drainage problems

Explorations for underground water

Erosion control

Earthquakes and landslides

Soil Mechanics

Problems as diverse in character as those listed above all involve both engineering and geology, and their satisfactory solution demands proper consideration of both of these aspects. Most of these problems usually occur in such fashion that the engineer is primarily responsible for their overall solution, but in working them out he frequently needs all the help the geologist can give him. In the past he has not been fully aware of this fact, and in many instances would have been well advised to seek expert geological advice when he has not done so. This situation is steadily improving and the mutual dependence of the two professions is much better recognized than it was three or four decades ago. It should, however, be pointed out that the knowledge and competence of geologists in dealing with these matters has also increased greatly during that period, particularly in regard to such matters as underground fissures and solution channels. There are past instances for example, in which the geologists' advice and opinion in regard to dam foundations was bad and erroneous.

It is now generally recognized that in the development of such a project as a major dam or tunnel, geological exploration is essential before the exact site can be finally determined and actual design commenced. As the project progresses, the engineer and geologist work hand-in-hand in making and in interpreting results. One of the best presentations of such work which has recently come to the writer's attention is Technical Report No. 22 of the Tennessee Valley Authority, entitled "Geology and Foundation Treatment". This publication is obtainable from the TVA and is well worthy of perusal by anyone interested in cooperative work between engineers and geologists; particular attention is directed to Chapter 3 entitled "Role of Engineering Geologists". Similar attention might well be directed to Chapter V of Professor Legget's excellent book on "Geology and Engineering".

Geology usually plays a larger part in tunnel and dam projects than is the case with bridges and buildings. In brief this may be thought of as being due, in the case of tunnels, to the fact that the entire structure is below the ground so that the geology is evidently of the utmost importance, and in the case of dams to the fact that even minor imperfections far below the dam foundations may be serious, together with the fact that a dam failure may be more disastrous than that of any other type of structure.

The bridge engineer, except in the case of the very largest structures, is generally his own geologist in regard to foundation matters. This results in part from the limitations generally existing as to available funds, and in part from the fact that geological problems as affecting bridge design are usually much simpler than in the case of dams and tunnels. The results of this policy have on the whole been satisfactory; the experienced bridge consultant is usually reasonably competent in the general run of geological matters affecting his own practice, and will ordinarily know when he is confronted with a problem of this nature beyond his depth. One of the most valuable attributes which the engineer can possess is knowledge of when to call for assistance from experts in another field, and this applies with particular force to geological advice on unusually difficult foundation problems. Such a situation, for example, would arise in the case of a bridge which might lie across a geologic fault. Interesting examples of large and difficult bridge piers in which geologic considerations played a major part are the bridges at San Francisco, and the Hell Gate Arch Bridge in New York (see Trans. Am. Soc. Civil Engrs., Vol. 82, 1918). The foundation problems confronting the designer of buildings are somewhat similar to those faced by the bridge engineer, but in general are likely to be less complex.

The geologist is often called on to advise the engineer in the interprotation of borings and other subsurface explorations. As indicated before, it is particularly important here that his classifications and descriptions be readily understandable not only by the engineer but by the contractor. Since the logs of all borings for engineering constructions should be included in the contract drawings, it is essential that they not be subject to misinterpretation. The writer has found it desirable to omit all vague terminology such as "fine" sand, "soft" clay, and the like, since "fine" and "soft" are relative terms. If the samples taken during the borings are available for inspection, as they should be, the size of gradation is evident from inspection, and the degree of firmness, as for example in the case of wash borings, is best described by the number of blows of a given weight falling a given distance required to sink the casing (of known size) a given distance at the elevation of each sample. Core drilling samples of course speak for themselves. Certain ambiguous terms, such as "marl", which does not mean the same thing in all parts of the country, should be precisely defined. Vague terminology may lead to a lawsuit and at best is likely to lead to disputes.

The highway engineer, as well as the railway engineer, can profit greatly from geological assistance. In addition to the matters previously discussed, which are of interest to such engineers, the geologist can be of assistance in various other ways. One important service he can perform is in locating satisfactory sources of sand, gravel and stone for concrete aggregates, and also of soils for subgrade stabilization. His particular knowledge of the geology of an entire region may enable him to suggest undeveloped sources near a particular project which might escape the engineer, or conversely to save time and money by preventing a fruitless search. Such investigations may be more or less piecemeal, or they may be systematic and widespread. Projects of the latter character may involve the construction of complete soil maps of large areas or of an entire state. For instance, the Joint Research Project of Purdue University and the Indiana State Highway Department is carrying on a project of this nature covering the entire state of Indiana, county by county. Most of this work is done by proper interpretation of aerial photographs, supplemented by any necessary field work on the ground. In all phases of such work the services of geologists are practically essential.

In problems of alignment and grade, with the resultant cuts, fills and drainage, the advice of the skilled geologist can also be exceedingly helpful. He may assist in avoiding an alignment involving constant trouble from bad subsurface conditions, poor drainage or similar causes, and may suggest another which will be relatively free of such difficulties. He may also prevent the use of an embankment material of unsuitable or doubtful character. Instances are not uncommon in which considerable stretches of roadway or roadbed have been constant sources of annoyance, expense and perhaps danger, with prohibitively expensive relocation or reconstruction the only real solution, all of which might have been avoided in the first place by a different location or a different embankment material. The value of geologic advice in the making and interpretation of soil studies for a projected line should hence be obvious.

Within the past two decades there has been an enormous increase in engineering knowledge of soils as foundation materials, with the development of a field of investigations usually termed "soil mechanics". It has become possible in many cases to predict actual settlements with considerable accuracy, based largely upon laboratory tests of consolidation and other properties of undisturbed samples taken at the site. While generally considered to be within the field of civil engineering, the problems of soil mechanics present many geologic aspects. Geologists may therefore be of great assistance to those who work in soil mechanics. When settlement is to be expected, the engineer usually wants to know "how much?" He will ordinarily look to the soil mechanics expert for the numerical answer to this question, but in getting it the soils engineer may well seek assistance from the geologist. When rock is encountered, the engineer wants to know how far into it he should carry his exploratory drilling for assurance of safety, and a geologist's answer to this question may be very valuable.

In a symposium held at Knoxville in December, 1950, under the auspices of the Department of Geology and Geography of the University of Tennessee, Mr. Everett Scroggie, Director of Bridges of the Tennessee Valley Authority, presented a paper entitled, "What Does the Engineer Expect of the Geologist?" The writer has found Mr. Scroggie's paper quite helpful in preparing this discussion and should like to make due acknowledgement.

Numerous aspects of the question of how the geologist may assist the engineer have been touched on herein very lightly or not at all. Enough has perhaps been said to indicate that the geologist may be of great help in a great many ways. It may be not inappropriate to suggest that there are ways in which the engineer can be of assistance to the geologist. One thing of this character which engineers can do is to inform geologists of information obtained on engineering projects which may have geological value. For example, some years ago in a neighboring state on the Atlantic Seaboard numerous deep wash borings were made in connection with a bridge project in a location wherein previous borings to such depths had not been made. The State Geologist was informed and was much interested in the logs of the borings and in the samples. Numerous other opportunities for exchange of valuable information doubtless exist and engineers have perhaps been remiss in not calling such matters to the attention of geologists.

It is hoped and expected that cooperation and collaboration between engineers and geologists will continue to increase, for this would be very beneficial to both professions. After all, they are both interested in building upon a sure foundation, both literally and figuratively, and both can earnestly join in the eloquent prayer of the psalmist of old:

"And establish Thou the work of our hands upon us; yea, the work of our hands, establish Thou it".

THE USE OF PLATE BEARING TESTS IN THE THICKNESS DESIGN OF FLEXIBLE PAVEMENTS

by

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(Delivered in Richmond, Virginia, February 16, 1951, at the Symposium on GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING)

In the rational design of flexible pavements, three important factors must be considered. They are

1. Wheel loads and their application,
2. The distribution of the pressure caused by the wheel loads, and
3. The strength of the subgrade and pavement.

Evaluation of the wheel loads is made from truck-weight surveys, or estimated, and the pressures exerted by the wheel loads can be determined from the inflation pressures carried by the tires. It has been found that the pressure exerted by a tire when loaded to approximately its rated capacity is about 10% greater than its inflation pressure due to tire wall stiffness.

Pressure distribution is a very controversial subject as its application is difficult to measure. Some work along this line has been done and some is still in progress. Many designers assume a pressure distribution angle of 45 degrees while others use theoretical treatment for its determination. There does not seem to be a great difference in the thickness obtained by the use of most of the current methods of pressure distribution when applied to the same subgrade bearing value.

In flexible pavement design the strength of the subgrade is usually expressed in terms of its bearing value and unless the expression is qualified, the value is practically worthless. For instance, the value obtained from a test using a 30-inch diameter plate can be much less than the value obtained with a plate 12 inches in diameter, and this value can be much less than the value obtained with a 6-inch plate. Also, the moisture content of the soil at the time of test influences the value tremendously. Since the bearing value of a soil unlike its bearing capacity, is a measure of its strength within certain limits of settlement (provided this value is less than the value for bearing capacity,) it is obvious that the degree of consolidation of the soil will also have a marked effect on its magnitude.

It will be seen from the above that load tests must be carefully made under controlled conditions and that the values obtained correctly interpreted, or else the use of the values in flexible pavement design will produce pavement thicknesses that are erroneous. In this paper the author will describe the conditions under which a load test should be performed, the correction of load test values obtained under other conditions, and a load test procedure that will permit complete analysis using sound engineering principles.

If possible the soil should be tested in a condition that it will assume when serving under the most adverse conditions that actually exist conducive to producing low strength. It has been found that the moisture content of the soil in a subgrade or base fluctuates, although protected by an impervious surfacing and situated above the influence of a ground water table.

The amount of moisture contained in the soil is found to vary with the soil type. An investigation was made by the Soils Laboratory of the North Carolina State Highway and Public Works Commission in 1947 to determine the moisture contents existing in subgrade soils and bases of roads in service during the year, as well as the amount of consolidaton they acquired under traffic. A report of this investigation and study was made at the annual meeting of the Highway Research Board in 1948 and is published in the Proceedings for that year. Table No. 1 gives the moisture contents expressed in terms of the Standard Optimum and degrees of consolidation expressed in terms of the Standard Density to be used when load testing soils belonging to various subgrade groups. The data contained in this table is the result of this investigation and study and are applicable to soils found in North Carolina, serving as subgrades and bases under the conditions existing in that State.

When load tests are made in the field, it is seldom that the conditions recommended in Table No. 1 are found to exist, so the values obtained should be adjusted. The amount of this adjustment may be determined in the laboratory by performing some form of strength test such as shear, unconfined compression, etc., on a sample of the soil in the condition found to exist in the field and comparing this value with the value obtained from the same test made on the soil remolded in the condition desired. It is assumed that these two values bear the same relationship with one another as the two bearing values.

Table No. 1

RECOMMENDED
MOISTURES AND DENSITIES
TO BE USED WHEN LOAD
TESTING SOILS

HRB SUBGRADE GROUP	MOISTURE % STANDARD OPTIMUM	DENSITY % STANDARD DENSITY
A-1	80	100
A-2-4	80	100
A-2-6	105	100
A-3	80	100
A-4	110	97-100
A-5	115	97-100
A-6	110	97-100
A-7-5	120	97-100
A-7-6	110	97-100

It is possible to perform load tests on soil at moisture contents and degrees of consolidation to suit desired conditions. To do this a small section of a road must be constructed of the soil at the proper moisture content and degree of consolidation. The dimensions of the section must be such as to avoid boundary effects or else the bearing values will be influenced by confinement. In highway work the minimum dimensions should be 4 feet wide, 2½ feet deep, and 16 feet long if the section is constructed in a trench or a steel bin. If an embankment is used it should be constructed at least 12 feet wide, 2½ feet deep, and 24 feet long. These dimensions will permit the use of circular plates up to 12 or 13 inches in diameter which is the equivalent diamter of the contact area of a large truck tire. The length of these sections permits four different load tests to be made, a feature to be desired for accurate and dependable work.

The Soils Laboratory of the North Carolina State Highway and Public Works Commission performs load tests on subgrades composed of various soils, and on base courses placed on subgrades. The tests are performed in the laboratory under controlled conditions of moisture and consolidation. The testing rig consists of a steel bin 42 inches wide, 30 inches deep, and 14 feet long and a loading frame consisting of a steel beam supported by two steel columns located near the ends of the bin. Loading is done by means of a 25-ton Black and Decker loadometer attached to the beam. The arrangement is such that the loadometer may be moved anywhere along the beam permitting loads to be applied along the center of the section. Photographs of the rig are shown in Figures 1 and 1-A. This bin provides four test sections, 42 inches square, which are tested using four circular steel plates having diameters of 13, 10, 8, and 6 inches.

The testing technique used follows that developed by Professor Wm. S. Housel of the University of Michigan and has been used by him in his work as a consultant on foundations. He reported this test technique at the International Association for Bridge and Structural Engineering at Munich, Germany, in 1936. The load test is of the static type and consists of applying loads to circular steel plates of different diameters in cumulative increments and observing settlements that occur at definite time intervals. Each load increment is held for one hour before applying another increment, or until all settlement has ceased for a period of 15 minutes. Settlement is measured with micrometer dials reading to .001" at intervals of 5, 20, 35, 50, and 60 minutes. After the expiration of one hour the next increment of load is added to the preceeding one and the procedure of observing and recording settlements repeated. Load increments are chosen so that about 8 are used in the test, four or five of which are less than the load that will cause failure. After the final increment has been applied for one hour and the settlement recorded, the entire load is removed and time allowed for complete recovery of the soil before the dials are again read. The settlement shown by this final reading is the permanent deformation the soil has undergone due to having been loaded, and the difference between this settlement and the total settlement is the amount of recovery.

The data obtained from a load test is used to construct a load-settlement diagram from which the strength characteristics of the soil are determined. A load-settlement diagram is not unlike stress-strain curves used in strength of materials studies, in fact, it is identical. The unit loads are plotted as ordinates and the corresponding settlements for one hour loadings plotted as abscissae. The resulting curve may be used to determine such characteristics as bearing value, bearing capacity, modulus of elasticity, and consolidation due to loading. While the determination of these characteristics may be made graphically from the load-settlement diagram a more accurate method has been developed by Professor Housel.

According to Housel, the supporting power of soil is the result of two stress reactions, perimeter shear and developed pressure, which he designates as m and n, respectively. Perimeter shear is the resistance the column of soil beneath the bearing plate offers to being moved downward by the load and is expressed in units of pressure per unit of length, since the only measurable dimension is its perimeter. Developed pressure is the resistance the soil column offers to being crushed or failed in compression and is expressed in units of pressure per unit of area.

Under this concept, a load, W, applied to a bearing plate of perimeter, P, is supported by the reaction consisting of the Perimeter, P, times the perimeter shear, m, and the reaction consisting of the area, A, times the developed pressure, n.

This can be expressed algebraically as

$$W = Pm + An \quad (1)$$

which may be reduced to an expression in terms of unit load or pressure by dividing through by the contact area, A

$$\frac{W}{A} = p = \frac{P}{A}m + n, \quad (2)$$

the basic equation for bearing capacity.

It will be noticed that equation (2) is a linear equation of the intercept form, with values of p as ordinates, $\frac{P}{A}$ as abscissae, m the slope of the line, and n the intercept, or value of p when $\frac{P}{A}$ is zero. Values of p and $\frac{P}{A}$ are always known, but the values of m and n must be solved. The solution of equations having two variables requires two equations, which in this case will necessitate two load tests conducted with plates of different diameters.

The use of plates of different diameters in load testing soils is essential to accurate and dependable results, and is a requirement if an analytical treatment of the test data is to be attempted. While a minimum of two plates of different diameters is necessary when the two stress reactions, perimeter shear and developed pressure are to be determined, the use of at least three plates of different diameters is much more desirable. Since the load testing work being done in North Carolina is of a research nature, four plates of different diameters are used. By plotting values of p at or near the bearing capacity, and for the same settlement, against $\frac{P}{A}$, the perimeter-area ratios of the plates, a straight line should be obtained. The three plates having values lying nearer a straight line are chosen for making combinations for the solution of m and n by simultaneous equations. When only two plates of different diameters are used, only one combination is possible, but when three plates are used, or chosen from tests made with more than three plates, three combinations are possible and three values of m and n may be determined. An average of these values can be made and used in determining the bearing capacity of the soil for any size of contact area.

Figure No. 2 shows the uncorrected load-settlement diagrams made from the data obtained from load tests performed on the same soil using bearing plates of 6, 9, and 12 inches in diameter. These diagrams must be corrected for dead load and settlement due to initial consolidation and seating of the plates. The determination of this correction may be done graphically as shown in Figure No. 3 which consists of drawing the best straight line between the first two or three points of the diagram and extending the line below zero load to a point equal to the unit dead load. Settlements to the left of the ordinate line are additive and those to the right are subtractive. The correction, then, is the unit dead load, which is added to the unit loads applied, and the settlement which may be plus or minus.

Figure No. 4 shows the load settlement diagrams shown in Figure No. 2 after the data has been corrected. From these corrected diagrams the data for the simultaneous equations may be obtained by selecting values of p for each plate corresponding to small increments of settlement. Settlement increments of 0.05" are used by the author. The values of p and their corresponding perimeter-area ratios, $\frac{P}{A}$, for the same settlement are substituted in formula (2), the basic formula for bearing capacity. Using the values obtained from three plates of different diameters, three equations can be made which can be combined in three pairs. Solving the three pairs of equations, three values of m and n may be obtained. Theoretically these three values should be the same, but it rarely happens that they are, due to the fact that a soil mass is not a perfectly homogeneous material, so an average of these three values of m and n is used. One advantage in using data from load tests made with three or more plates is that an average bearing capacity of the soil is obtained. The values of m and n are plotted against their corresponding settlements as shown in Figure No. 5.

The stress reaction curves do not indicate the bearing capacity of a soil, but the stress reaction values may be used in the determination of two coefficients, one of which will indicate its bearing capacity. The ratio of settlement and its corresponding value of n gives a coefficient of settlement, K_1, and the ratio of m and n gives a coefficient of stress reaction, K_2. The values of these coefficients are plotted against their corresponding settlements to form K_1 and K_2 curves as shown in Figure No. 5. The minimum value of K_1 or the maximum value of K_2 occurs at the "critical settlement" and the corresponding values of m and n are the stress reactions for determining the bearing capacity of the soil. Substitution of these values in the basic formula for bearing capacity, $p = \frac{P}{A} m + n$, will give the bearing capacity of the soil for any size of plate.

The data used in the construction of Figures Nos. 2, 3, 4, and 5 was obtained from an actual test made in the Soils Laboratory of the North Carolina State Highway and Public Works Commission on a heavy clay soil. The soil was obtained from the B horizon of the Cecil Series and contained 19% sand, 12% silt, and 69% clay. The Liquid Limit and Plasticity Index of the soil was 77 and 21, respectively, and its Standard Density and Optimum Moisture were 88.0 pounds per cubic foot and 34%, respectively. The average relative density of the soil when tested was 97.3% and its moisture content was 32.9%.

It will be noted from Figure No. 5 that the minimum value of K_1 indicates the "critical settlement" to occur at a settlement of 0.200" when the values of m and n are 18.1 pounds per lineal inch and 24.1 pounds per square inch, respectively. Substituting these values in the basic formula for bearing capacity, $p = \frac{P}{A} m + n$, we have the following values for bearing capacity for plates having diameters of 12, 9, and 6 inches.

P (12" plate) = $(1/3 \times 18.1) + 24.1 = 30.1$ p.s.i.

P (9" plate) = $(4/9 \times 18.1) + 24.1 = 32.1$ p.s.i.

P (6" plate) = $(2/3 \times 18.1) + 24.1 = 36.2$ p.s.i.

The definition for bearing capacity is that unit load greater than which will produce progressive settlement or plastic flow, while the definition for bearing value is that unit load producing a certain specified settlement, provided, of course, that the settlement specified is not greater than the "critical settlement" in which case the bearing capacity is equal to the bearing value.

Let it be assumed, for illustration, that the <u>bearing value</u> of this soil is desired for a contact diameter of 30 inches and at a settlement of 0.1". From the analysis of the test data the values of m and n at 0.1" settlement are 17.1 pounds per lineal inch and 10.2 pounds per square inch, respectively. Substituting these values in the basic formula for bearing capacity, we have

$$P = (.133 \times 17.1) + 10.2 = 12.5 \text{ p.s.i.}$$

If the value of the modulus, K, used in Westergaard's formula for the thickness of a concrete pavement is desired, it may be calculated, as

$$K = \frac{12.5}{0.1} = 125$$

As stated above the load test was made on this soil at a moisture content of 32.9% which is slightly less than the Standard Optimum, 34%. This test was made before it was found that a soil of this type should be tested at a moisture content equal to 120% of its Standard Optimum. (See Table 1.) This same soil has since been re-tested at a moisture content equal to 120% of the Standard Optimum and its bearing capacity was found to be 25.7 p.s.i. for a 12-inch diameter plate instead of 30.1 p.s.i.

In this paper the author has attempted to stress the importance of load testing soil in a condition that it is to serve, and has pointed out the manner in which this condition can be determined for various types of soils. Details of a reliable test procedure have been given and the analytical treatment of the test data discussed. Although the example given involved the testing of a subgrade soil, the same test technique may be used in testing the strength of a flexible pavement structure placed on a subgrade. In analyzing the data, however, it must be borne in mind that a structure composed of layers of soil, with or without a flexible surface course, is being tested and not a soil. Tests of this type are being conducted in the North Carolina Soils Laboratory and it is strongly indicated at the present time, that the type of granular base course, whether it be of the fine aggregate type or the coarse aggregate type, or of the gravel aggregate type or the crushed aggregate type, is not the governing factor in the strength of the pavement structure within the limits of highway loads, but its thickness is. This work has not reached the stage where it can be reported, but it is hoped that a report can be made in the not too distant future. In this work the rational design for pavement thickness is also being investigated and the results at this stage, although not conclusive, are very promising in that the calculated values of the supporting power of the pavement structure agrees fairly well with the values obtained by test.

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CONTROL OF GROUND WATER IN CONSOLIDATED ROCKS[1/]

By G. .D. De Buchananne[2/]

Abstract

(Presented by G. D. De Buchananne at the Symposium on Geology as Applied to Highway Engineering, sponsored by the Virginia Department of Highways, Richmond, Va., February 16, 1951).

Subsurface water, either in the zone of saturation or in the zone of aeration above the water table, often affects the construction and maintenance of highways. Consolidated rocks are frequently the source of such water, which moves into the residual overburden or into a man-made fill and thereby creates problems.

The solution of problems due to ground water requires adequate information on the occurrence and movement of the water and a means of controlling its movement either in the aquifers or after it has been discharged. These methods of control can be divided into two general types. One is by exclusion, where the water is prevented from entering the critical area. The other is by removal, where the water is removed from the danger spot.

Each problem is unique and requires that the engineer adopt one or several methods to obtain a solution. However, if the occurrence and movement of ground water at the site is understood, the problem of its control is easier to solve.

1/ Publication authorized by the Director, U.S. Geological Survey.

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Identification of Rock Types

Presented by D. O. Woolf, Senior Materials Engineer, Bureau of Public Roads
At Symposium on Geology as Applied to Highway Engineering
Richmond, Virginia, February 16, 1951

The method for the identification of types of rock presented here is based primarily on the needs of the engineer. By experience he has learned that a given type of rock has certain properties which may or may not be suitable for a construction project under consideration. If the engineer can identify the rock in question he will be in a better position to judge the suitability of the rock for the intended use. This method is intended for use in the field, and requires very little apparatus. Although the method involves the problem of multiple choice, the selections to be made are so distinctive that there should be little difficulty in the identification of most types of rock.

In the use of this method, the rock is first placed in one of five general groups:

I. Glassy, wholly or partly.
II. Not glassy; dull or stony; homogeneous; so fine-grained that the grains can not be recognized.
III. Distinctly granular.
IV. Distinctly foliated; no effervescence with acid.
V. Clearly fragmental in composition.

Each group is subdivided into subgroups or types of rock on the basis of simple physical or chemical determinations. Glassy rocks in Group I are identified as quartz, obsidian, or pumice depending on their physical characterisics.

A fine-grained rock in Group II is examined for its hardness with respect to steel. If it is easily scratched, it is tested with acid. Rock which has a brisk effervescence with cold acid is identifed as limestone. If the effervescence is brisk only when the rock is powdered or the acid heated, the rock is considered a magnesium limestone or dolomite. Other types of soft, fine grained rock include shale and serpentimite, and these are identified by their physical characteristics.

The fine-grained rocks which are not scratched readily by steel include felsite, chert, and basalt. The color, relative hardness, type of fracture, and structure of a given sample help to identify these types. The luster or appearance of the rock by reflected light is of great assistance in the identification of chert.

If the rock is granular, it is placed in one of three subgroups depending first, on its hardness and second, on the uniformity of size of the rock grains. If the rock is easily scratched with the knife, it is treated with acid for identification as crystalline limestone, marble, or dolomitic marble. If the rock is harder than steel, and is composed of grains of approximately the same size, the type of rock is named by determination of its color and its mineral constituents. A rock composed essentially of quartz and feldspar, and generally of light color, is identified as granite. If the rock is light colored and composed essentially of feldspar with very little quartz, it probably is syenite. A rock of medium color, containing feldspar and a dark ferromagnesian mineral but with feldspar in excess, would be termed a diorite. If the amount of ferromagnesian minerals is equal to or in excess of the feldspar content, the rock will have a dark color and will be named a diabase if the grains are just large enough to see, or a gabbro if the rock is coarse-grained. A granular rock composed essentially of quartz is either sandstone or quartzite, and can be named by examination of a broken surface of the rock. If the fracture passes around the grains, the rock is sandstone, but if an appreciable number of broken grains is found, the rock probably is quartzite.

The third subgroup of granular rock is composed of rocks which have large, distinct crystals in a fine-grained groundmass. A rock of this type is called a porphyry. Many rocks of this type are similar in mineral composition to those previously mentioned in which the grains are of about the same size.

In the foliated group of rock, identification is based on the fineness of grain and the degree of perfection of the foliation. All of these rocks will break more or less readily along one plane. A rock which is very finely grained and splits readily into thin slabs will be recognized as slate. If the rock has a coarser grain and a less perfect foliation, it probably is some variety of schist. Rock which is still coarser and which breaks with very imperfect surfaces is named as gneiss. Separation between gneiss and schist may also be made by examination of the rock for feldspar. If this mineral is found in any abundance, the rock should be identified as gneiss.

In the last group, the rocks are composed of rounded or angular fragments cemented together with any one of a number of materials. If the fragments are rounded, the rock is termed a conglomerate, but if the fragments are angular, the rock is named as breccia. Sandstone may also be found in this group, and is identified by its appearance and high quartz content.

The user of this method is expected to be able to identify quartz and feldspar as constituents of granular rocks. Both of these minerals may be light in color and have nearly the same hardness. Among the most definite means of separating them may be mentioned the following:

Quartz has a glassy luster and is frequently transluscent while feldspar looks more like porcelain and is opaque.

Feldspar cleaves into fragments with an angle of about 90° between the sides. These cleavage faces may readily be found by examination of the surface of the rock in good light. Quartz has no cleavage faces.

In the cooling of molten lava, feldspar crystallizes before quartz. Hence feldspar is more likely to develop in crystal form while quartz occurs in shapeless masses.

In the use of this method, some difficulty has been found in the identification of the intrusive or coarse-grained igneous rocks. Possibly this is due to too much emphasis on the feldspar content of these rocks and too little on the other essential mineral constituents. If petrographic analyses of intrusive rocks are studied, granite will be found to be the only rock with an appreciable amount of quartz. Gabbro will be found to contain the greatest amount of the dark ferromagnesian minerals (mica, hornblende, and augite) with diorite containing the next greatest amount. Consequently if the rock is hard, with visible interlocking grains of approximately equal size, and contains an appreciable amount of quartz, it probably is a granite. On the other hand, if the rock has the same characteristics as given above, but contains little if any quartz, it can be named by reference to the ferromagnesian content as indicated by the color of the rock. A rock of light color may be a syenite, one of medium color a diorite, and one of dark color a gabbro.

Attention should be given to the transition of rock by insensible stages from one kind to another. Granite will grade into syenite, and syenite into diorite, for instance; or a coarse-grained rock will grade into a fine-grained rock of the same mineralogical composition. Consequently the identification of the hand specimen by the method described here does have some limitations. However, a careful study of the specimen following the method outlined, should permit the user to name the rock with a relative small margin of error.

PRELIMINARY CLASSIFICATION

Group I - Glassy, wholly or partly
Group II - Not glassy; dull or stony; homogeneous; so fine-grained that grains can not be recognized
Group III - Distinctly granular
Group IV - Distinctly foliated; no effervescence with acid
Group V - Clearly fragmental in composition

GROUP I - GLASSY ROCKS

1. Glassy luster; hard; conchoidal fracture; colorless, white, smoky gray QUARTZ
2. Solid glass;brilliant vitreous luster; generally black. OBSIDIAN
3. Cellular or frothy glass. PUMICE

GROUP II - VERY FINE-GRAINED ROCKS

Subgroup IIA - Softer than steel

1. Grains imperceptible; clay odor; laminated; no effervescence. SHALE
2. Brisk effervescence with cold acid, LIMESTONE
3. Brisk effervescence if acid is heated. DOLOMITE
4. Soapy feel; thin edges translucent; green to black. SERPENTINE

Subgroup II B - Harder than steel

1. Light to gray color; scratched by quartz, FELSITE
2. Very hard; conchoidal fracture; waxy luster. CHERT
   If dark gray to black, FLINT.
3. Heavy; dark color. BASALT

GROUP III - GRANULAR ROCKS

Subgroup III A - Softer than steel

1. Brisk effervescence with cold acid. LIMESTONE OR MARBLE
2. Brisk effervescence if acid is heated. DOLOMITIC MARBLE

Subgroup III B - Harder than steel; grains of approximately same size

1. Mainly quartz and feldspar; usually light colored. GRANITE
2. Mainly feldspar; little quartz; light colored. SYENITE
3. Feldspar and ferromagnesian (dark) minerals
   (a) Mainly feldspar; medium color. DIORITE
   (b) Ferromagnesian minerals predominant; dark color
      (I) Grains just visible to unaided eye. DIABASE
      (II)Coarse-grained. GABBRO
4. Mainly quartz
   (a) Fracture around grains. SANDSTONE
   (b) Fracture through grains. QUARTZITE

Subgroup III C - Harder than steel; large distinct crystals in a fine grained groundmass

Group of rocks similar in mineral composition to a number of rocks in Subgroup III B; called PORPHYRY due to their structure

GROUP IV - FOLIATED ROCKS

1. Medium to coarse grain; roughly foliated. GNEISS
2. Finer grained and foliated. SCHIST
3. Very fine grain; splits easily into thin slabs. SLATE

GROUP V - FRAGMENTAL ROCKS

1. Pebbles embedded in a cementing medium. CONGLOMERATE
2. Angular fragments embedded in a cementing medium. BRECCIA
3. Quartz grains, rounded or angular, cemented together. SANDSTONE

THE CONSTRUCTION OF HIGHWAY BRDIGES AND SEPARATION STRUCTURES IN UNCONSOLIDATED SEDIMENTS

Many of you know of the speaker's limitations in the field of foundation engineering. My practical experience has been restricted to design and construction, and I must therefore approach the subject of structures on unconsolidated sediments from the viewpoint of a designer or constructor. Like others of similar age and experience, my formal education covered a period when soils mechanics courses were being given in American colleges for the first time; and, even though I completed one of those early courses, it was a most inadequate foundation on which to build, especially when its content is compared with the instruction now available in undergraduate and graduate courses on the subject. However, in common with my contemporaries I have been forced to expand my knowledge of the behavior of soils under load by reading and by association with men engaged primarily in soils work; and I can assure you that it is a great satisfaction to the designer and constructor to know that there are many men available today who qualify as experts. The designing engineer who does not rely upon their knowledge of the theory of soil mechanics and skills in field and laboratory techniques is passing up an opportunity for improving his designs and saving his employer money.

A lazy engineer is fortunate if he can work in Virginia, particularly in central Virginia. In my own case, being fundamentally lazy, I frequently find myself in the offices of others. If the problem is one of stream flow, possible scour or runoff, Mr. Donald Wallace of the Water Resources Board is sure to be interrupted in prompt execution of his multiple tasks; or, if the problem is one of ground water, the regular duties of Messrs. McGill and Sinnott in the Geological Survey are sure to suffer. Questions in historical or structural geology lead to impositions on Messrs. Nelson, Roberts and Edmundson of the faculty of the School of Geology; while near my doorstep are Messrs. Shelburne and Stevens of the Virginia Council of Highway Investigation and Research who are always gracious in advising on problems in soil mechanics. Even in my own field, structural engineering, I rely on Messrs. Glidden and Clary of the Virginia Department of Highways for the solution of problems which are too difficult for me. Virginia is fortunate to have within her borders not only these men upon whom I rely, but many others of equal caliber engaged in the practice of geology and civil engineering.

The best remembered experiences are likely to be early ones, and my first experience with structures on unconsolidated sediments is one I shall not forget. During the years of 1927 through 1930 I was employed by the Chesapeake and Ohio Railway on the construction of the present bridge across the Ohio River between Covington, Kentucky, and Cincinnati, Ohio. The project included the elevation of the tracks through downtown Covington and the elimination of perhaps a dozen grade crossings. Foundations for the approaches to the big bridge were located in the flood plain of the river and were supported on precast concrete piling, but the grade separation structures were built on a sandy soil which contained considerable clay or silt. This soil gave good support as long as it was confined, and prior to the experience I intend to relate no foundation difficulties had been encountered except under the wing of one abutment. In this instance a pipe line relocation was excavated below and too near the wall foundation with the result that the soil flowed out from under the wall. However, a successful underpinning plan was devised and the trouble was corrected at small expense.

Later my first assignment as an inspector was the construction of piers and abutments for a long span plate girder bridge which carries street traffic over the tracks in the Covington yards. On the day the foundation for the pier at one end of the plate girder span was poured, I had been told that I could go to the race track at Latonia during the afternoon if the footing was completed in time. Since in those days contractor's forces worked a six-day week, during which the length of the work day depended upon when the forms were filled, and Sunday was the day the engineering crew worked on the monthly estimates, it was quite a privilege to have half a day off to see the horses run. At any rate, I like to think that my decision on the adequacy of the pier foundation was a hurried one, for as soon as the soil looked reasonably firm I allowed the footing to be poured. In subsequent months the piers were completed, the superstructure erected, and the floor slab poured. Fortunately in the course of these operations a bench mark was set on the bridge seat of the pier, for a definite dip in the surface of the floor and some cracking of the slab adjacent to the pier developed. These faults were attribtted to poor workmanship by the contractor until someone tied a line of levels into the bench mark. As best I remember, the circuit failed to close by more than four inches and it was then that we realized the pier was settling. An immediate investigation was made, and it was discovered that I had built the pier over an 18-inch vitrified clay sewer which had crushed under the load of the pier, saturating my firm foundation and affording a perfect channel for the material to flow out from beneath the pier. As soon as the line was relocated and the old pipe was packed with grout, the subsidence was arrested and today the bridge is in good shape. Perhaps it is well to add that I was not fired, but on the other hand the bosses' remarks about my judgment of foundations were not complimentary and since that time I have made it an inviolate rule not to build heavy structures on unconsolidated sediments that are underlain by city sewers. And it was also during those weeks that I resolved to spend more time with the publications of Terzaghi, Krynine, and Casagrande and less with the Daily Racing Form. Some of you know what happened to that resolution.

The designing engineer who is called upon to plan a structure on unconsolidated sediments should first assemble all the data available on the construction and service records of nearby structures which are likely to rest upon similar soils. Highway and railroad engineers can usually find some information in the files, and although it is necessary to supplement these data with an actual exploration of the proposed site the record of an existing structure is of immense value. Experience is of paramount importance to the foundation engineer, and although I completely disagree with the frequently heard remark that there are "no good young foundation engineers," I would certainly distrust any engineer who disregarded the experience of those who had actually built structures in the area.

The recommendation of securing data on nearby structures is not intended to deemphasize the importance of examining the actual site of the proposed structure. Such an examination is necessary if only to verify the fact that the same conditions exist at the new site as at the old. It does not take a lot of experience with foundations to know that extreme differences in soil strata and bed rock occur in small areas. Within the past few years I visited a railroad job in southwestern Virginia on which one abutment of a bridge across a two-lane highway rested on rock, while the other abutment, which was not more than fifty feet distant, was built on piles about twenty feet long. Another similar instance occurred (in Grundy, Virginia, I think) while I worked for Mr. Glidden. There plans had been prepared and the work had been started on a rigid frame bridge of about 100 ft. span before it was discovered that one leg had to be built on piles, whereas the other one set on solid rock as had been originally planned. I have no exact figures on the cost of these unexpected changes, but it may have been considerably more than the cost of obtaining adequate foundation data before the designs were prepared. These and similar experiences elsewhere have given me a keen interest in the work of Mr. Parrott, Mr. Woodson, and other members of the Department of Highways staff; and I know it must give the Bridge Division great satisfaction to work with good foundation data.

As soon as the designing engineer has secured all available data on structures in the neighborhood of the proposed construction, he should, if possible, visit the site in the company of a geologist and soils engineer. Discussions of the geology of the site, particularly the history of the formation and the soil strata that are likely to be encountered, are always more enlightening when the terrain is under observation. The opinions of the geologist and soils engineer are much clearer to the designer when he can actually see the features of the site which influence their judgment. Also, while at the site they can probably decide upon the extent of the preliminary investigation which is required.

The designing engineer must be given a more active part in site explorations for structures on unconsolidated sediments than is necessary when the foundation is rock or well consolidated soil. Soils engineers and geologists who have worked with designers know that they approach their task with the following questions:

1. How deep is bed rock?
2. If bed rock can't be reached, can piles be driven through the unconsolidated materials to firm soil?
3. How deep is the first strata of soil on which a spread footing can be built?
4. If both types of foundations can be used, which will be cheaper--piling or a spread footing?

These questions can probably be answered after the first few test borings have been completed, but it is not until then that a definite program of investigation can be scheduled. It is important that all persons concerned with the foundation investigation recognize from the beginning that the foundation engineer and designer must be given a free hand in directing the work and specifying the depth of the borings and the number and location of the samples obtained. It must also be recognized that if the foundation is in unconsolidated sediments the engineer cannot estimate the extent of investigational work which will be required until a considerable amount of preliminary work has been completed. It is a matter of fact that there are many sediments on which foundations cannot be constructed. An extreme example of this type is the mud and silt which covers the bottoms of the streams in Tidewater Virginia. Another case in point is some of the sediments which overlie the dolomite and limestone formations in the Shenandoah Valley. A recent experience there convinces me that even the lightest of structures will suffer destructive differential settlements if the elevation of the ground water fluctuates due to natural or artificial causes. This condition is not evident in excavations above the water table; in fact, the soil is frequently classified as sandy clay containing some gravel and looks good. However, samples from the underlying strata immediately reveal the soft clay which causes the settlements. This is only one of many illustrations which could be used to show that the foundation engineer must have a thorough knowledge of the soil strata at an appreciable depth below the foundation. Clarence W. Dunham, in his book Foundations of Structures,* estimates that the minimum average unit pressures to consider when estimating probable settlements are:

Deep soft clay	300 psf.
Deep stiff clay	500 psf.
Deep silt and very find sand	500 psf.

The foregoing quotation of selected values from a table on page 62 of the book indicates that the depth of a bore hole should be determined by consideration of the foundation pressure, the probable distribution of pressure through the soil mass, and the type of soil encountered. When the foundation exploration is begun it is not likely that any of these three variables can be accurately estimated, consequently the designer and foundation engineer should be free under the contract to specify the extent of the work.

On the other hand, the method of performing both field and laboratory work should be carefully specified. The accuracy of the data obtained from foundation investigations depends upon the knowledge and skill of the field and laboratory men engaged in the work to a greater extent than most of the testing required by structural engineers. Ignorance, lack of skill and carelessness in the field or laboratory will, of course, result in misleading data. Those of you who, like myself, have only been exposed to the usual undergraduate laboratory course

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*McGraw-Hill Book Co., Inc., New York, 1950.

in soil mechanics probably share in my feeling of amazement at the consistency of results that skilled technicians obtain from the very tests which gave us so much trouble and which, for me, never gave the same results when repeated.

Recently Mr. J. C. Stevens of the Virginia Highway Department was kind enough to review the field and laboratory work that has been done on the proposed bridge over the Rappahannock River at Gray's Point. A brief outline of this work may give some idea of the procedures followed in the foundation investigations for a highway structure on unconsolidated sediments. The field work consisted of soundings to determine the depth of the water and the sinking of more than twenty test holes. Many of these holes penetrated to a depth more than 150 feet below sea level, and several approached or exceeded the 200-foot level. Spoon samples were taken at frequent intervals from all holes and used by the field men for the visual classification of the soils and in a number of instances were also tested in the laboratory. In addition, a number of tube or "undistrubed" samples were secured and sent to the laboratory. The field forces also recorded the number of hammer blows required to drive the sampling tube 12 inches as is usual in all soil investigations. Laboratory tests included grain size determination, consolidation (0 to 16 ksf), liquid limit, moisture determination (per cent of dry weight), unconfined compression test, and triaxial compression with 2 ksf lateral stress test, and others. Time does not permit either a discussion of the tests or the results; however, those of you who are engaged in soils work will be interested to know that settlement computations made from the results of consolidation tests were in good agreement with estimates based on liquid limits and calculated by means of the formulas for compression index given on page 66 of Soil Mechanics in Engineering Practice* by Karl Terzaghi and Ralph B. Peck.

In preparing the plans for a structure, the designer should keep the needs of the field forces in mind. The contractor must have foundation data to prepare a bid just as the structural planners require it for their designs. All too often the very information which would enable the contractors to bid lower is buried in the designer's files. Furthermore, the designer should make available to the men responsible for the execution of his plans as much of the data on which his work has been based as possible. In no other way can there be complete coordination of effort.

There is one fact that my small experience in foundation investigation has taught me. I pass it on for what it may be worth. When directing exploration work on which your final design is to be based, decide upon the minimum requirements as soon as the preliminary investigation permits. List then the least number of test holes and samples you believe to be necessary and as the work progresses expand your original plan if advisable, but never reduce it. We engineers are natural born money savers; a good part of our training is devoted to methods

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*John Wiley and Sons, Inc., New York, 1948 of doing work more cheaply, and our handbooks contain many "economy tables." As a result, when the data from several of the first holes sunk confirm our original opinions, we are inclined to try to save money by skipping a hole or spacing them farther apart or decreasing the depth. My advice is to resist this inclination. I have yielded to it more than once, and I can honestly say that at some time in the course of design or construction I have learned to my sorrow that the money I tried to save would have served my client or employer better if it had been spent in obtaining additional foundation data.

I've enjoyed being with you and have benefited from the splendid papers that have been presented. I trust this one gave you the rest that we all need towards the end of a well spent day.

Third Annual
Southeastern Symposium on

"GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING"

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SYMPOSIUM ON GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING

Virginia Military Institute

Lexington, Virginia - February 29, 1952

Application of Geology to Highway Work as Practiced by the States, A Questionnaire Summary - Ernest Dobrovolny, Assistant Chief, Engineering Geology Branch, United States Geological Survey, Denver, Colorado

Studies of Aggregates on Okinawa - Allen H. Nicol, Military Geology Unit, United States Geological Survey, Washington, D. C.

Problem Minerals in Concrete Aggregates of the Southeastern States - Robert H. Nesbitt, Office of the Chief of Engineers, Department of the Army, Washington, D. C.

Geology and Highway Engineering - A Continuing Mission - Rex S. Anderson, District Engineer, United States Bureau of Public Roads, Richmond, Virginia

The Design of the Slopes of Highway Rock Excavations in West Virginia - R. F. Baker, Engineer of Soil Mechanics, State Road Commission of West Virginia, Charleston, West Virginia

The Geology of a Highway Slide at Gatlinburg, Tennessee- Jarvis B. Hadley, Geologist, United States Geological Survey, Gatlinburg, Tennessee

The Application of Geology to Bridge Foundations - Everett Scroggie, Bridge Design Engineer, Tennessee Valley Authority, Knoxville, Tennessee

Geological Problems in the Design and Construction of Highways in Virginia - W. T. Parrott, Engineering Geologist, Virginia Department of Highways, Richmond, Virginia

APPLICATION OF GEOLOGY TO HIGHWAY WORK AS PRACTISED

BY THE STATES, A QUESTIONNAIRE SUMMARY

(Abstract)

Delivered at the Third Annual Symposium on Geology As Applied To Highway Engineering

Compiled by S. E. Horner, Chief Geologist, State Highway Commission of Kansas, and E. Dobrovolny, Geologist, United States Geological Survey.

Presented by

E. Dobrovolny

Increase in the use of geology in highway work is demonstrated by the results of two questionnaires submitted to all state highway departments; one questionnaire was circulated in 1949 and another in 1951. Replies were received from all states. In 1949 sixteen states employed a total of 43 geologists on their staffs; in 1951 twenty-three states employed 89 full time staff geologists.

In order of their most frequent mention in the replies to the questionnaire, geologists are employed in the following:

1. Materials Surveys
2. Foundation Studies for Bridges
3. Soils and Soil Mechanics
4. Investigation of Sub-drainage Problems
5. Analysis of Aggregates
6. Classification of Excavation
7. Research
8. Surface Hydrology Studies

Fields of expansion are considered to be primarily in the same areas of work in which the geologists are currently active.

The most important fields of research, in the opinion of the people who answered the questionnaire, are:

1. Sub-drainage
2. Foundations
3. Clay Minerals and Soil Performance
4. Analysis of Aggregates
5. Materials Survey Techniques
6. Landslides

ENGINEERING GEOLOGY ON THE ISLAND OF OKINAWA

Delivered at the Third Annual Symposium on Geology As Applied to Highway Engineering

by

Allen H. Nicol, United States Geological Survey

The purpose of this paper is to describe some of the problems and procedures involved in preparing an engineering geology map and report on Okinawa and to summarize a few applications of geology to engineering problems there.

Engineering Geology forms only one part of the Series of Military Geology Folios now being prepared on various Pacific Islands already mapped by geologists of the U. S. Geological Survey for the Corps of Engineers, U. S. Army. When completed, this series will contain sections on basic geology, engineering geology, basic soils, soil engineering, ground water, surface water, and trafficability.

The map accompanying the Engineering Geology is a special-purpose map showing both surficial deposits and bedrock, based entirely on field geologic mapping, which shows units derived by grouping or splitting certain geologic formations on the basis of engineering properties and use rather than on basis of stratigraphy or geologic age. Such a map is intended for use in preliminary and general investigations at specific sites.

The Engineering Geology Map of Okinawa, on a scale of 1:100,000, has 13 map units and covers the entire island, an area of 485 square miles. These units are beach deposits, alluvium, residual clay, clayey granular

*Published by permission of Director, U. S. Geological Survey.

material, limey granular material, coralline rubble, fine sand, compact gray clay, crystalline limestone, greenstone, platy foliated rocks, sandstone, and porphyry dike rock.

These materials are widely distributed on Okinawa. In hilly to mountainous northern Okinawa the principal bedrock units consist of platy foliated rocks, sandstone, crystalline limestone, and greenstone. The highest points range from 1450 to 1500 feet above sea level. In central Okinawa and the Motobu Peninsula the predominating bedrock units consist of platy foliated rocks, sandstone, clayey and limey granular materials, crystalline limestone, and greenstone. In southern Okinawa, where most of the military installations are located, the principal bedrock units consist of unconsolidated clayey and limey granular materials, coralline rubble, fine sand, and compact gray clay. The maximum relief is 750 feet. Most of the pits and quarries that supplied base course and fill for airfields and roads are located in limey granular material and coralline rubble.

I have stated that the 13 units shown on the Engineering Geology Map are the result of grouping some geologic formations together and of splitting others to derive new units on the basis of engineering properties and use. In order to do this, each geologic map unit or formation requires a thorough appraisal in the field. On Okinawa the field reconnaissance was made by an engineering geologist and field geologists. The party chief and field geologists, because of their considerable knowledge of the entire island, were in a favored position to suggest the best sites or locations within each geologic unit where typical or representative material could be examined, as well as the most expeditious way of reaching these sites. Whenever possible, exposures were studied in road cuts, borrow pits, quarries, cliffs, and canyons. Sample sites were tentatively located and decided upon during this phase of the work. Detailed observations were made at each exposure, and notes were taken as to the thickness of the material in the unit, texture, composition, bedding, jointing, condition of weathering, amount of overburden, topographic expression, method and facility of excavation, fragmentation and crushing characteristics, suitability for tunneling, underground storage, and road construction, pit and quarry exploitation, slope stability, drainage, and erosion conditions. A tentative field appraisal was also made of the suitability of the material for use as concrete aggregates, heavy riprap, light riprap, rubble masonry, granular material for stabilized base, embankment, fill and subgrade, and binder. Construction materials appraisals were later correlated with the results of engineering tests for the report.

As a result of the field reconnaissance and observations made at representative locations, it became possible for the geologists to combine or split the geologic formations into engineering geology units. A great many difficulties attend this operation. The governing considerations include the state of consolidation, the facility of excavation, and the suitability for use as construction material.

The field work involved sampling each of the engineering geology units for laboratory testing. The total number of samples taken was limited by the funds available for testing; hence only the most widespread units and the units of greatest engineering importance were considered. Variations in the material were taken into account, and each sample was evaluated in relation to the unit as a whole. Ordinarily only those samples believed to represent the dominant material in each unit were taken for testing, but some units required additional samples to show the range in engineering properties caused by weathering, by changes in composition, and by changes in texture. The engineering geologist recommended the site selection, supervised the sampling, decided the amount of material to be taken, and determined what engineering tests were to be made. Locations of all sample sites and of important pits and quarries are shown on the map.

The final phase of the work was mainly office detail and consisted of two parts: (a) the geologists compiled the engineering geology map from the basic geologic map, and (b) the engineering geologists integrated and evaluated the data from the testing laboratory, correlated them with the field data, and prepared the report.

The map and report are intended to supply the engineer with generalized geologic data to a maximum depth of 150 feet below the surface, which probably represents a zone of adequate depth for most engineering work on Okinawa except for ground water and deep underground storage; and structural conditions at a depth greater than 150 feet are given in a report on Geology. Detailed engineering data strictly applicable to soil within 10 or 12 feet of the surface are given on Soil Engineering Maps, scale 1:50,000, published in 1951 by the Engineer Section, GHQ, FEC.

In connection with the utilization of any special-purpose map based on geology, detailed field investigations are always required at speific sites. Operations of engineers on Okinawa in the past have demonstrated a need for geological advice on the location and exploitation of new supplies of rock suitable for concrete aggregates, on quarrying problems, and on foundation problems at airfields, bridge sites, and other building sites. The Engineering Geology Map delimits those areas where rock of various types occurs and thus materially aids the engineer in prospecting for new quarry sites. It is also useful in providing general subsurface and bedrock data prior to and during drilling investigations.

A problem on which geological advice was frequently requested by engineers on Okinawa concerned recovery of aggregates for concrete from coralline limestone deposits. Our military forces during and after the war developed many quarries on Okinawa. A few quarries were developed from native or Japanese workings; but most of the quarries were newly developed for the recovery of base course for roads and airfields in southern Okinawa. The expansion of military installations after the war created a demand for aggregates for Portland cement concrete. Only a few of the base course deposits supplied aggregates, and these rarely complied with rigid quality and durability requirements prevailing in the United States. Subsurface exploratory investigations by core borings, beginning sometime in 1949, were utilized to locate compact rock; but owing to the very irregular distribution of weakly cemented rock and clayey weathered material, it was necessary to place the borings on such close centers that the cost was almost prohibitive. Widely spaced borings did not intercept the soft, rubbly and clayey zones, clay pockets, and irregular-shaped masses of other material unsatisfactory for aggregate. Materials at many quarries were extremely heterogeneous and their occurrence was unpredictable; plastic clay overburden penetrated the openings and fractures of the underlying rock to depths of 20 or 30 feet. Most of the rubbly limestones showed a surface cementation or "case hardening" condition due to exposure. This surface cementation often gave an impression of hardness and durability that was sooner or later disproved by operations at the site. All the coralline limestones on Okinawa are more porous than crystalline limestone, and the innumerable cavities and openings are commonly filled with clay. These conditions, and the inexperience of the drillers in working with coralline material, make core recovery by drilling generally very low. Examination of drill cores generally showed compact but porous rock, which was relatively clay-free, as most of the clay was flushed out during drilling. This situation often led to an erroneous assumption by the operators that most of the rock in place would have properties similar to those seen in the cores. A careful study of the cores, of drilling operations, and of rock faces exposed after blasting readily demonstrated the fact that most of the core lost during drilling represented porous, friable rock of an undesirable quality for aggregate. It also proved that the compact but porous rock recovered contained too much clay in the cavities for removal by normal plant processing. The verification of these conditions provided the engineers with a sound basis for eventually abandoning many quarries and for planning the exploration and development of a supply of aggregate in crystalline limestone of uniform quality in another part of the island.

A very serious problem confronting the engineers at airfields is that resulting from the presence of cavities, caverns, and openings both in residual soil and in the underlying coralline limestone formations. Initial attempts to locate concealed cavities in residual soil by borings were both unsuccessful and expensive. A complete geological evaluation of this problem was requested at one of the major airfields. This included preparation of a detailed geological map of the area on a scale of 1:4,800, subsurface investigation by borings in a few selected areas in order to reveal better the subsurface conditions, a thorough consideration of drainage conditions, and geophysical exploration to locate the caverns and cavities.

Geological investigation and study of the Ryukyu formation, which predominates throughout most of the area, showed it to be very porous, rubbly, and pervious limestone. Residual soil overlying the limestone ranges from a few feet to 40 feet or more in thickness and contains a fairly high percentage of clay, silt, and sand. The extremely irregular surface of the bedrock was found to be too complex -- with innumerable closely spaced pinnacles and potholes -- to show in cross section. Mapping of several surface collapses in the residual soil delimited a few zones of underground channels beneath the runway, but borings failed to locate any concealed caverns. As studies continued, cavities in the residual soil appeared to be caused by a disturbance of original drainage through recent construction. This activity accelerated the undermining and sapping action of water flowing through and enlarging channels and openings in the limestone underlying the residual soil. The process proceeded upward to the surface by gradual enlargement of channels and caverns until roof support gave way. This belief was later substantiated by excavation and grading in the soil that exposed new cavities a few feet below the ground surface.

This cause of the collapse in residual soil emphasized the need for steps to control adverse drainage conditions. Two other problems remained, however; first, locating the cavities in the residual soil before construction; and second, treating the cavities, once they were located.

Seismic methods were employed in an attempt to locate the cavities in the residual soil at very shallow depth, but they were not successful. This failure was due in part to the fact that uncontrollable vibration in the immediate vicinity of the tests caused by airplanes landing and taking off, by the movement of heavy trucks and equipment, and by blasting at infrequent intervals at a nearby quarry seriously interferred with the interpretation of the seismograph records. Later experimentation and research with a sonic method for locating cavities in residual soil demonstrated that this method was successful in the volcanic ash soil of Japan. The equipment, however, needed modification to make it portable and operative under general field conditions.

Experimental geophysical work was conducted at a different location on Okinawa, with a seismic fan shooting method to locate a large cavern in limestone bedrock. Initial interpretations of the seismic measurements did not correlate well with known field conditions. The problem undertaken was an exceedingly difficult one, and it is not surprising that the first more or less superficial examination of geophysical results did not correspond with known facts. However, a new method of interpretation of the basic seismic data was devised, and results shown in a second report based on and derived from the original data proved that the method was surprisingly successful in locating the cavern and establishing its depth.

Thus geological investigation was used to determine the cause of the cavities, to propose steps to avert their formation, and to devise a method of locating them. The final decision as to how to treat the cavities is in the province of the engineer.

PROBLEM MINERALS IN CONCRETE AGGREGATES OF THE SOUTHEASTERN UNITED STATES

Delivered at the Third Annual Symposium on Geology As Applied to Highway Engineering

by

R. H. Nesbitt, Office of the Chief of Engineers

Abstract

In the past few years, procedures adopted for improving the durability of concrete have focussed attention on the properties of river aggregates and quarry rock which make these materials acceptable or useless as ingredients of concrete. Keeping pace with, if not exceeding, the advancements in techniques for mixing, placing, and curing concrete, is the vast amount of research conducted by federal, state, and private agencies in an effort to determine the physical and chemical properties of aggregates which make some of these materials satisfactory, others unsatisfactory. The paper discusses the extent to which petrographic analyses may be used in determining the suitability of materials, and some of the more important characteristics of aggregates in Southeastern United States which contribute to deterioration of concrete, are described and illustrated.

GEOLOGY AND HIGHWAY ENGINEERING, A CONTINUING MISSION

Delivered at the Third Annual Symposium on Geology As Applied To Highway Engineering

by

Rex S. Anderson, District Engineer, Bureau of Public Roads

There is no question of the need for and growing trend toward closer bond between geologists and engineers. This third annual symposium is testimony of that trend. At each meeting we have explored many fields of joint activity. I would like to renew and channel our thinking by mentioning, without elaboration, some of the problems which can be solved best by the joint efforts of the geologist and highway engineer. This list is taken from a paper by Mr. Marshall T. Huntting, found in the December, 1943, Proceedings of the American Society of Civil Engineers.

1. Locating suitable road-surfacing material pits and quarries
2. Determining the suitability of various earth materials for surfacing, concrete construction, and other highway uses
3. Subgrade treatment and classification
4. Frost-heave problems
5. Predicting the character of material to be excavated
6. Preventing and correcting landslides
7. Appraising the competency of materials for bridge foundations
8. Judging the desirability of bridge sites and road sites with respect to possible changes of stream channels
9. Investigating proposed tunnel sites
10. Investigating and predicting subsurface water conditions and locating underground water supplies

11. Evaluating mineral lands to be bought or condemned for highway location, and

12. Acting as witness in litigation.

To an engineer, geophysics is closely related to the science of geology. Probably most engineers in the room would be vague in stating the difference between the two. My Webster's Collegiate says that one is concerned with agencies which modify the earth, and that the other treats with the history of the earth and its life, especially as recorded in its rocks. This kinship should permit me to mention, even to an audience dominated by geologists, a few of the services and helps that the highway engineer can expect from the geophysicist.

There are two geophysical tests of particular interest to the highway engineer. The first, and most limited in application, is the refraction seismic method. The second is the earth-resistivity test. Both were described and discussed in some detail at our 1950 symposium in Richmond and more recently at the Annual Highway Research Board Meeting in Washington last December, by Mr. R. Woodward Moore. I am indebted to Mr. Moore for his suggestions and examples of the use of these tests.

The seismic test uses the velocity of sound transmission through various kinds of materials to determine the character and depth of a given geological formation. This method is efficient in locating rock formations of local extent, as for a bridge foundation, tunnel, or dam. It is not recommended in areas where the geological formations are made up of alternating beds of shale and limestone or sandstone, but it can locate solid rock beneath talus and boulder formations that are very troublesome in drilling operations. The type of seismic equipment commonly used for highway work costs about $4,000 to $5,000.

The earth-resistivity test requires equipment which can be purchased for about $800. This test is faster and has wider application than the seismic method. It consists of passing a direct current of electricity through the ground and measuring the resistence to current flow. Since different engineering materials may possess similar resistivities, it is essential that calibration readings be taken in each general area before surveys are made.

The principal applications of these tests, and a few examples of each use, may be summarized briefly as follows:

1. Classification of earth work for slope design: In twelve working days, with an inexperienced crew, Mr. Moore explored about 22 miles of proposed roadway in the rugged Ozark Mountains of Arkansas. These tests led to proper slope design for earth and rock cuts, and identified the sections where self-loading scrapers could be employed to move large quantities of earth. The tests also located cuts that would require the use of explosives and power shovels. The value and accuracy of this survey have been confirmed on two grading projects already completed.

2. Foundations for bridges: Tests made in New Hampshire, Georgia, Kansas, and at the Memorial Bridge in Washington have given accurate determinations of the depth of solid rock when compared with results obtained by drilling.

3. Investigations for tunnel sites: Five tunnel sites and several miles of grading along the Blue Ridge Parkway in western North Carolina were investigated in about four days' time.

4. Study of subsurface conditions in swampy areas, slide areas,

and soil-boulder or glacial-till formations where drilling is very troublesome.

5. Location of materials of construction: Possibly this is the most unique application of those mentioned. A resistivity survey on a three-mile section of the new Baltimore-Washington Parkway produced more than the 85,000 to 90,000 cubic yards of granular sub-base material that was needed between the pavement and the underlying clay soil. The first step in this survey was to study geological maps, aerial photographs, and records of drill holes and borings which had been made within the area. An existing geological map showed scattered remnants of plateau gravels, laid down by ancient river action. Aerial photographs helped identify well drained soils in the area. A resistivity contour map was plotted from four test traverses and was used to locate large quantities of suitable granular material, even though the presence of this material had not been disclosed by a boring test made prior to the resistivity survey.

Leaving the general field of geology and geophysics as applied to highway engineering, I would like to offer a few comments of a more personal nature - at least within my personal experience - regarding the geologist-engineer teamwork within the Virginia Department of Highways. The Engineering Geologist, Mr. Parrott, and his small staff are doing and excellent job of spreading themselves over the State, bringing in the answers from the field, and giving the engineers the benefit of their findings. We, in Public Roads, are very much pleased to find a sheet entitled "Engineering Geology" in the contract plans for almost every project that includes a bridge of any size. The geological information influences the selection of the proper types of structures and assures the maximum economy and stability in our bridges. All of us can think of examples in the past where a little more preliminary geological data would have resulted in radical changes of design and elimination of costly change orders.

One of the responsibilities and specialties of a bridge engineer is to appraise and evaluate the safe bearing capacities of foundation materials. Here the geologist and the "Engineering Geology" sheets are especially helpful. However, most bridge designers and highway engineers are not geologists and are sometimes puzzled by geological terms for materials that are quite familiar in the field. In this connection, I have a suggestion that might make geological data more useful and more used. For example, the following terms, taken from "Engineering Geology" sheets, may mystify some engineers:

Catoctin Green Stone

Wissahickon Schist

Aporhyolite

Beekmontown Dolomite

Augen Gneiss

Whitesburg Limestone of the Blount Group

Schistose

Grey Green Mica Chlorite Schist

Terra Rosa

That last one, "Terra Rosa," sounds like red clay to me. While the geologist undoubtedly will solve this problem by inviting the engineer to undertake a program of self-education, it is my suggestion that the geologist might add a short description of the materials, including the probable weathering properties when exposed, show the safe bearing values, and indicate whether or not the material could be penetrated with timber, concrete, or steel piles. We are happy to note that comments relative to safe bearing capacities have been shown in the State's 1951 "Geological Yearbook," and we anticipate that the "Engineering Geology" sheets will carry that information in the future.

This observation, which has general application in geologist-engineer relationships, is offered as a constructive suggestion and is intended to detract in no way from the fine job the Engineering Geologist is doing in Virginia.

Last December I attended a two-day conference of highway engineers of the Central Atlantic States, held in Harrisburg, Pennsylvania. One of the subjects discussed at that meeting was the question, "Should a Geology Section be Made a Part of a Highway Department?" From a discussion of this question, it was learned that the Ohio Department of Highways employs a full-time geologist who is attached to the Design Section. He is used particularly in connection with the design of earth slopes, the prevention and correction of slides, and the determination of pavement and base thicknesses. This arrangement is somewhat different from that in Virginia, where the Engineering Geologist is attached to the Division of Tests and functions with a great deal of freedom and independence, particularly in conducting field investigations throughout the State. The Pennsylvania Department of Highways has started hiring geologists to work in their District Offices. This will place them very definitely in the field. It is my understanding that Pennsylvania plans to use the earth resistivity method of subsurface exploration where this method is adaptable to field conditions. These are the only highway departments in the Central Atlantic group that now employ geologists. Other highway departments engage geologists on a consulting basis for specific problems. However, I believe the trend is toward the establishment of a separate Geology Section as an organic part of the highway department. We, in Virginia, think this is an excellent arrangement and a forward-looking step toward the better solution of many of our highway problems.

For those of us who may be inclined to look back at our past accomplishments and rest on our oars, taking pride in the vast progress which has been made in the field of highway design and construction, let us consider for a moment just where we are going from here - what the future will ask of us. I propose to show that we have really just started and that our future tasks will be much more pressing and complex than those in the past. For this closing part of my discussion, I shall draw heavily from information collected by our Highway Transport Research Branch and presented recently to a meeting of the National Safety Congress by Mr. E. H. Holmes. Mr. Holmes' subject was "Traffic to Come."

During the past thirty years, the level of our annual rate of travel in the United States has grown from the small "luxury" category of the early twenties to the present phenomenal rate of 500 billion vehicle miles. Five hundred billion is half of a trillion. Traffic is putting a new word into our vocabulary.

Percentage-wise, the greatest growth in traffic volumes occurred in the early twenties, but in absolute terms the present rate of increase is very much greater. Between 1920 and 1923, traffic volumes practically doubled, increasing from 45 to 85 billion vehicle miles. From 1947 to 1950, the three-year increase of only 23 percent represented 85 billion vehicle miles. What about the future?

Engineers are habitually conservative, and we have not deviated from this pattern in our past predictions of traffic growth. All have fallen far short of the goals which have actually been reached. Recent studies have shown a striking parallel between traffic and economy. Looking back over the past twenty years, we can show that traffic and the nation's economy have moved steadily, one with the other, except for the period influenced by the artificial restrictions of the war. This striking parallel is not coincidence. It is due to the fact that highway transportation has an integral place in our national life. We are absolutely dependent on the motor vehicle to get us to work, for our shopping, many other social and business trips, and for the spending of our leisure time, particularly over longer weekends and during vacations. Similarly, we depend on the motor truck and its well established and growing part in our national economy. Figures show that ton-mileage by truck increased from 1949 to 1950, almost unbelievably, by one-third.

We have every reason to believe that traffic volumes will continue to grow along with the national economy. This has been estimated to mean an average growth of about four percent per year, a rate that would cause present volumes to double in about 18 years. Whether the four percent figure is too low or too high is not of great consequence for even a modest percentage increase in travel continued over a few years will see us submerged in a flood of traffic. Mr. Holmes gives us this summary of "Traffic to Come": "It will come, and it will come in such a flood that without the increasing and persevering application of our greatest skills we shall be overwhelmed. The future offers no room for complacency; it offers only a challenge."

Highway engineers and the geologists who are working with us are servants of traffic. This is <u>our</u> challenge and the reason why we have a continuing mission.

THE DESIGN OF THE SLOPE OF HIGHWAY ROCK EXCAVATIONS IN WEST VIRGINIA

Delivered at the Third Annual Symposium on Geology As Applied to Highway Engineering

by

R. F. Baker, Engineer of Soil Mechanics

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Introduction

One of the most troublesome problems in the design of highways in West Virginia is the proper slope to be used in Rock excavations. The problem is confused by several factors. One of these is the wide variety of sedimentary rocks that are present. The formations range in strength from the indurated clays of the Croston Red Shales of the Dunkard Series to the hard limestones of the Greenbrier formation and the equally hard Berea Sandstone of the Portage Series.

Slope design is also difficult due to the trend toward deeper cuts in order to obtain higher types of grade and line. Past experience offers less and less assistance as more and more remote sections of the state are improved by the new highways.

West Virginia like all other states must satisfy the tug-of-war between the construction and maintenance costs. If construction costs were no problem it would be simple to design a road that was maintenance free insofar as the rock cuts are concerned. The ideal solution, however, is to cut construction costs until there is no more excavation than necessary to achieve a condition requiring no maintenance.

Finally, adequate slope design will require a certain amount of field work to identify the type of rock that will be encountered. Existing equipment for drilling ranges from the expensive core-drill to power augers or churn-drills. It is possible that geophysical surveys will produce a more economical approach. This latter method is very inexpensive if proper correlation is possible. The high cost of drilling minimizes the number of applications since an over-design may be acheived less expensively than drilling.

The following report covers the West Virginia problem of slope design in rock cuts, and outlines the procedures that are followed. No effort is made to discuss the ever-present problems of landslides. Rather, a study is made of the specific problem of designing highways so that maintenance costs due to rock weathering are held to a minimum.

The literature on the subject is practically non-existent. The writer knows of no direct reference to the slope design of rock cuts other than handbook values for the general classification of various types of rock. These values are not satisfactory except as a very general guide to highway problems.

The methods described in the following are not original in principle. West Virginia has been using a similar type of design for at least twenty years. Other states have made casual reference to a similar type design. The reasoning and specific nature of the solution described was developed by the Department of Soil Mechanics of the West Virginia State Road Commission. In addition to Highway Engineering, principles of Geology and Agronomy have aided in arriving at the present status.

Lest there be a misunderstanding, there is considerable evaluation remaining before a completely satisfactory approach will be available. There is a system suggested, however, that should ultimately lead to a type of design that will strike the proper balance between construction and maintenance costs.

Principles of Slope Design in Rock Cuts

Initially, it would be well to summarize some of the basic principles that are involved in arriving at the proper design of slopes in rock excavation. These principles are essential to a highway problem but may not be applicable to other types of problems.

1. The primary purpose of a good design is to eliminate or minimize maintenance costs due to the weathering of exposed bed-rock. One should remember that if this purpose is not fundamental, there is no problem to designing slopes. The debris from the exposed face tends to (1) clog ditches that result in pavement failures, (2) block shoulders that lead to a more dangerous and less useable highway, and (3) produce rock-falls onto the pavement proper, leading to dangerous conditions for drivers and vehicles.

2. The constructed slope must be as steep as possible in order to keep construction costs at a minimum. This second requirement combined with the first forms the bracket for the design problem. One factor that influences the design is the relative costs of removing material by contract versus that of maintenance. In West Virginia the cost per cubic yard of excavation by contract ranges from 1/3 to 1/4 that required for maintenance forces to remove the debris. The figure will vary from State to State, but contract work will be less expensive due to the larger quantities involved with contract work, and the more localized nature of the earth moving.

3. The proper design of slopes is directly related to the physical characteristics of the bed-rock. While there can be little argument with this statement, there is considerable question as to what measureable characteristic of the bed-rock is most suitable for correlation

with the slope-design problem. In addition, the effects of climate, type of blasting, and erosion are related to the proper slope design. It is the opinion of the writer that there is no reliable technique for determining the ultimate slope of an exposed bed-rock, unless there is an exposure of the same material in the same area and the exposure has been open for a period practically equivalent to the design life of the proposed grade.

Principles of Existing Types of Design

There are three principle types of design currently in use. These are: (1) a uniform slope from ditch-line to the top of the slope, (2) a slope consisting of straight sections of varying angles depending on the type of bedrock, and (3) straight slopes connected by near-horizontal benches. The three types will be discussed individually.

1. Uniform slope. The principle problem with a uniform slope is the establishment of the proper angle. This will be related to the height due to the erosion problem. In addition, certain types of weak materials appear to have a critical height for a given slope. Thirdly, for the stronger bed-rock materials, there is a difficult construction problem involved with obtaining a slope of 1/2:1 or 3/4:1 (Horizontal: Vertical). Finally, in interbedded materials, a uniform slope must obviously result in an improper design for some of the layers. This may or may not be serious. For the majority of the inumerable small cuts less than twenty feet in height (vertically), a uniform slope will generally be the best solution.

2. Varying angles. The most serious problem with this solution is the determination of the proper angle for the various materials. However, with varying angles the erosion problem may be reduced. In

addition, it becomes possible to get the theoretically proper slope for various layers in an interbedded material.

3. Benches. This type solution is based on the assumption that a certain amount of weathering is inevitable in most freshly exposed bed-rock faces. Furthermore, due to lack of design criteria it offers the best opportunity for establishing a proper balance between maintenance and construction costs. The design is based on three variables: (1) width of bench, (2) vertical height between benches, and (3) the slope angle between the benches. The method is not so susceptible to difficulties if an accurate estimate of the weathering characteristics of the bed-rock is not possible. For shales and other similar rock, the erosion problem with a bench design is not so great due to the reduction of velocity of water moving down the sloping exposures. Generally, the construction is simpler with benches than with a uniform slope, since more nearly vertical slopes are possible. Finally, for some materials the slopes between benches should be steeper than the ultimate, so that the inevitable weathering will not result in more material having been moved than was necessary. The location of the benches is related to the type of bed-rock encountered, but controlled primarily by the permissible height of the slope that will produce debris directly into the ditchline.

Methods Employed in West Virginia

Currently in West Virginia, both field and office studies are made. Conditions vary depending upon which stage of the planning the work is accomplished. For Primary Roads, slopes are designed with only the centerline grade and typical cross-section given. For Secondary Roads, the plans have generally progressed further, and a preliminary slope indicated.

Field Study. Considering a given section of excavation, every effort is made to determine the (1) depth of soil over-burden, and (2) the thickness and type of various bed-rock layers for a depth five feet below the ditch-line. Occasionally, specific consideration is necessary as to whether the area is an old landslide. Ordinarily, landslides in West Virginia are quite easily recognized. In order to obtain a guide as to the ultimate slope, all nearby existing exposures of bed-rock are studied, and if applicable, the types considered certain to be present in the cut are located as to elevation. The slopes of these exposures are measured, also. The histories of the exposures are determined if possible.

Where no reliable exposures are available, auger or core drilling is used to delineate the various layers. Due to the high cost of drilling and due to the number of holes required for reliable design values, use will be made of electrical resistivity in the near future.

Office Study. The field data is plotted on the cross-section of the area. For cut sections with a vertical height of less than twenty feet, a uniform slope is used. This slope will vary approximately as follows (Horizontal: Vertical):

Indurated Clay - 3:1

Weak Shale - 1:1

Medium Shale - 3/4:1

Strong Shale - 1/2:1

Weak Sandstone or Limestone - 1/2:1

Strong Sandstone or Limestone - 1/4:1

In the event an exposure is available as a guide, the ultimate slope is used rather than the above.

For cut sections with a vertical height greater than 20 feet consideration is given to benching the cut. In the range of vertical heights of 20 to 30 feet, benches may or may not be used, depending upon the weathering characteristics of the rock. For the more resistant types, benches are not used.

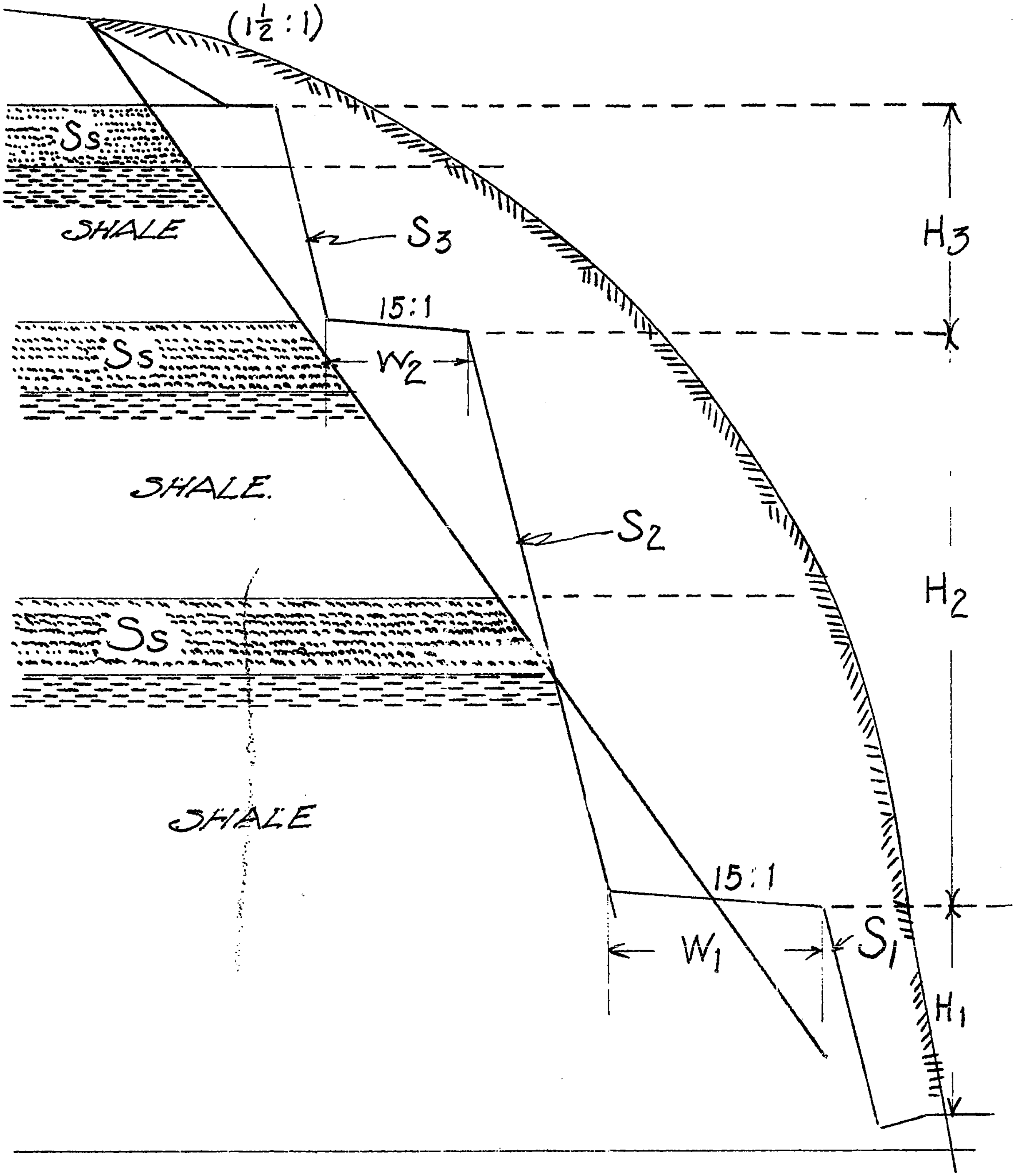
It is the author's opinion that bed-rock of the type encountered in West Virginia should always be benched when (1) the height of vertical cut is greater than 30 feet, (2) the height is between 20 and 30 feet and the rock is not highly resistant to weathering, and (3) the lack of knowledge of the ultimate weathering characteristics prevents an adequate slope design without benches.

As indicated in the section concerning the principles of design, slope designs using benches have three variables: (1) the width of the benches, (2) the vertical height between benches, and (3) the angle of the slope between benches. It will also be recalled that the design is based on the principle that the excavation should be the minimum required so as to avoid more than the minimum maintenance. In Fig. 1, there is a sketch showing the variables and their designation. In Table 1, there is a summary of the guide for values for the various types of bed-rock common in West Virginia. It is to be emphasized that the values in Table 1 are general and are not to be used in place of known values of the ultimate angles of weathering.

The use of benches should be more economical than either a uniform slope or one with varying angles. This will be true because the slopes should be designed steeper than the ultimate angle, and the weathered material will serve as an "insulator" and reduce the amount of weathering. The width of the benches will be somewhat controlled by construction

MAJOR ROCK EXCAVATIONS

— FOR VERTICAL HEIGHTS GREATER THAN 25'



ADDITIONAL BENCHES SHOULD BE USED IF THE VERTICAL HEIGHT OF CUT IS SUCH THAT TOLERANCES WOULD BE EXCEEDED

Fig. 1

methods. Benches that will materially increase the cost of excavation change the economic picture. However, the steeper slopes will be easier for the contractor to construct, and might reflect itself in lower excavation costs.

One principle needs re-emphasis. In West Virginia the benches have been considered as "clean-off" areas; i.e., periodically equipment would remove the debris, making room for additional debris. It is true that benches do permit such a procedure, and cleaning the bench will be necessary when the weathering characteristics have been under-estimated. However, to produce a maintenance free condition, the slope should be disturbed as little as possible, and ultimately should be seeded. If not seeded, weeds and brush will produce some cover and reduce the erosion problems. The factors involved are shown in Fig. 2.

As a final point in the discussion of the methods used in West Virginia, the direction of the slope of the bench is towards the ditchline. Most engineers prefer that the slope be towards the back of the bench. There are advantages and applications for each of the two. These are summarized below:

Sloping toward ditchline		Sloping toward rear of bench	
Advantages	Disadvantages	Advantages	Disadvantages
1. No water trapped	1. Tendency for rock to slide down contact	1. Resists sliding of rock debris	1. Water may be trapped
2. Less tendency for sliding of clay-debris	2. Increases erosion below bench	2. Minimizes erosion of slopes below bench	2. Greater chance for sliding of clay debris

PRINCIPLES OF BENCHING ROCK CUTS

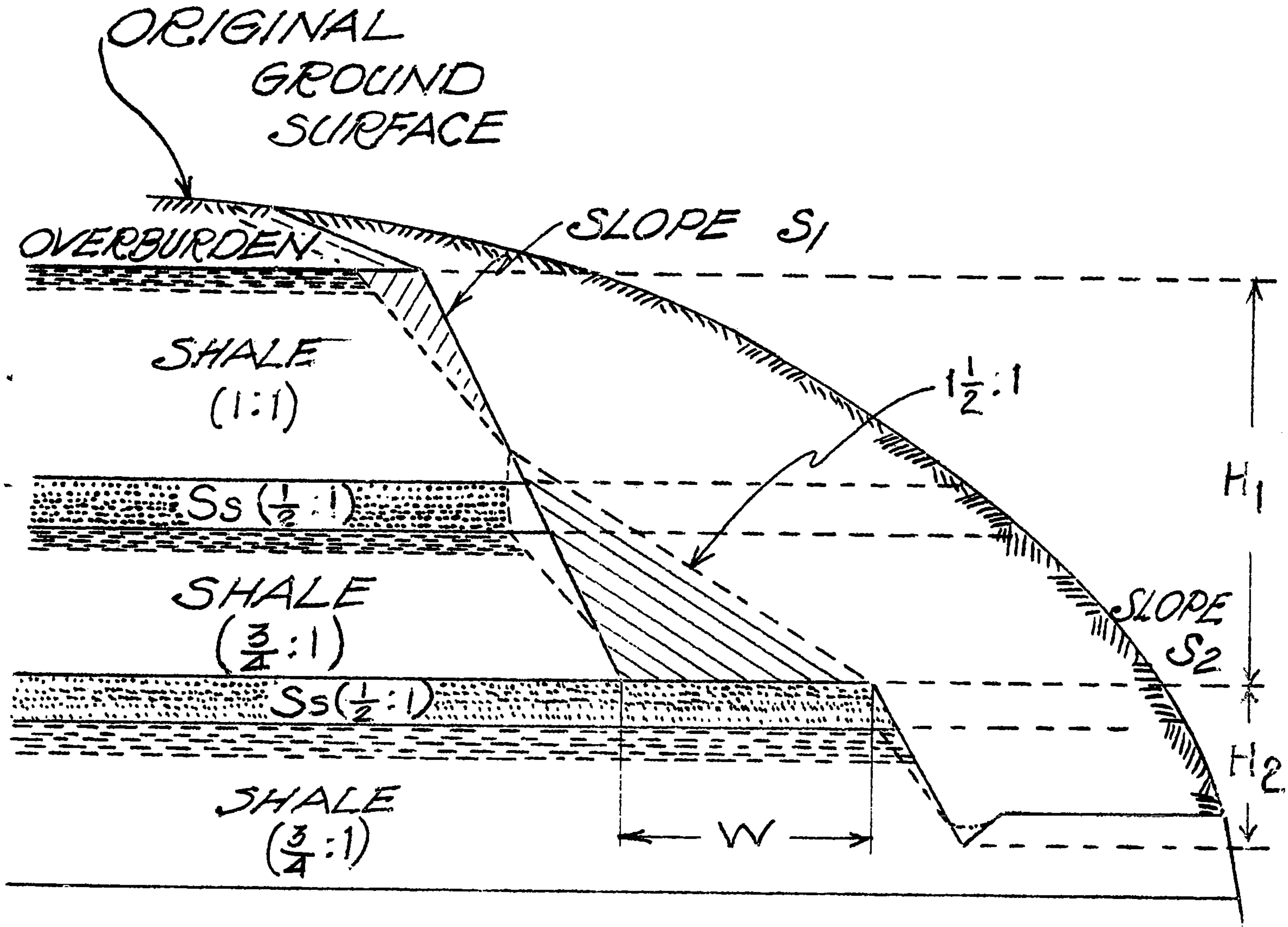


Fig. 2

Thus, it would appear that if little or no clay is to be present in the debris that is caught on the bench, the bench should be sloped toward the rear. However, since very few of the cuts in West Virginia are of this nature, the general design is to slope the cuts toward the ditchline.

Application of Geology

Thus far, little or nothing has been said concerning the place of geology in this problem. There is definite evidence of the presence of geologic principles in the foregoing discussion. In the author's opinion, there is a direct and definite use of geology in these designs. The weakest point in the slope design problem lies in the lack of information on the ultimate slope that a given bed-rock will develop after having been excavated to a given slope. It is the author's opinion that a geologic approach offers the best opportunity for solving the problem.

This is based on the fact that much of the bed-rock in West Virginia has been mapped and classified by geologic age. Thus, a geologist by careful observation and classification of excavated bed-rock could ultimately produce a table of values for a given formation within areas of similar weather conditions. Such an approach would be entirely empirical, and could best be accomplished through geology. For an Engineer to accomplish such a survey without reference to the geologic classification, a long and ardous task should be in prospect.

Another more direct, more generally applicable and more rapidly produced method would be to combine geology with either geologic or engineering tests. Such a study would involve observation, classification, and sampling of bed-rock exposures. The original slope of the excavation, the number of years of exposure, and the general nature of the climate would have to be estimated. In addition to the older exposures, new excavations should be catalogued, observed, and measured. During the ensuing years, routine observations will produce valuable data on the weathering characteristics. Such an approach, to be more rapidly available for use, will require the assumption that a given geologic formation will give essentially the same performance regardless of its location. Eventually, the refinements of differences in the formations as well as climate can be applied.

As a further guide, the bed-rock could be tested. Which test or tests would be applicable is difficult to say at the outset. Certainly an accellerated weathering test would appear desirable. The physical characteristics of the bed-rocks are important. In sedimentary materials, grain size of the sediment and the type of cementing agent is important. Much of this information can best be analyzed by a geologic approach.

In West Virginia, a semi-rational design of the slope of rock excavations is just being initiated. The empirical methods used in the past have proven the value of benching the bed-rocks in West Virginia. The problem lies primarily in the new areas being entered and in the greater depths of cut that are being used. As an adjunct to the routine work of the Department, a long range study of the weathering characteristics of bed-rock is under way. No direct nor rapidly developing study will be possible due to the shortage of personnel and the requirements of routine design problems. The work will be directly supervised by a geologist on the Department staff.

Summary and Conclusions

In summary, the following statements are considered worthy of final emphasis:

1. The problem of the slope design in rock excavations has increased in importance in West Virginia and elsewhere due to the locationing in new areas and due to the application of high geometric standards and consequently deeper excavations. It is becoming increasingly more important as the maintenance costs continue to assume a larger and larger percentage of the funds available to highway agencies.

2. Adequate slope design must be the balance between the lowest construction costs commensurate with little or no future maintenance. The design can afford to require slightly more excavation than would appear absolutely necessary, since maintenance costs are understandably higher than construction costs.

3. The proper design of a slope in rock is directly related to the weathering characteristics of the bed-rock, and the weathering characteristics are undoubtedly related to the geologic formation of the bed-rock and the climate to which the bed-rock is exposed.

4. There are three principle types of slope design in rock excavations for highways:

a. A uniform slope from the ditchline to the base of the overburden.

b. A cut with varying slope angles for the various types of bed-rock encountered.

c. A cut consisting of near-horizontal benches connected by straight slopes.

5. The use of the benching method is preferred in West Virginia for the following reasons:

a. The method is more economical, since the design calls for slopes steeper than the ultimate, and the debris that collects on the bench protects some of the bed-rock from continued weathering.

b. The steeper the slopes, the easier the construction particularly if the material must be removed by blasting.

c. Maintenance costs will be lower in the event the design has been inadequate, due to the fact that benches permit the removal of greater quantities of material by the maintenance forces at any given time.

d. Knowledge of the ultimate weathered slope for a given bed-rock is not sufficient to permit a good design using either of the other two methods.

6. The principle obstacle to a rational slope design is the lack of knowledge of the behavior of a given bed-rock. Geologic principles offer the best approach to the urgently needed information.

TABLE I - VARIABLES IN SLOPE DESIGN OF ROCK CUTS

(All values subject to change if ultimate angle of weathering is known)

	Height between Benches (Feet)		Width of Benches (Feet)		Angle of Slopes between Benches (Horizontal: Vertical)	
Type of Rock	H_1	H_2, H_3, Etc.	W_1	W_2, W_3, Etc.	S_1	S_2, S_3, Etc.
Medium to Soft Shale	5-10	Less than 25	Minimum 15	15-20	1/2:1 to 3/4:1	1/2:1 to 3/4:1
Hard Shale	10-15	Less than 30	Minimum 15	10-15	1/4:1 to 1/2:1	1/4:1 to 1/2:1
Medium to Soft S_s or L_s	10-20	Less than 30	Minimum 15	10-15	1/4:1 to 1/2:1	1/4:1
Hard S_s or L_s	15-20	Less than 40	Minimum 10	10-15	Vertical	Vertical to 1/4:1
Interbedded S_s and Sh or L_s and Sh	5-20	Less than 30	Minimum 15	10-20	1/4:1 to 3/4:1	1/4:1 to 1/2:1

Height between benches should be the maximum, but are adjusted (1) to fit over-all weight of cut, (2) to avoid future undercutting of a resistant layer, (3) to produce bench on top a more resistant layer.

Width of benches should be minimum except where height and angle of slope between benches are at a maximum and additional width is deemed necessary.

The angle of the slope between benches should be slightly steeper than the anticipated ultimate slope.

GEOLOGY OF A HIGHWAY SLIDE AT GATLINBURG, TENNESSEE

Delivered at the Third Annual Symposium on Geology As Applied to Highway Engineering

by

Jarvis B. Hadley, United States Geological Survey

Abstract

A landslide during construction of a section of Tennessee State Highway 73 east of Gatlinburg made necessary the removal of about 50,000 cubic yards of material in excess of planned excavation. Removal of this material resulted in a cut 170 feet high. Stabilizing this cut and finding areas on which to put the excess fill presented unexpected problems to the construction company and the highway engineers in charge. Later investigation of the slide area revealed unusual geological conditions, from which these difficulties could have been predicted.

The excavation was made in a slope of 50% to 70% grade, underlain by friable, deeply weathered sandstone overlying less easily weathered slate and siltstone. These beds, dipping about 15° toward the center line, are cut by a high-angle thrust fault, on which a slice of slate and siltstone was brought in vertical attitude against the sandstone near the base of the slope. The natural retaining wall formed by this slice of relatively sound rock was removed in excavating the new grade, releasing the more weathered sandstone above. Movement of the weathered material was aided by separation along steep joint surfaces in the sandstone and by slipping on surfaces of bedding and slaty cleavage in the underlying slate and siltstone.

APPLICATION OF GEOLOGY TO BRIDGE FOUNDATIONS

Delivered at the Third Annual Symposium on Geology As Applied To Highway Engineering

by

Everett Scroggie, Bridge Design Engineer, Tennessee Valley Authority

General

The story is told of a construction engineer discussing a problem in rock excavation with the geologist on the job. The geologist gave the engineer his opinion on the conditions to be expected at certain depths. The engineer disagreed with the geologist and closed the discussion with this statement: "Young man, you can't see a bit farther into that rock than I can!" Literally, the geologist couldn't see very far into the rock; actually, with adequate data on bedding planes, strikes, dips, faults, and explorations, he could project and predict very closely on conditions in nearby areas. I have worked with geologists on foundation problems for the past 18 years and have watched their predictions come true too many times to believe otherwise.

Usually, a bridge engineer has to make his own interpretations of the available data and explorations. Seldom does one have expert geologists available at a moments notice for consultation and advice on bridge foundation problems. In fact, the usual small bridge is designed from information obtained by rod soundings and it is up to the bridge engineer, based upon his experience in the area, to determine the permissible design loads and how they are to be transferred to the foundation materials. On large structures such as arches, or one with continuous spans, or where an unusual load over a large area could cause general displacement, more elaborate data should be obtained in advance of design. Such data include wash borings, core borings, soil samples, open excavations, test loadings, and test piling. The engineer has to decide when he needs a geologist and should not hesitate to call upon him for expert advice. It is difficult at times to limit a decision to geology alone and soil mechanics must be used. This will be discussed later under various types of foundations.

Bridge foundations are usually supported in one of three different ways; namely:

1. Bearing directly on rock of some type.
2. Bearing on piles driven to rock.
3. Bearing on clay or other compressible material.

These conditions are found in many structures with rock varying from granite to limestone and dolomite, shale, sandstone, graywacke, gneiss, mica schist, chert, etc. Limestone areas are the most treacherous and trouble usually shows up in the form of faults, solution channels, seams and caverns -- local areas are unpredictable without complete and detailed information for any particular site.

I am fortunate in that I work in an organization containing a staff of expert geologists who are thoroughly familiar with the entire region and who have had actual construction experience in solving more difficult problems than bridge foundations. I have made use of their services and knowledge on numerous occasions.

Services Rendered by Geologists

Some specific services rendered by the geologists are as follows:

1. Major Bridges Across Reservoirs. After the general direction of the crossing has been established, I usually study the terrain with a geologist for the purpose of making a definite location of the structure

where foundation conditions are the most favorable when considered with all other factors. For instance, a location on one side of a fault may have perfect foundation material, whereas one on the other side may have very poor material. One good illustration is the Tuckaseege River railroad bridge just downstream from Bryson City. The preliminary location was made to avoid excessive damage to adjacent property, but the foundation conditions for the bridge were found to be unfavorable and would require deep excavation in water. The geologists were called upon and made an inspection of the site. They selected a location a few hundred feet upstream and above a fault. A solid rock foundation was obtained for the piers at a much higher elevation.

Opposite to this is the example of four bridges in South Holston Reservoir constructed by the Authority for the Virginia Department of Highways. The foundation conditions were found to be so good that we had to reassure the geologists that they had not been neglected during construction operations.

2. <u>Interpretation of Subsurface Explorations</u>. After a major crossing has been pinned down, a tentative layout for the bridge is made and core borings are drilled at the locations of the various parts of the substructure. These cores are always logged and identified by the geologists. In addition, if sufficient holes are drilled, geologic sections are plotted as an interpretation of the entire crossing. This step is one where various interpretations can be made between the several drill holes and usually the blackest picture is painted. These core logs and geologic sections are used as the first definite basis for the design of the foundations.

3. <u>Foundation Problems Uncovered During Construction</u>. Regardless of usual core borings and probings in advance of construction, problems arise during construction which may require a modification of the foundation design. This is the problem where the geologists are given the design loads and asked "What will happen"? If the load is too great for the foundation material and settlement is to occur, it's up to me to determine if the structure can take it, or if the design has to be changed. Another approach to a solution is possible consolidation of the foundation material by grouting or other means. Of course, these particular problems come outside of the limits of pure soil mechanics or simple pile foundations.

One good example is the foundation for the South Holston River Bridge on U. S. Highway 421 in Tennessee which required expert attention during construction due to the presence of soft, fractured shale which was inadequate for bearing or resistance to erosion by wave action, yet was too hard for the driving of piles. Cast-in-place concrete piles were used in open excavated holes with the geologists checking each for sound rock at the bottom before the concrete was poured.

Another example is the foundations for two large concrete bents in Chickamauga Dam Bridge at the crossing of North Chickamauga Creek. The foundation design is unusual in that fixity depends upon anchorage of a circular footing, about 15 feet in diameter, for a depth of approximately 20 feet in sound rock. The rock is interspersed with bentonite beds which vary in thickness from 1" to 10" at this particular site. The geologists were used to a good advantage in checking the location of these bentonite seams and in giving advice on the adequacy of the rock for the foundation as designed.

Special Problems

Let's think about some of the specific problems which have been solved or aided by the application of geology. I know of one problem I worried uselessly about because I did not remember my geology. About fifteen years ago during the very preliminary study of a stream crossing called Frog Pond, it was noted that the flat, marshy area about 1/4 mile wide was completely saturated and that one would bog up to his hips without any notice whatever. I wondered on numerous occasions if we would ever get piles long enough to reach bearing in this swamp. Subsequently, the area was drained for mosquito control, and a survey was made for the new highway. When I next visited the site I found, to my amazement, a station wagon parked at the edge of the creek and a survey party taking rod soundings. Limestone in a nearly level bed was found under the entire area at a depth of about 2 feet below the stream channel. Belatedly, I remembered that a high water table indicated an impervious material below.

On one occasion I was sent into an area entirely unfamiliar to me to make some soundings for a bridge across a small stream adjacent to a main river construction project. I was told that there was about 40 feet of overburden in this area and that it was just a question of determining the rock surface at the location of the bents so that piling could be ordered. Using only a sledge hammer and drill steel to start with, I found rock just a few feet below the ground surface. This was contradictory to all previous ideas of the site and I couldn't afford to travel back over 300 miles and report rock at such a shallow depth without being sure. So, taking some additional soundings and tracing the bed back to a bluff, I found the outcropping of limestone at the very bottom of the bluff. Then I was sure of myself. This incident serves as a reminder not to be certain of the geology of a specific area without a detail check.

Another incident comes to mind that occurred on a crossing of the main channel of the Savannah River. Only one core boring was made for a certain pier. The drill entered a 6 inch seam and didn't cross it until reaching a depth of about 15 feet below the general rock surface. The design was made, a contract was let for a pneumatic caisson foundation, and construction started. The higher rock surface was encountered with surprise. The design was revised and the footing was raised about 15 feet. The point to be remembered here is that the design was made on the basis of incomplete data - at least four borings should have been made for this pier, preferably five. If the actual rock surface had been known, the design would have been made for open excavation from a cofferdam instead of through a pneumatic caisson. The difference in height of 15 feet simply exceeded the safe limits for open excavation. The very opposite to this can occur when the area contains pinnacles and the drill strikes one at an elevation higher than the general rock surface. In such a case, one really has trouble in redesigning for a lower elevation during construction.

Soil bearing footings are used for minor structures where there is no danger of scour and pressures can be kept low. Grade separation projects are good examples of this type of design. However, some consolidation of the soil can be anticipated and the layout is usually one with simple spans so that adjustment in the bearings is relatively easy. The main problem is to select a safe design load for the soil. If test data are not available, the engineer has to make a decision based upon his judgment of the material. My practice has been to use from one to two tons maximum load per square foot. In the case of soft shale 4 to 6 tons per square foot have been used. One major bridge with spread footings on soil is worthy of mention here. The portions of Chickamauga Dam Bridge located on the south embankment of the dam all have spread footings which bear on the embankment. A design load of 2 tons per square foot was used. The spans are simple, or articulated. Provisions for jacking the bents back to grade if settlement should occur are included in the design. These provisions include a jacking yoke with bolts for attaching to the steel columns, hydraulic jacks with pumps, and extra projection on the anchor bolts for engaging shims. Of course, the embankment isn't an ordinary earth fill, and not very much settlement is anticipated.

The right abutment of the bridge across the Little Tennessee River just downstream from Fontana Dam is supported on spread footings founded on the spoil bank from the rock quarry. Some local grouting was done under the footings, and after seven years service, no settlement has been observed. The end spans were articulated just in case settlement should occur.

Pile foundations are the most difficult, from the standpoint of design, because one seldom has adequate information on the behavior of the piles in advance of construction. Unless the foundation material is entirely uniform, different results will be obtained in the various parts of the same bridge. The load capacity of any end bearing pile can be determined mathematically within reasonable limits if the supporting material is unyielding. The load capacity of a friction pile is never known until it is tested, and considered with the entire pile group.

Let me present a problem in determining the means of supporting an abutment for a bridge and let you decide what type of foundation you would use. Suppose we have a bridge ending in a deep reservoir, or flood plain, with the approach roadway supported on an earth embankment as follows:

a. Height of fill is 35 feet above ground surface, and is to be compacted to maximum density with optimum water contact. Slopes have been designed for stability under 30 feet of saturation and rapid draw-down. Revetment will be provided for protection against scour and wave action.

b. Sound rock is located 20 feet below the ground surface, or 55 feet below the highway grade. The overburden is sandy loam and it is estimated that it will compress about 1 inch under the new embankment.

c. The bridge span at the abutment may be a simple span or one of a continuous unit.

What type of foundation would you use? Would you support it on the embankment, or on the overburden, or on piles, or on the rock?

If it is supported on the embankment, how deep should the foundation be below roadway grade? What is the allowable bearing pressure?

If a pile foundation is to be used, will the bottom of the footings be just below the fill surface or at the top of the overburden? If at the top of the overburden how will the fill be compacted? If at the top of the embankment, when will the piles be driven? Will the piles be friction or end-bearing? Will they be precast, cast-in-place, or steel? Will jetting through the fill damage the fill and also destroy lateral support for the piles? If cast-in-place piles are used, will lateral pressure in the overburden cause collapse of the metal shells before they can be inspected and filled?

If a spill-thru type of abutment is used with the foundations supported on rock, will the backfill be tamped to a suitable density? How will the embankment be placed and compacted around the abutment so as to obtain compaction equal to the adjacent embankment where thorough compaction is readily done? What pressures will be used in designing the abutment?

I've stated this problem and asked these questions to demonstrate that the bridge engineer has to solve such problems with the application of soil mechanics, geology, and judgment based on experience, economics, and common sense.

Foundations supported on rock are not free of problems, particularly in limestone areas. Bedding seams, cavities, and solution channels can be very troublesome. If a small seam or cavity is found at a depth of say 5 to 10 feet below foundation grade, treatment may not be required. If a considerable area is affected and there is doubt that the rock can safely distribute the load, more excavation may have to be made or, the area may be washed out and grouted. I usually leave this decision to an expert geologist after giving him the design pressures. Costs for making additional excavation and extending the pier are compared with grouting or other treatment. Grouting may be done before or after the footing is poured.

Solution channels are numerous in limestone areas and can be troublesome. In general, they can be cleaned out to a suitable depth and backfilled with concrete. Usually, wedging action is obtained from the shape of the channel; if not, support for the footing may have to be provided by extra thickness or reinforcing steel. Some wide channels have been spanned with heavily reinforced slabs.

The occurrence of a fault under a foundation isn't unusual. Several come to mind. One occurred under a large railroad bridge pier in the lower reaches of the Tennessee River. The material on one side was soft, black shale, with chert and clay on the other side. Neither was capable of taking the design loads, so steel H-piling was used instead of the original design for a spread footing.

Another instance of a fault causing trouble was on the Holston River Bridge in Cherokee Reservoir. The fault was in limestone and dolomite and the material was fractured to very small pieces for a considerable distance on each side. Forty-eight inch diameter Calyx drills were used to penetrate to solid material, and the holes were filled with concrete to form huge columns, or piling, under the main footing.

From the points I have brought out, I'm sure you will agree that better and more reliable foundations can be constructed economically by the application of geology to the problems involved.

GEOLOGIC PROBLEMS IN DESIGN AND CONSTRUCTION OF HIGHWAYS IN VIRGINIA

Delivered at the Third Annual Symposium on Geology As Applied to Highway Engineering

by

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The Commonwealth of Virginia from east to west includes parts of the following natural divisions: the Coastal Plain, Piedmont, Blue Ridge, Valley and Ridge, and Appalachian Plateau Provinces. The geological conditions including bedrock, structure, and land forms, in each province are radically different.

The Coastal Plain, representing at least one fifth of the total area of the Commonwealth, is composed of unconsolidated sands, gravels, and clays of Cretaceous, Tertiary and Recent age. As the nearest commercial source of macadam stone is over fifty miles away and with many sand and gravel beds at hand, it is no wonder that concrete rather than macadam roads, ribbon the Tidewater section of Virginia. The topography varies from dissected uplands to broad flat tidal marshes near sea level. Contemporary writers have divided this region into a number of terraces which will neither be named or discussed in this paper. In the dissected portions, where the cuts are deepest, annoying slides sometimes develop between the granular material and the thick impervious clay beds. These slides have been checked by the installation of lateral drains and the cutting of flatter slopes. In other areas this thick bed of clay has resulted in adverse drainage conditions. To remedy this, vertical sand drains up to eighty feet in depth, have been installed. Since their installation, no trouble has been noted in this section.

As the water table is quite high, grades have to be raised higher than normal and where the roads cross marshes and swamps the normal settlement of fills is magnified. On one such fill, across a marsh near Yorktown, settlement seemed to defy all corrective measures. Finally, a series of holes were drilled through the fill and into the marsh. The holes were 45 feet in depth and each loaded with 300 pounds of dynamite. When the shot was made, the muck underlying the fill rolled back on each side, like the Red Sea, and the fill dropped into place. There has been some settlement to date.

Bridge foundations in the Coastal Plain are always thoroughly explored by means of a wash boring unit. In the past, the rod soundings which were used, have turned out to be a unanimous vote for placing all structures on piling. In a large number of cases it is to be expected that all structures will be placed on piling. However, the explorations made with the wash boring outfit have proven that in a number of instances, the beds of sand and gravel were close enough to the surface to enable a gravity type abutment to be used, thus, reducing the cost of construction. It is recognized that wash borings have their limitations, but the time and money spent in explorations has paid dividends to the Department of Highways. On the very large jobs across the Rappahannock and York Rivers, foundation information has been obtained from contractors who were equipped to make deep-water soundings.

In the Piedmont Province, the principal task of the geologist in highway work, is in locating quarry sites, aiding in determining slopes, and combatting slides. The south and central portions of the Piedmont are underlain by granites, gneisses and schists. Despite this wealth of raw material, good quarry sites are extremely rare. The reasons for this are the softness of the rock, and the depth of the soil mantle.

The specifications of the Department of Highways adhere to those of the American Society of Testing Material and the American Association of State Highway Officials, with respect to the abrasion loss of the aggregate, and in the Southern Piedmont, very few stones meet the standards of "A" and "B" stone, most of them being either "C" or "D". Coupled with the softness of this stone is the deep mantle of residual soil which makes the cost of stripping prohibitive for small quarries. An interesting sidelight on this condition is that on the Buggs Island Dam in southern Virginia, the Corps of Engineers used a soft granite for rip rap, concrete aggregate and road metal. This same stone was rejected by the Virginia Department of Highways as being too soft, a grade "C". This is not a criticism of the Engineers, but merely shows a difference in specifications. In the construction of the Philpott Dam in Henry County, the contractor moved some 40 feet of overburden and weathered stone in order to reach suitable rock.

These soft stones and the soil derived from their weathering constitute a serious problem in the design of the road bed. As is to be expected the soils formed from the disintegration of granites and gneisses are extremely high in mica content. This soil is light, fluffy and almost impossible to compact. Consequently, the subgrade must be blanketed with selected subgrade material in order to secure a stable base.

In the central and northern portions of the Piedmont, the question of aggregate is relatively simple. Large areas are covered with limestone conglomerate, red sandstone, and large basalt flows, all of Triassic age. In addition to these, the usual assortment of granites, gneisses and greenstones supplement the supply. It is the Triassic basalt which causes the highway geologist and highway engineer the most trouble. The weathering of the basalt forms a black plastic clay, locally called "blackjack". The more it is worked, the more plastic it becomes and is almost impervious to water. Consequently, unless selected borrow material is used, the road breaks up quickly. In addition, suitable underdrains and lateral ditches must be installed to provide for proper drainage. It is in the Piedmont Province that one observes several of the few deleterious minerals noted so far in Virginia aggregate. Two of these minerals are pyrite and a hydrous, iron, magnesium chlorite known as diabantite. A quarry in the basalt had been used for some years with the stone being used on both primary and secondary roads. On the roads which were not surfaced, the stone performed well; on those which were surfaced, only a short time elapsed before distress and failure in the road were observed. The stone passed all the conventional tests but when a road composed of it failed and was studied, we found that a thin layer of plastic gray mud accompanied by moisture occurred just below the surface treatment. More extensive tests were conducted including the petrographic examination of the stone. These tests revealed that the joints of the rock as well as the hairline cracks had been completely filled with diabantites. This mineral is quite stable if immersed in water, but when the moisture is sealed in, plus the movement of the stone under traffic, the diabantite quickly disintegrates into the plastic mud. As a result of these studies and tests, the quarry has been abandoned. Some traces of this mineral have been observed in other quarries, but not in sufficient quantities to be objectionable.

Another injurious mineral which is found in large quantities in the Piedmont of Virginia is pyrite. Groundwater percolating through the pyrite bearing rocks is converted into a relatively weak sulphuric acid which rapidly corrodes metal pipe. As a result, all drainage structures in this area are of necessity constructed of concrete. Corrosive action also occurs in the Coastal Plain, but here it is due to the salinity of the water brought in by the tides.

Only in the deepest cuts do slides present any problem. Generally speaking, if a slope is cut $1\frac{1}{2}$:1, it will remain stable. The majority of the slides occur in a heterogeneous formation known as the Wissihickon schist. This schist has been granitized in spots with alternating hard and soft layers. The slides usually have their inception between the contact of the hard and soft layers.

The varying difference between the soil mantle and bedrock make thorough exploration for bridge foundations absolutely necessary. In many areas in the Piedmont, the residual soil is very thick, approaching 100 feet; whereas in other areas bedrock is exposed. In our explorations we drill 25 feet where no rock is encountered and where it is found a few feet below surface it is drilled for a depth of 5 feet to make sure it is not just a shell or does not contain clay seams. In the foundations in the schist area, the alternating hard and soft layers cause a great deal of trouble both during drilling and in construction.

In the Blue Ridge Province, the geological problems concerning location and design resolve themselves into a constant fight with slides and rock falls. The more noteworthy slides have occurred in the area underlain by Catoctin greenstone which is a metabasalt. This stone when weathered presents a smooth surface with the overlying residual soil. Water passing along this contact lubricates the rock and before long a slide of major proportions develops. One such slide defied all methods to stabilize until it was dynamited, thus giving a rough surface to the stone. This plus the erection of a stout toe wall has effected a stable condition.

There is no problem involving good aggregate in the Blue Ridge province, since it contains an inexhaustible supply of grade "A" igneous and metamorphic rock. In addition to the rocks which could be quarried, the major stream beds and floodplains are filled with tremendous quantities of material which range in size from one man stones to fine sand. In many cases the material is passed over a grizzly and used without crushing.

Bridge foundations generally present no trouble in the Blue Ridge as bedrock is either exposed or a few feet below the surface. The most serious difficulty found in bridge construction in this province is along the flood plains of the rivers on both flanks of the mountains. Here the thickness and the size of river gravel is so devastating on drilling equipment that a test pit is usually employed. Should the test pit extend through the layer of river jack to bedrock, this is then drilled in order to establish a firm foundation. Usually a layer of soft clay or mud is encountered before bedrock is reached.

The Valley and Ridge Province comprises the western third of the State, but occupies at least 85% of the engineering geologist's time. The bedrock and structure present problems not found in the other physiographic provinces. Only in the case of aggregate is there no intensification of such problems. Experience has proven that the shales found in this region are suitable only for fills and only when properly compacted and drained. The abundant limestones and dolomites found in this province are almost perfect for highway aggregate needs.

There is only a deficiency of the limestone and dolomite in the Coal Measures of the Appalachian Plateau. Here the predominate rock types are sandstone and shale which do not meet with State specifications. The shales exhibit a characteristic of deep weathering far beyond the depth normally expected in this region.

The only detrimental mineral so far observed in the limestones and dolomites has been chert. It has been proven by McConnell, Rhodes, Mielenz and others that any cryptocrystalline silicate such as chert, jasper, and chalcedony are especially detrimental when an integral part of concrete aggregate due to its reaction with high alkali cement. To my knowledge only one instance of this has occurred within the highway system in Virginia. In addition to the unfavorable reaction of chert with the cement, a low gravity chert will cause spalling, map cracking and pop-outs in the pavement. The investigation done in Indiana by Sweet, Woods, and others has determined that in order for these evils to become apparent the chert must be of low gravity (2.50 or lower). One formation within the Valley, the New Scotland limestone of the Heldeberg group, has been found to contain detrimental low gravity chert. In addition to the injurious effects in concrete aggregate, a prepondance of this mineral in bituminous mixes causes excessive stripping, or lack of adhesion between the bitumen and the mineral.

Bridge foundations are more critical in this province than in all the others. Here the cavernous condition of the limestones and dolomites make careful exploration mandatory. The extremes of caves and solution channels found in this region are amazing. In one abutment steel piling was used. The shortest pile was driven 19 feet and the longest 268 feet. On another project, solid rock was exposed, while 10 feet away a clay pocket 70 feet in depth was encountered before solid rock was reached. In some cases, even the shales are somewhat leached forming a rather "vuggy" structure. The sandstone so far has not given any trouble from leaching.

The deformation of the rock which has been caused by intense crustal movement has further accentuated the necessity for the securing of adequate and accurate foundation information. This is exhibited in the very steep dips of both bedding planes and shearing planes. Contrary to the practice prevalent in the Midwest where a limestone of 10 or 20 feet is considered thick bedded no attempt is made to differentiate as to horizon markers in the thick bedded limestones and dolomites of the Appalachian Valley and Ridge Province.

It is in this physiographic province that the most trouble from slides is encountered. This may be explained in several ways, One is the dip of the rock along which water forms an excellent lubricating agent with the residual soil and two, the movement of rock and soil along old fault planes. Most of the slides in this section are composed almost entirely of earth and occasionally the Highway Department is troubled with rock falls.

I recall particularly one highway in the southwest section of the State, where for a distance of 5 miles, the road passes through one slide after another with no economical place to relocate. As in many other mountainous sections, unfortunately, the railroads secured the best grades and locations which generally follow along the flood plain of a river. This leaves the highways perched on the side of a mountain. The intricate folding coupled with the dip of the rock make a careful slope determination necessary. These slopes usually range from vertical to $1\frac{1}{2}$ percent slopes.

Another slide problem of major proportions was encountered in the relocation of U. S. Route 220 between the towns of Clifton Forge and Iron Gate, in Alleghany County. Here the engineering staff of the highway department was faced with the possibility of wholesale slides that not only would have disrupted highway traffic, but would have broken the main line of the Chesapeake & Ohio Railway, as well as destroyed trunkline wire services.

It was on the northern flank of the anticline that the slides first showed evidence of developing. Since the rock was highly fractured and folded, water from the mountains percolated through the cracks and broke out above the base of the new grade. To complicate matters, the old road was perched on top of a narrow, shelf-like indentation, and it was under this that the slides first started developing. The accompanying cross-section shows the geological structure, and the old and new grades.

The engineering staff believed that a massive retaining wall would have a tendency to stabilize the slides. Such a wall was built, and was very effective in checking any large earth movements; however, further creep and seepage caused the bank to slough off and break back into the pavement of the existing road. The geological staff, in consultation with the U. S. Geological Survey, considered several possibilities, and finally decided on drilling vertical sand drains. The idea of sand drains is not new. They have been used with great success in the marsh-lands of New Jersey and California; however, this was the first time, to the writer's knowledge, that the principle has been applied to talus material and to the checking of a slide.

A company that does deep-well drilling contracted to do the work, and 10 holes of 6 inches inside diameter were drilled 80 feet into the overburden and solid rock. This placed the bottom of the holes 10 feet below the grade of the proposed highway. Upon completion of the first hole, it was found that contrary to all expectations, the water did not drain freely through the gravel and clay, since it was compacted to an impermeable state. This difficulty was overcome by lowering five sticks of dynamite into the hole and breaking the ground to allow free drainage.

The installation has been completed for more than 2 years; and at the present time the slide has been checked to the point of a very slow creep, which does not exhibit itself except in extremely wet weather. It is estimated that the drains are now functioning with about 90 percent efficiency. The entire cost of installing the drains was a little more than $5,000 (as against an estimated $8,000), and the savings in maintenance and repairs have doubtless exceeded the original cost.

Other problems which confront the highway geologist is the appraisal of property from alleged damage by blasting, the investigation of springs and wells which have been dried up as a result of blasting, and the appraisal of the mineral values on land used by the Department of Highways. As the Commonwealth of Virginia uses unclassified excavation, this problem which confronts the geologist in other states using classified excavation is nonexistent.

One of the first projects undertaken by the Geological Section, the Statewide Aggregate Survey in which all potential and existing sources for both fine and coarse aggregate are being tabulated and tested, is nearing completion, with approximately 95 percent of the State being covered. As each construction district is completed, a report is written on the same for general distribution. We are furthermore in the experimental stages of using resistivity surveys both for bridge foundations and in the deepest cuts.

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APPLICATIONS OF GEOLOGY TO HIGHWAY ENGINEERING IN KENTUCKY

by

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and

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Kentucky Department of Highways

Kentucky, too, is a state where geology abounds. Little credit for this, however, lies with the highway field, because such a prosaic subject is frequently overshadowed by such features as Mammoth Cave, Sky Bridge, Cumberland Falls, Pine Mountain, and extensive underground mineral deposits. Neverless, the influence of geology in the highway industry is growing, and it is obvious that some potential contributions have not yet been realized.

General applications of geology to highways have actually been in practice several years. Probably the most tangible of these pertains to the development of aggregates. Almost all evaluations of natural aggregates - sand, gravel, or stone - which are proposed for highway construction are made under the direction of a graduate geologist in the Highway Department's Division of Materials. These are not limited just to the usual application of acceptance tests, but involves classification of materials - sometimes on a petrologic or a chemical basis, and always with a view toward the intended use. For example, every producing quarry in the state is included in the Department's Annual Quarry Report (1);

which is, in effect, an inventory and rating of quarry materials, ledge by ledge. So far as the annual report itself is concerned, there are no specific geologic interpretations identifying the formations as time units. Such information would prove superfluous to quarry operators and to highway engineers in general. However, stratigraphic identification has been a part of the work, and these identifications are ultimately published by the Kentucky Geological Survey for the good of those who are in a position to use them (2) (3).

Prior to the time that the annual inventory was developed, many quarries were worked somewhat haphazardly with little or no geologic or mineralogic correlation between ledges. This, of course, led to expensive operations in some ledges before there was any evidence from physical test and chemical analyses to show whether all the stone was acceptable for use. Correlation of ledges greatly reduced the chances of producing large quantities of unsound aggregate, and put aggregate production on a more scientific basis.

In some cases, quarries had to be abandoned because there was not sufficient material of high quality to justify continued operation. At one location, a surface quarry was abandoned, and a shaft was opened to a 30-foot ledge of high-quality limestone laying 240 feet beneath the surface. It is interesting too that a similar operation, but in a drift mine running laterally from the face of an entrenched river valley, was already producing high-quality aggregate from precisely the same formation at a location 20 miles away. Geologic knowledge of the formations - their continuity and their uniformity - was a basis for confidence in an expensive undertaking of this sort. Core-drill records; logging the desired ledge, naturally provided fianl confirmation.

Formative research in aggregate development has involved more specific uses of geologic and petrologic approaches. For example, there have been numerous instances, not only in Kentucky but in practically all states, where current test procedures indicated that an aggregate had dependable qualities, yet service experience has ultimately proven it unsound. In other words, the test- criteria failed to detect these faulty characteristics, and it took several years of service experience to reveal them. One instance of this is illustrated by Fig. 1. The concrete pavement shown there, was built with an aggregate which then showed no apparent signs of weakness. It was hard, resistant to abrasion, had low absorption values, held up well in soundness tests, and seemed well suited for the use intended. After a service periof of about 14 years, the pavement suffered very extensive cracking. It was obvious that the aggregate had "grown", thus exerting a tremendous pressure within the concrete. Ultimately, pieces of coarse aggregate at the surface began to split or "pop" out in fractured sections.

At that time a comprehensive aggregate study (4) was in progress, and one of its objectives was to determine what possible component of stone- limestone in this case - might have caused the destructive expansion. Some indications of the ledge or ledges at fault were revealed in weathered quarry tailings, such as the disintegrated block in Fig. 2. During the course of the study, the face of the quarry was sampled foot by foot, and numerous tests were made. In addition to those normally used for engineering purposes, thin sections were prepared for observation of gross structure; and other tests such as porosity, insoluble residue, and clay mineral identification by x-ray diffraction were included. Concrete specimens were made in the laboratory, and tested by prolonged exposure to water and by alternating freeze-and-thaw temperatures. The freeze-and-thaw test, within the limits usually applied, did not have the severe effect anticipated. However, after an extended period of combined exposure in the durability test, deterioration such as that shown by the specimens in Fig. 3, closely resembled that observed on the concrete pavement.

Another application of geologic techniques - this again having to do with aggregates - was fundamental to the development of sandstone as a useable paving material. The sandstones and shales of Pennsylvanian origin, which are so prevalent in West Virginia, also blanket the entire eastern portion of Kentucky. As shown by Fig. 4, this region represents almost one-fifth the area of the State.

Deep in the heart of this eastern region, where the distance to sources of service-tested aggregate are great, the shipping cost of importing these materials sometimes reaches almost twice the cost of the material itself. Under this stimulus, highway engineers have long been interested in possibilities for utilizing local sandstone. In 1949, a program of development was started inearnest, with the Division of Maintenance prepared to staff and implement quarry and plant operations as well as hot-mix bituminous plant and paving operations for a 34-mile test

Fig. 1. Failure of concrete pavement which began after 14 years of service under moderate traffic.

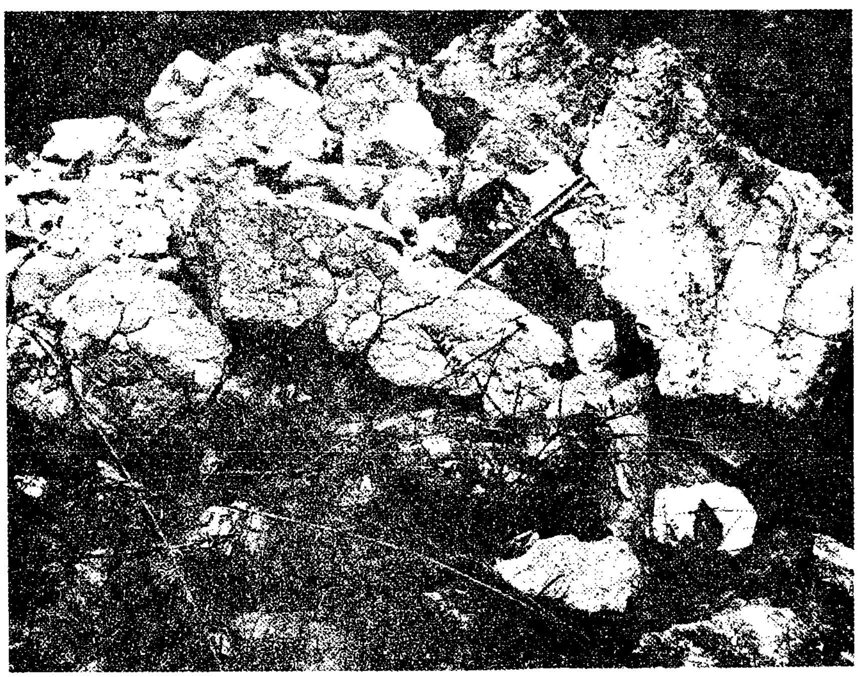

Fig. 2. Weathered quarry tailings such as these offered a clue to the ledge causing the failure.

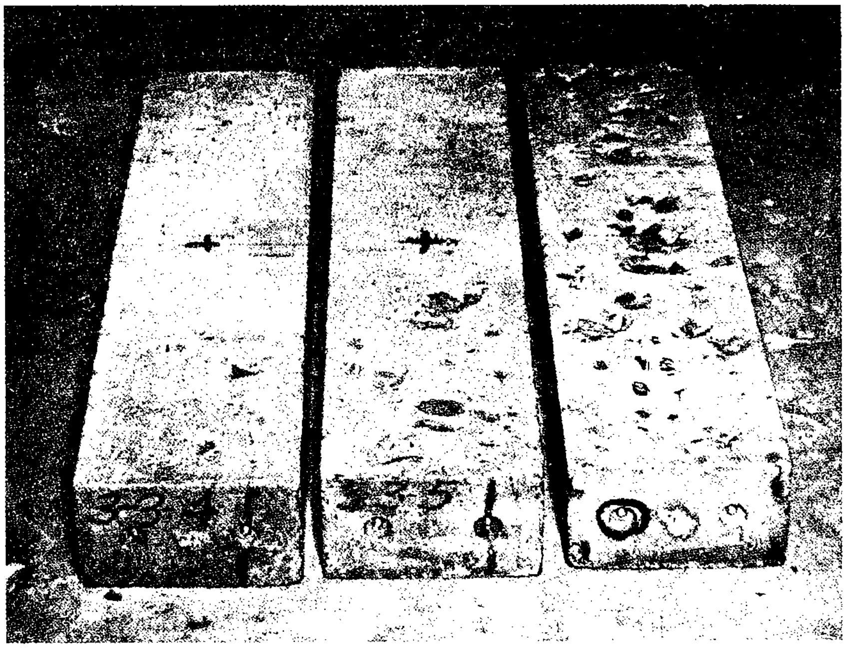

Fig. 3. Concrete specimens after prolonged freeze-and-thaw weathering.

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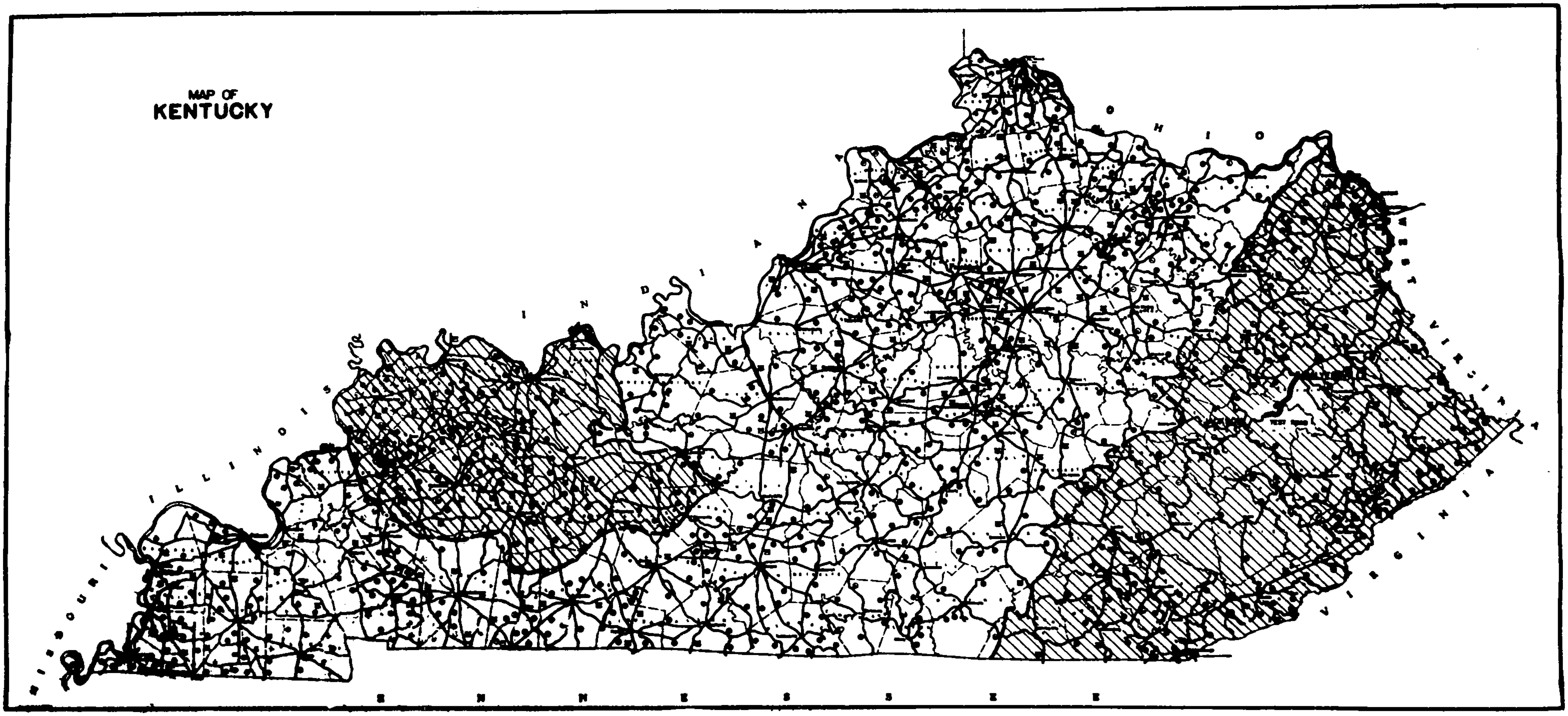


Fig. 4. Map showing Pennsylvanian Sandstone regions and location of test road.

road project. In its formative stage the program was, to a large extent, dependent upon the success of the Research Division in establishing fundamental engineering design data applicable to sandstones and to locate the most promising quarry sites. Geologic considerations came to the forefront, not only in the field surveys and in sampling the outcrops, but also in the laboratory tests for intrinsic properties.

From the beginning, evaluations were based on the premise that the utility of sandstone would be determined largely by the degree of strength imparted to the stone by the cementing media. This general principle was augmented by physical tests for properties such as permeability, porosity, and voids; chemical tests for soluble silicates and basic salts; and optical determinations for roundness of grains, size and frequency of grain distribution, degree of grain-interlocking, type of cementing material, percentage of cement, and mineralogical composition.(5). As expected, composition was extremely variable with major minerals consisting principally of quartz. Minor companion minerals were plagioclase feldspar, muscovite, calcite, biotite, sericite, and chlorite. Predominant constituents of the interstitial or cementing materials were sericite, quartz, chlorite, calcite, and various iron oxides. Fig. 5 shows a typical specimen as viewed through crossed nicols on the petrographic microscope at about 80 x magnification. Quartz, of course, predominates. The interstitial material in this case is largely sericite and quartz, but iron oxides, chlorite, and calcite contribute prominently to the cementing action.

The final laboratory evaluation, preparatory to tests with small-scale bituminous mixes, consisted of strength determinations under combined compressive stresses. This provided a basis for correlation between the properties previously mentioned and the strength factors measured by the triaxial compression test. A sample under test is shown in Fig. 6. Confining pressures ranged from 0 to 5000 lb. per sq. inch; and intrinsic cementing strengths, normally termed cohesion, varied from 600 to 3300 lb. per sq. inch. Grain-to-grain friction, of course, constituted a second strength factor.

Some correlation between the measured intrinsic strength and the degree of cementation as established by optical determinations was apparent - the cementing value of this sense representing the combined influences of percentage cement and degree of grain-interlocking. Discrepancies were observed which may possibly be attributed to variations in the type of cementing material, but the analyses were not sufficient to substantiate this assumption from the standpoint of mineralogical composition alone.

Combined work in the laboratory and on the test road carried over a period of three years; and this project, along with three contract surfacing projects using sandstone, were completed in 1952. Stone for these pavements came from two large quarries, one of which is shown in operation in Fig. 7. Four general grades of material, ranging from very weakly-cemented to firmly-cemented stone, were used successfully. Three of the grades were represented at different levels (Fig. 8) in this one

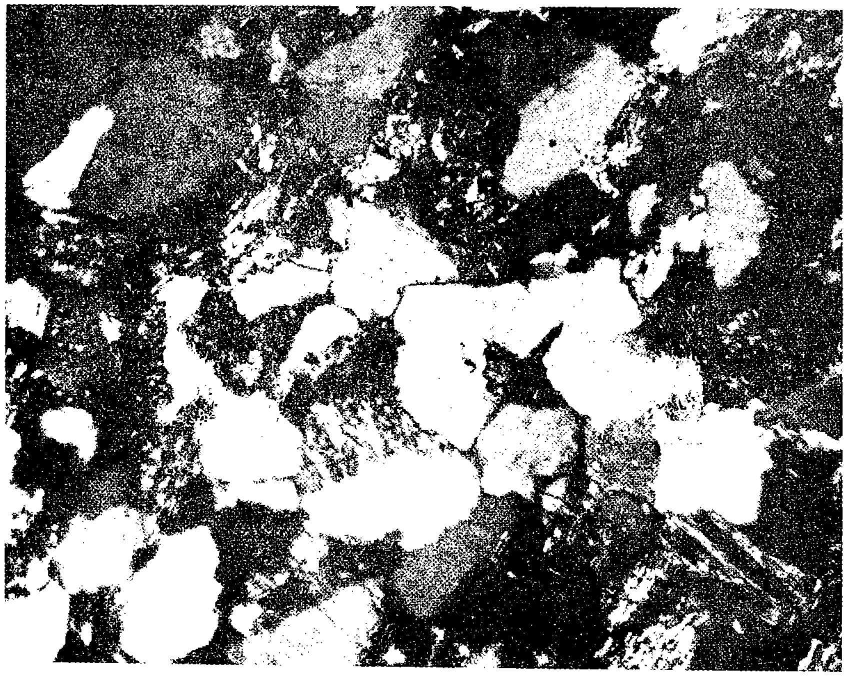


Fig. 5. Photomicrograph of sandstone thin-section, crossed Nicols, 80X magnification

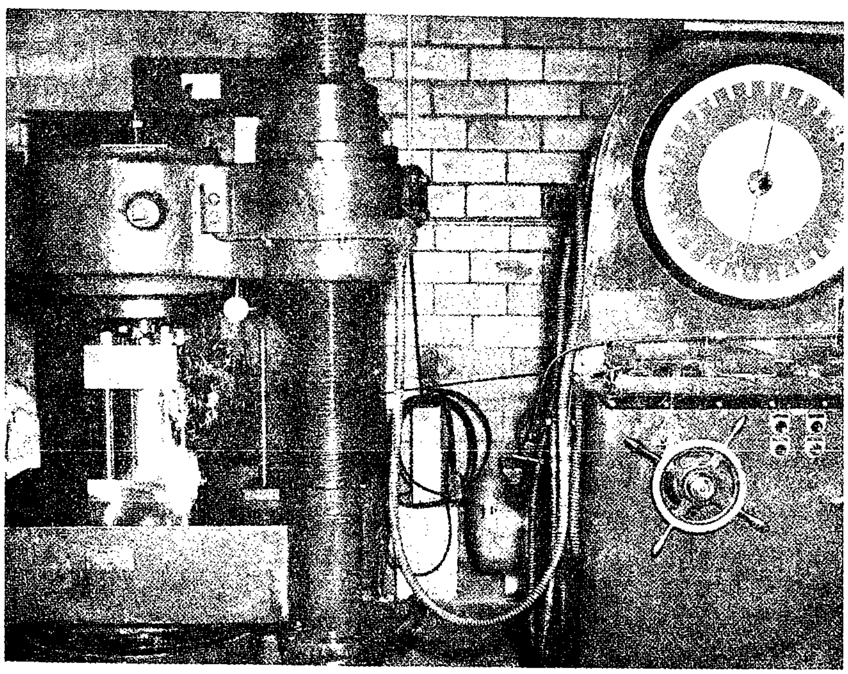


Fig. 6. Triaxial pressure cell used for testing inherent strength of sandstones.

Fig. 7. Sandstone Quarry at Quicksand.

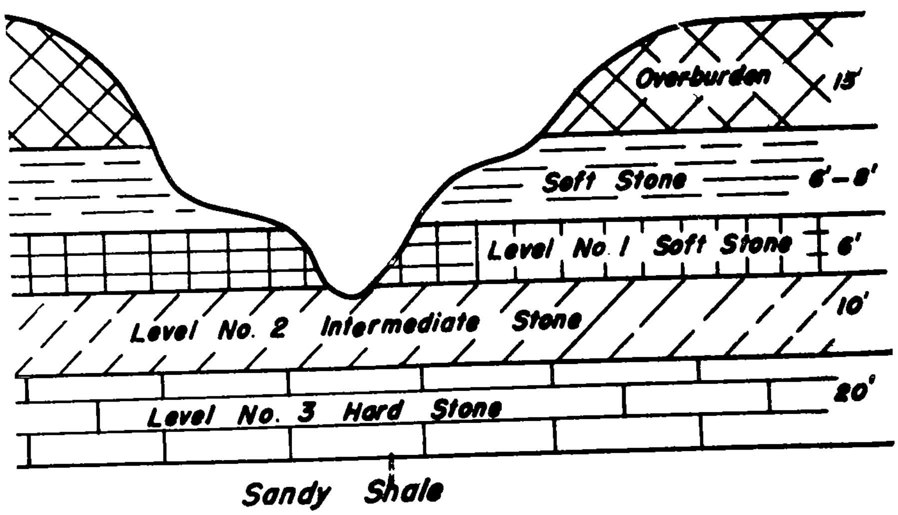


Fig. 8. Generalized Section of the Quicksand Quarry.

quarry. The entire research project definitely established the fact that even though sandstones are extremely variable, a wide variance in composition and physical characteristics are tolerable when the material is used as a plant-mix bituminous paving aggregate (6).

Contrary to the impression conveyed thus far, applications of geology to highway engineering in Kentucky have not been limited entirely to aggregate investigations. The role of geology in classifying soils according to origin is, of course, well established and requires no particular mention here. In Kentucky there has been no deliberate effort in the highway field to catalogue and map soils according to the pedalogic or geologic approaches, yet all the soil samples taken - at least for research purposes - are classified by these systems, where possible. This information now serves as a guide for correlation among samples, and it may ultimately provide a basis for a comprehensive project relating origin and engineering properties of soils throughout the state.

Studies in clay mineralogy and on the influence of different types of clay on soil properties have utilized a number of techniques familiar to the geologist (7). These have, in general, been methods for mineralogical analysis which, in their most practical sense, are simply the means by which the desired information is obtained; that is, information that will in some way enhance the understanding of these otherwise obscure soil constituents. One of the most interesting aspects of these studies is illustrated by Fig. 9. Certainly a knowledge of mineralogical composition is essential to any rational approach to soil chemistry and soil physics. Ultimately, and quite logically, mineralogical composition may be interpreted in terms of geologic origin - not on the basis of quartz sands and silts which occur almost universally but rather on the basis of clays and other complex silicate minerals. At least, there is a suggestion of this possibility from the present data (8) (9).

Physiographic features and subsurface conditions in different regions of the state have an important bearing on a drainage research project which is progress at the time of this writing. The objective in this program is to correlate rainfall and runoff on small drainage areas, and to develop hydrologic criteria for designing the hydraulic capacity of drainage pipe or culverts. The system now used is obviously obsolete, but is is treated in such a way that the design is always on the safe side - sometimes toward opening that are 3 to 4 times the size required for a so-called 25-year storm.

The work principally involves the analysis of long-time rainfall records to determine storm intensities and storm frequencies and measurements of rainfall and runoff relationships on specific areas stratigically located throughout the state. One such area, equipped with stream flow recorder and automatic rain gauges, is outlined on the aerial photograph listed as Fig. 10. Several areas under study, either with these elaborate recorders or with more limited facilities such a s staff gauges or peak stage indicators, are included in the project. The important point, at the moment, is the fact that geologic divisions seem to provide a reasonable basis for zoning the state according to runoff characteristics. This does not mean necessarily that the Mississippian regions are set apart from the Silurian regions, for example, nor that zoning according to similarities in rock type or structure is

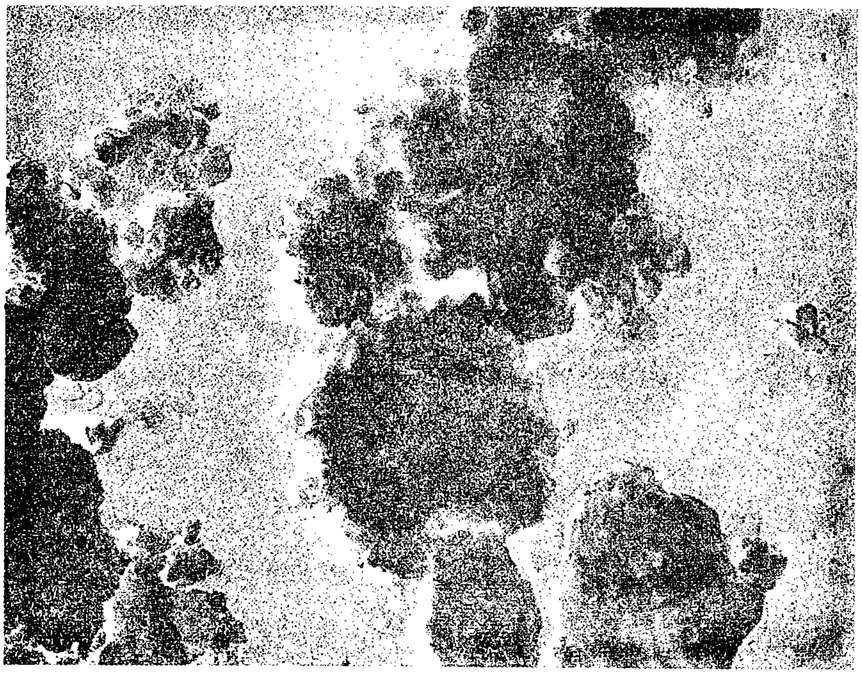


Fig. 9. Electronmicograph of a "shadowed" Illite-Kaolinite clay mixture extracted from soil. Particle size: -lu, Magnification: 22,700 times.

Fig. 10. Airphoto layout of a drainage test area.

a foregone conclusion. It does mean, however, that runoff on the Carbonaceous Devonian Shales in the Knob Region (bordering the Blue Grass) is distinctive; and that, for similar sizes and shapes of drainage areas, runoff under a given storm intensity can be predicted with reasonable, accuracy. For other regions the relationships may not be so definite, but it seems logical that such a grouping can be made within the general limits mentioned.

A rather serious manifestation of another type of drainage problem is illustrated by the severe corrosive action of highly acid mineral waters on concrete bridge piers. An example of this is shown in Fig. 11. Here, the relationship to geology in this problem is better defined. Damage of this type, even on small drainage structures and culverts has resulted in intolerable maintenance costs. Because of this, a survey was made throughout the state, and not only to determine the extent of damage already sustained, but also to determine where conditions producing such damage were most prevalent.

In the field survey, certain tell-tale features such as the presence of iron stains or the absence of vegetation were easily recognized. Water samples were analyzed and rated conductometrically as an indication of their corrosivity. From the earliest inception of the project, a general knowledge of geology, physiographic features, and mineralogical composition of underlying strata offered a general criteria from which to judge the corrosivity of waters in the principal areas of the state. To a large extent, the results of the field survey simply provided factual confirmation of these guiding generalities. This relationship to geology may be more fully realized from the generalized cross-section of the state, included here as Fig. 12. Sulfur-bearing coals and shales interbedded with the sandstones characterizing the eastern and western parts of the state provided the principal sources of corrosive solutes; but, of course, the limestone areas of the Blue Grass and Pennyroyal were particularly free from these acid-producing minerals. These structural and mineralogical aspects are naturally correlative with the principal physiographic regions of the state (Fig. 13); and the areas where high acidity revails are shown by the boundaries of the Eastern and Western Coal Fields. Several mineral springs and wells originating in sulfur-bearing shales within the Knob Region have had some historical significiance but have now fallen into obscurity. Field data from this area indicated a much milder degree of acidity than was generally found for the coal fields. This type of information, combined with about 17,000 inspections on in-service drainage structures (10), provided a reliable basis for the selection of corrosion-resistant materials for use within these critical areas.

These applications described are, of course, typically selected cases and possibly represent instances of personal association; but even this seems particularily appropriate since neither of the authors is a trained geologist - at least, in the academic sense.

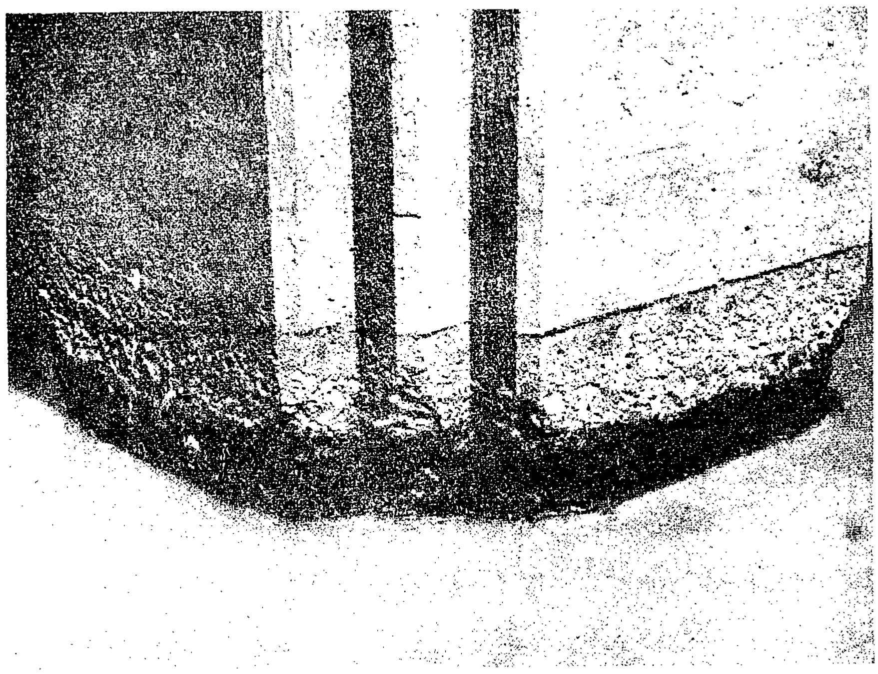
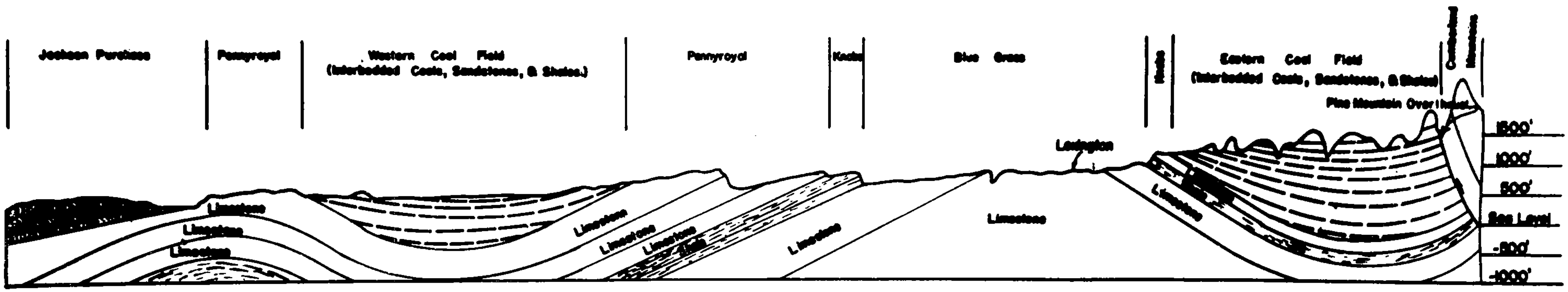


Fig. 11. Bridge pier damaged by exposure to highly
acid mine drainage.



GENERALIZED GEOLOGIC CROSS-SECTION OF KENTUCKY

Fig. 12

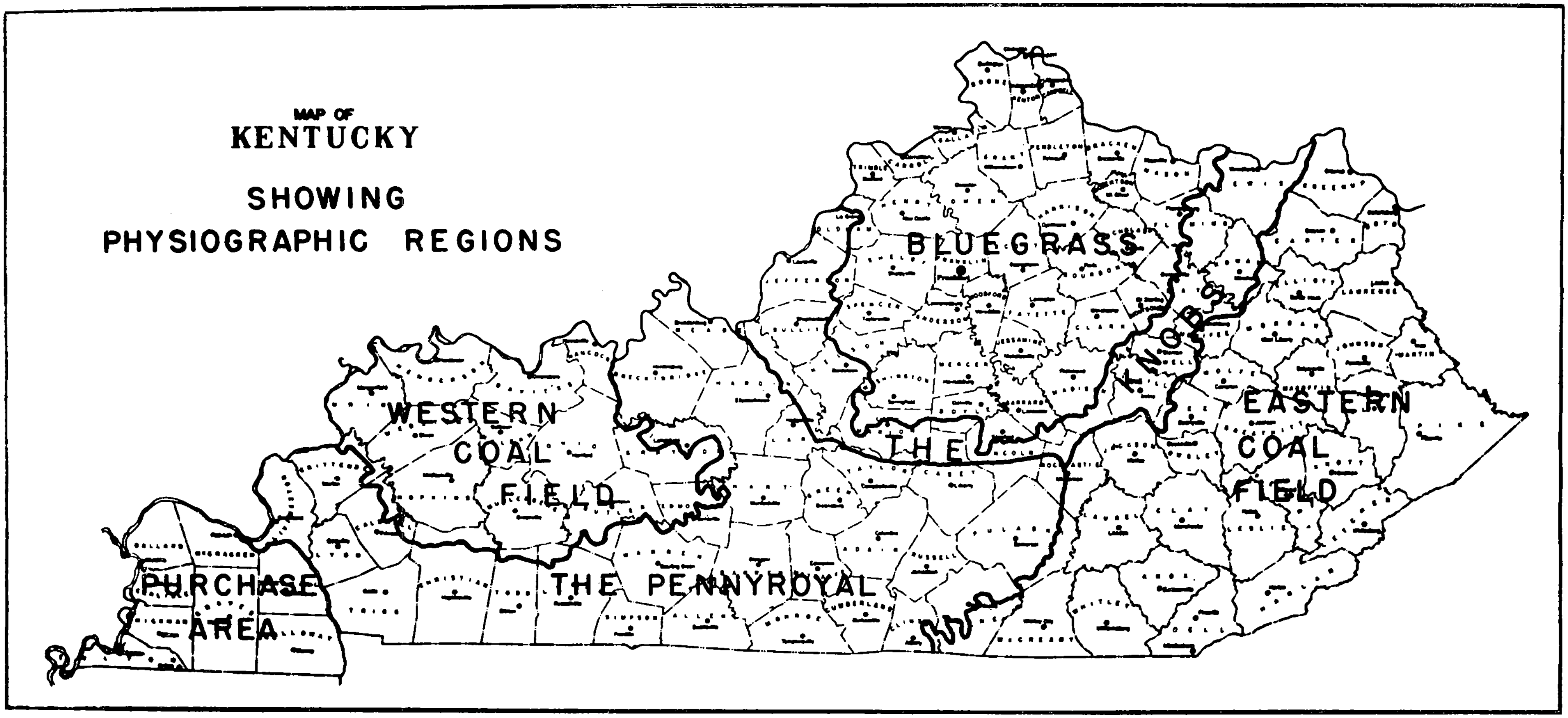


Fig. 13. Approximate Boundaries of Physiographic Regions of Kentucky.

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SOME EXPERIENCES OF THE DEPARTMENT OF HIGHWAYS WITH LANDSLIDES IN OHIO

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INTRODUCTION

Landslides constitute a major problem in the construction and maintenance of highways in a large part of eastern and southern Ohio. It would be difficult to estimate how much of a financial burden landslides have been in maintenance and added construction costs over the years, but it seems safe to say the amount has been considerable. On one project alone during the past year additional work necessitated by three separate landslides, together with payments to a neighboring railroad whose tracks were affected, cost in the neighborhood of a million dollars. On another project also under construction during the past year additional expenditures necessitated by a slide above us were on the order of $350,000. Fortunately, on this project the slipped material that had to be removed from a side hill cut could be used to form embankment for an additional pair of lanes which will now be needed in connection with the Atomic Energy Commission's new plant in Pike County. Still a third project was under contract during the past year at a cost of about $220,000, the sole purpose of which was to correct an unstable slope condition above a newly completed four-lane divided pavement on which two lanes only were considered safe for use due to fallen rock.

Last years landslide correction work on projects under construction was considerably greater than average because of the nature of the terrain in which the work was being done. It is certainly our hope that we will not have another year like it. However, it is the writer's opinion that in the years just ahead treatment of potential landslide areas and of slides that develop during construction is going to continue to be a major problem. The tremendous increase in traffic volume has brought with it a necessity for improving narrow, crooked and hilly sections of road which have long been left as is because, among other things the locale was recognized as being very difficult. We have now reached the point in many of these areas where reconstruction of a roadway of adequate width and suitable geometrics will have to be undertaken in spite of the terrain.

GEOLOGIC CONDITIONS

Ohio is divided into two major Physiographic provinces; namely, the Central Lowlands in the west and the Appalachian Plateaus in the east (Figure 1). The line dividing these provinces is across much of the State a rather clear cut escarpment. The escarpment parallels the south shore of Lake Erie westerly from the Pennsylvania-Ohio line to Cleveland where it turns southwesterly and passes just west of Mansfield and thence through the central part of the State along the east edge of the Scioto Basin, which it crosses at Chillicothe, and thence passes southwesterly to the Ohio River in Adams County near Manchester. Level to gently rolling plains make up the major portion of the State west of this escarpment while the Appalachian Plateaus section is quite hilly with local relief varying from something over 100 feet to about 600 feet along the Ohio River at the extreme eastern edge of the State.

The western and northern two thirds of the State has been covered by one or more advances of continental ice sheets (Figure 2). In Ohio, as in much of northern United States, the glacial drift has considerably modified the preexisting topography by filling the valleys with great thicknesses of till while leaving much thinner deposits on uplands. With the exception of the areas around Cincinnati, where the drift is thin, topographic conditions are not conductive to landslides. We do have occasional sloughing in cut slopes through some of the moraines. These sloughs are usually not of a magnitude great enough to be properly called landslides, but it might be mentioned in passing that they sometimes look big in the eyes of Engineers who have worked only in this part of the State.

Sloughing in these slopes usually occurs near seepage planes where lenses or layers of sand overlay less permeable layers of silt or clay.

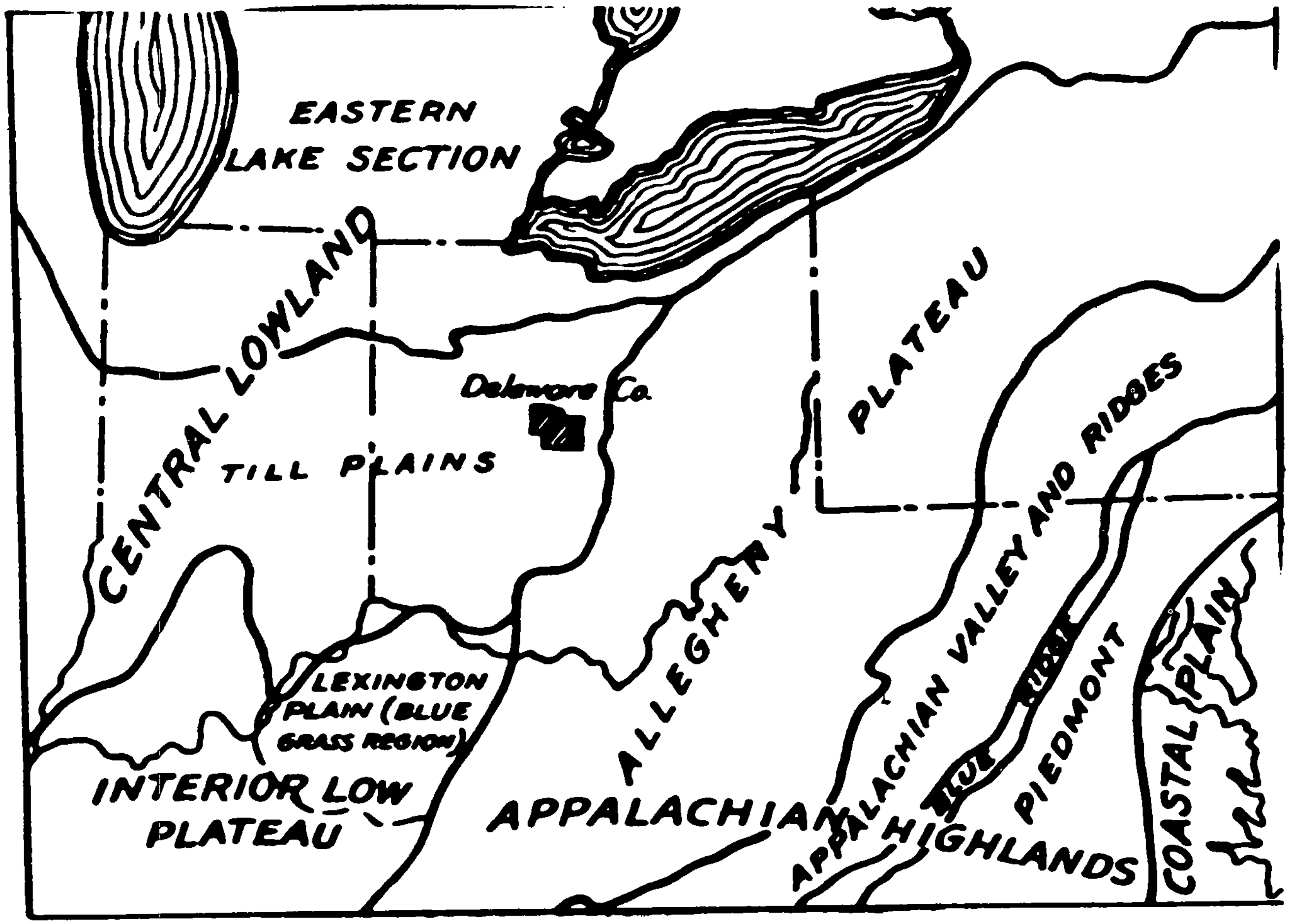
Beyond the limits of the ice advance itself certain valleys contain very fine silts and clays deposited in quiet water during periods when the outlets of the pre-existing drainage channels were blocked by ice. Further, the major river valleys, such as the Scioto, Muskingum and Miami, have extensive outwash deposits varying from gravel to silts. We have had several serious slides along the edges of these debris filled valleys.

The bed rock in Ohio (Figure 3) consist of sedimentary strata of Paleozoic Age. The total thickness of the rock as measured on the outcrop is about 5,000 feet. The principal structural feature is the Cincinnati Anticline whose axis extends across the western part of the State in a north-south direction. The dip of the rock which averages about 20 feet to the mile is so slight that at any outcrop the rock appear to lie horizontally.

In the southwestern part of the State within a radius of 50 to 75 miles of Cincinnati strata of Ordovician Age consisting principally of calcareous shales and thin bedded limestone outcrop. These rocks weather to a very plastic clay and this material coupled with a hilly topography have given rise to a good many landslide situations.

The remainder of western Ohio consists of till plains underlain with thick deposits of limestone and dolomite of Silurian and Devonian Age.

The outcrops of the Mississippian system cover a wide belt across the east central part of the State. The system consists principally of sandstone and shales with generally greater resistance to erosion than the underlying formations and, consequently, a fairly rugged topography has developed in the outcrop area. The over burden in much of the area of outcrop is rather thin and is somewhat sandy in character. Slopes in the area whether natural or man-made as a general rule are stable and landslides in highway work are relatively uncommon. However, where



PHYSIOGRAPHIC DIVISIONS
OHIO AND ADJACENT TERRITORY

FIGURE 1

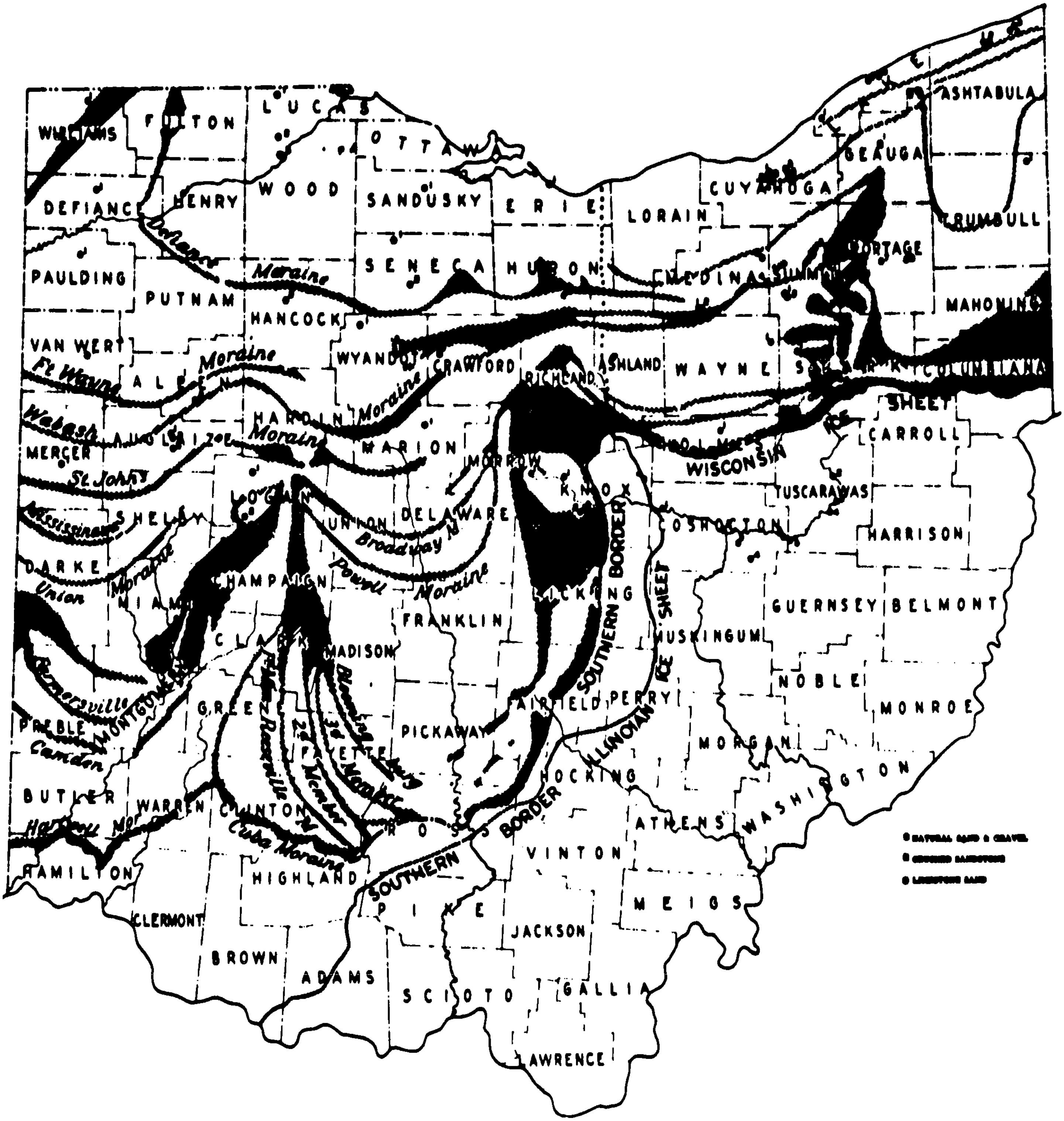
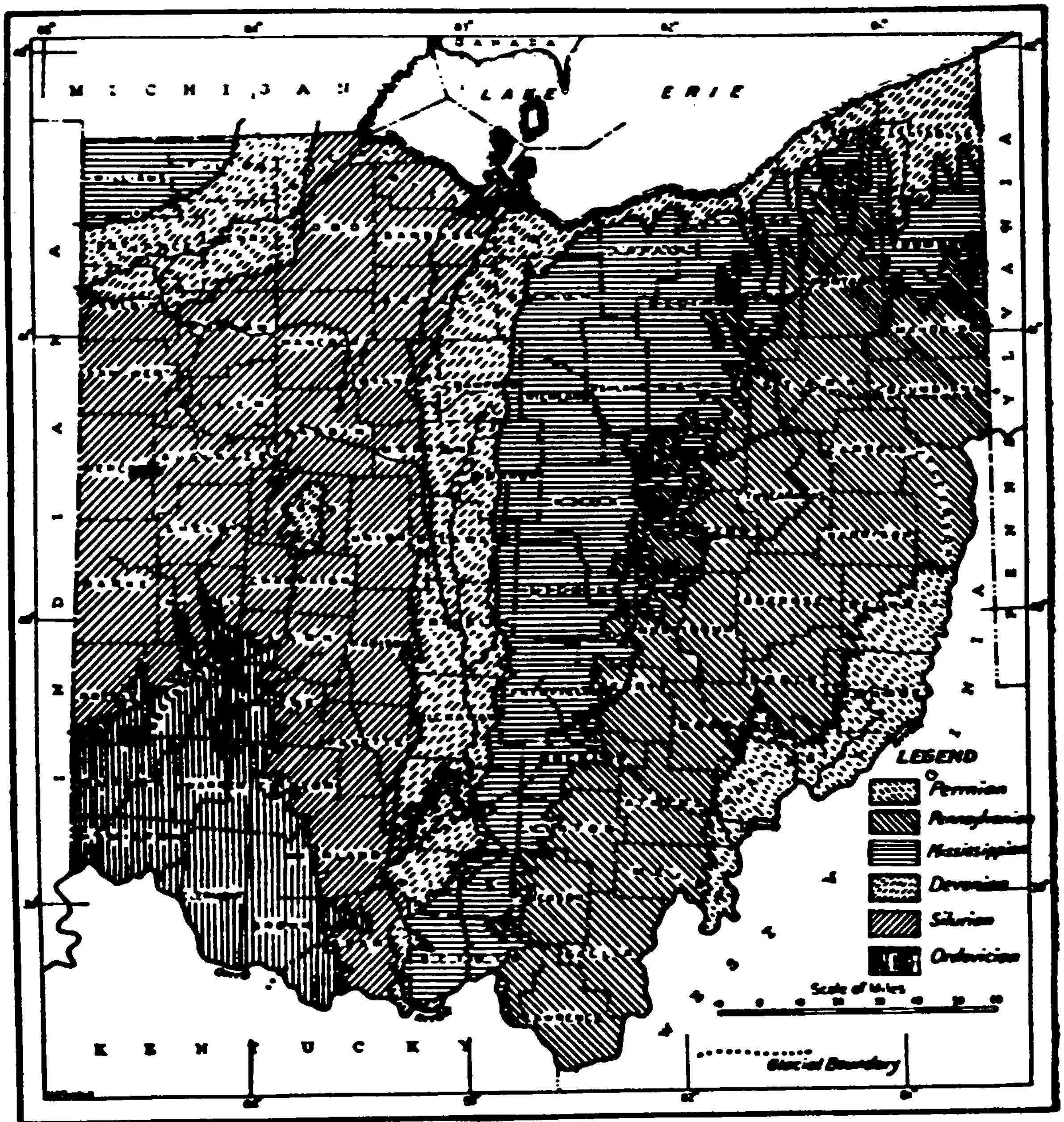
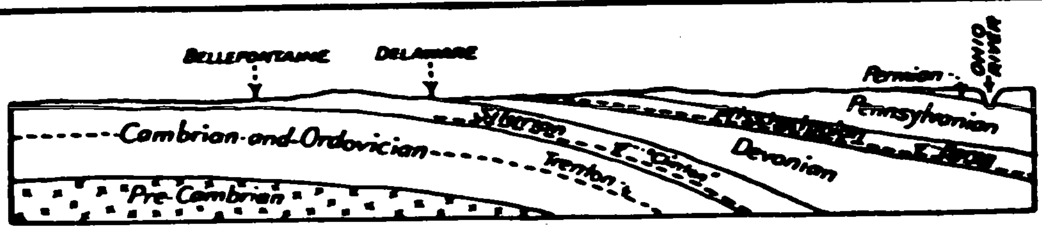


FIG. 2
GLACIAL MAP OF OHIO

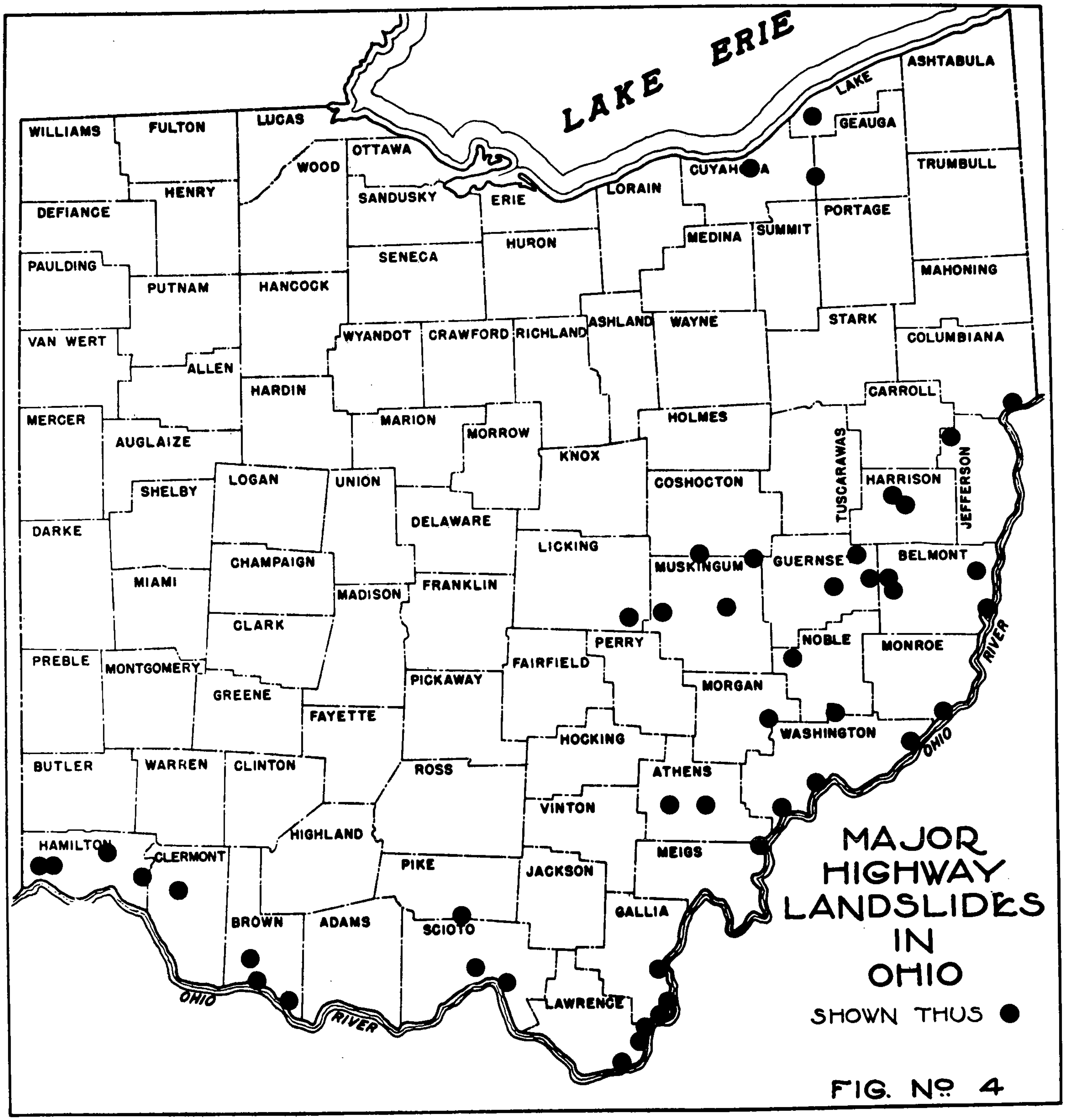
FROM OHIO GEOLOGICAL SURVEY
REPORT OF INVESTIGATION Nº 6





Below is a cross section from Bellefontaine, Logan County, through Delaware to the Ohio River.

FIG. 3
GEOLOGIC MAP OF OHIO
FROM OHIO GEOLOGICAL SURVEY
FOURTH SERIES BULLETIN Nº 30



MAJOR HIGHWAY LANDSLIDES IN OHIO

SHOWN THUS ●

FIG. No. 4

the slopes and valleys are plastered with glacial debris unstable conditions may be encountered.

The coal bearing formations of Ohio consist of a great series of shales, sandstones, clays, limestones, coals and clays occurring in irregularly recurring cycles. They outcrop in the hilly Appalachian Plateaus area of the southeastern part of the State. The extremely plastic fire clays in the Pottsville and Alleghany formations and the red clays of the upper part of the Connemaugh and higher rock units give rise to many situations where landslides develop.

DISTRIBUTION OF LANDSLIDES

Landslides occur in Ohio principally (Figure 4) in the unglaciated Appalachian Plateaus region of the southeastern part of the State. Here the combination of moderately hilly topography and coal measures shales and fire clays give rise to situations where construction of a modern highway often must be done through areas of unstable earth materials. Landslides develop also in the valley slopes in the dissected terrain in the vicinity of Cincinnati. Here the soft clay shales which occur interbedded with the thin limestones of the Ordovician System, particularly in such formations as the Eden, weather to very plastic clays which are extremely susceptible to sliding in highway cuts or in foundations under site hill fills.

From the viewpoint of the highway Engineer the landslides with which he must deal are divided into two classes, those which occur in slopes above the road and those which develop in the slopes below the road.

Slides coming in from above range from small movements in the slope itself which never reach the ditch through those which bring material down onto the shoulder and pavement. In many cases the slides are such that an upheaval will raise the pavement. Such slides usually occur where side hill cuts have been made in which the upper end of the cut slope emerges on a natural slope.

Fill slides, or those which occur below the road, are also almost always associated with a sloping ground surface where the natural ground continues to descend below the toe of the fill slope. Such slides may involve the shoulder and all or part of the pavement itself.

INVESTIGATION OF SLIDES

Modern highway engineering has come to the point where the need for careful and thorough study of soil and rock conditions along the proposed route is recognized as an essential part of sound and economical design. In terrain where landslides are a probability this investigation is especially important. In Ohio for each new highway project involving new construction or reconstruction on new line and grade a survey is made of soil and rock conditions. Test borings are made usually at intervals of 200 feet along the center line and at critical points transversely to the road in areas involving side hill cuts or fills. The procurement of test holes and of disturbed samples of soils has been very greatly expedited in recent years by the use of power earth augers. The work of the soil drills is supplemented where necessary by core drilling and within the past year by use of electrical resistivity equipment. Rock exposures in nearby cuts, stream beds and cliffs are noted and logged by geologists working with the soil surveying party and in many instances the information thus obtained will greatly minimize the need for extensive core drilling. The information obtained from these routine surveys is utilized by the design engineers for determining stability of foundations, side slopes in cuts and supporting strength for pavements.

Similiar equipment and methods are used in the study of landslides except that the work is done in a smaller area and on a more concentrated scale. The conditions are usually such that the most reliable and valuable information must be obtained by core drilling. The core drill is used to get information on the soils and slumped material and also to locate the slipping plane, the top of solid rock and the character of the bedrock underneath. Borings are usually made on cross sections extending from head to toe of the slide. Conditions are frequently encountered where it is very difficult or impossible to get the core drill in to the points where information is most urgently needed. On several slides in recent years we have used electrical resistivity methods with considerable success to determine the top of bed rock in areas too rough for the core drill. The resistivity work has also been very helpful on jobs where information must be obtained in a hurry if it is to be of any value in reaching a decision as to the approximate treatment of a slide. Many of the most serious slides develop on jobs under construction in which delay in instructing the contractor as to what is to be done may lay us open to claim becuase of the contractor's loss of time. Although we attempt not to leap at conclusions without proper information, any reliable means by which the data can be obtained more rapidly than by direct drilling is extremely worthwhile.

LANDSLIDE CORRECTION

Mr. Baker in his very excellent paper on landslides presented to the Highway Research Board in January, 1952, divided landslide corrective measures into two broad classes, Elimination Methods and Control Methods. Under elimination methods, Baker classes such procedures as relocation away from the slide or complete removal of the slip material and reconstruction on a solid base. Under control methods he classifies procedures which tend to arrest the movement while leaving the slide material more or less in place, such as improving drainage conditions lightening the load or counter balancing the toe. These methods might also be characterized as "brute force" on the one hand against "re-established balance" on the other. Both types have been employed extensively on landslides in Ohio. For big slides on primary roads and particularly on jobs during construction, our tendency in recent years has been to resort to the elimination types whenever possible even though it may substantially increase the cost of the construction. It is our thought that these treatments will pay for themselves in reduced maintenance cost later on. Further, the work necessary can be most economically and expeditiously completed with the contractor's equipment and organization which is available on the job during the construction.

DISCUSSION OF CORRECTIVE TREATMENTS

I. Benching and Reconstruction

Probably the most frequently used of the positive methods of landslide control in Ohio is the practice of simply digging out the slip material and reconstructing the cut or fill slope from a truly firm foundation. Before undertaking the benching sufficient sub-surface information must be obtained to show definitely that a suitable foundation can be reached, for considering the cost of this treatment there must be no doubt as to its final success. The following figures show one such treatment used on the $1,000,000 slide mentioned in the opening remarks. This slide is located between Wellsville and East Liverpool on the eastern border of Ohio on the north bank of the Ohio River. The valley slope here rises steeply from the edge of the river upward for a distance of 600 feet in something less than $\frac{1}{4}$ of a mile. On the lower part of this slope are located the double track line of the Pennsylvania Railroad and above it Ohio State Route 7. A project is under contract to reconstruct this section of Route 7 replacing the old 20 foot roadway with two 24 foot lanes separated by a 4 foot median. In the fall of 1951 a slide developed in the side hill fill near the middle of the project (Figure 5). Since the fill was nearly to grade the small amount of additional filling required was brought in after cracks first appeared. However, it was soon recognized that this procedure could not be continued since the additional loading caused by our fill was producing dangerous dislocations of the railroad below (Figure 6). After considerable investigation and study it was decided that complete removal of the slide and benching out to solid shale below was the most desirable treatment from the standpoint of providing the most positive protection to both the new road and the railroad. In the typical section (Figure 7) the method of benching the use of porous backfill between the material left in place and the reconstruction fill, and the revision of highway grade line are shown. The lowered grade line was cons-

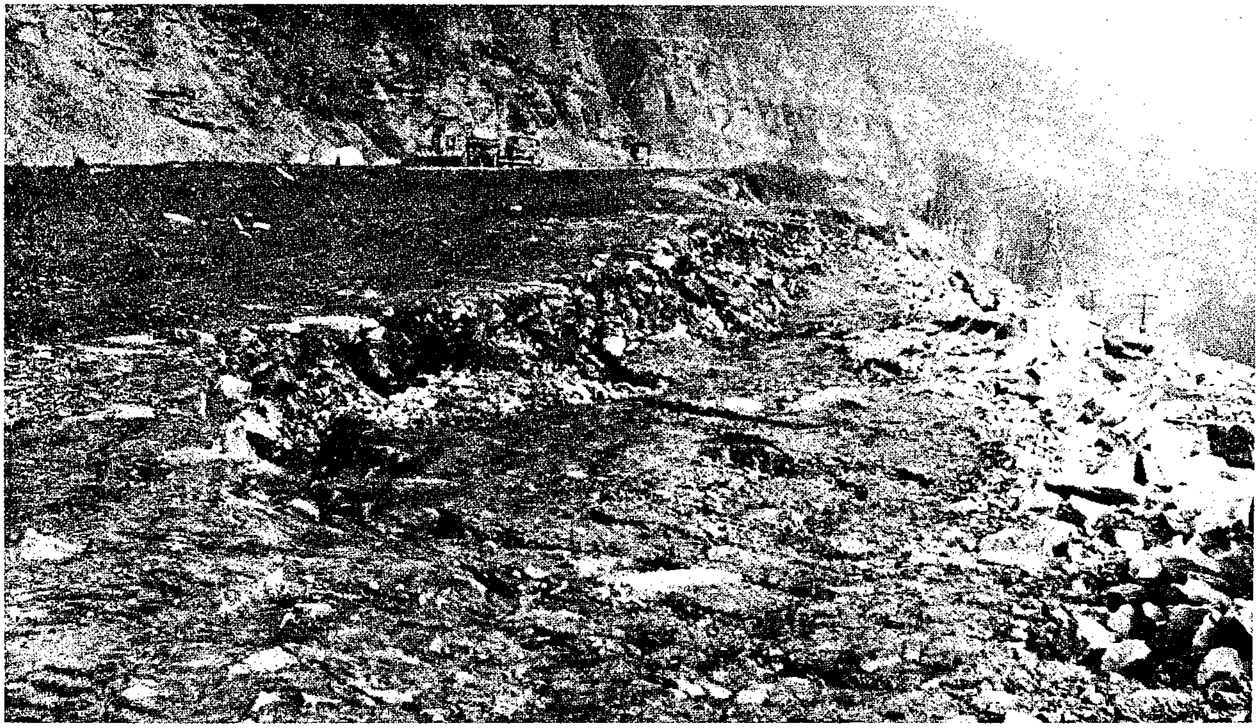


FIGURE 5

Land slide in full constructed on the lower part of the valley slope of the Ohio River near East Liverpool, Ohio, November 1951. Foundation for this fill was a clay and boulder talus resulting from the weathering of the shales and clays of the lower part of the Allegheny formation in the vicinity of the clarion clay.

FIGURE 6

Showing dislocation of railroad below toe of the highway fill shown in Fig. 5.

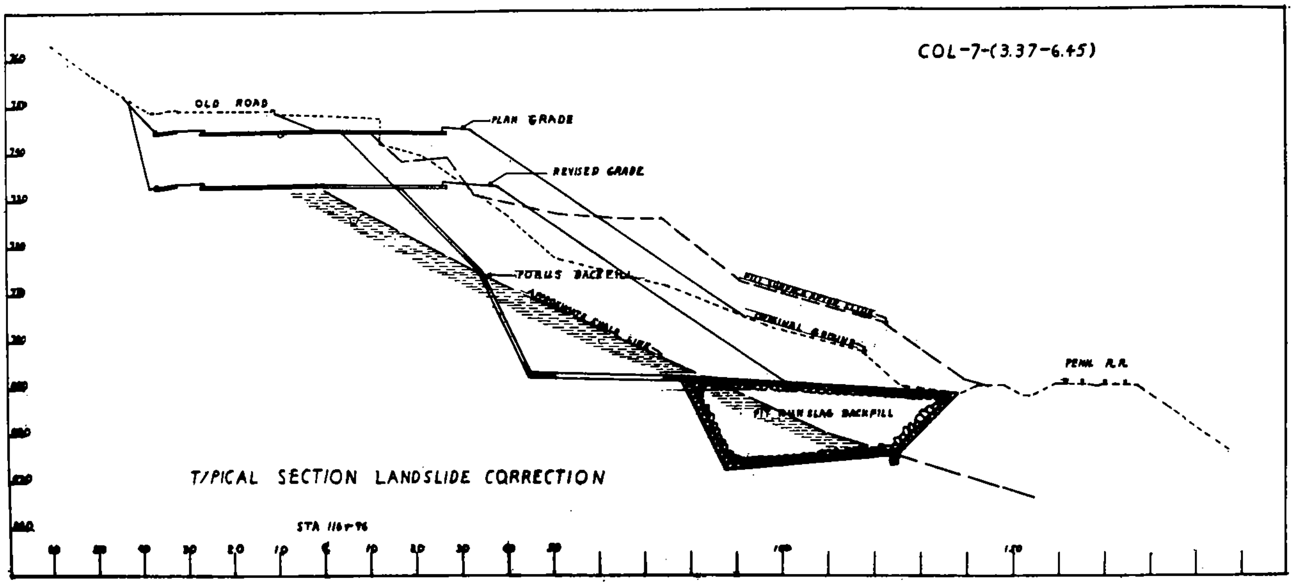


FIGURE 7

Typical Section showing benching done to get roadway fill in the East Liverpool slide founded on firm shale. Note slag backfilled trench and porous backfill placed up slope between the reconstructed fill and the shale.

FIGURE 8

View showing lowest rock bench from which reconstruction of the fill was begun. East Liverpool slip April, 1952.

FIGURE 9

Fill partially reconstructed. East Liverpool slip May, 1952.

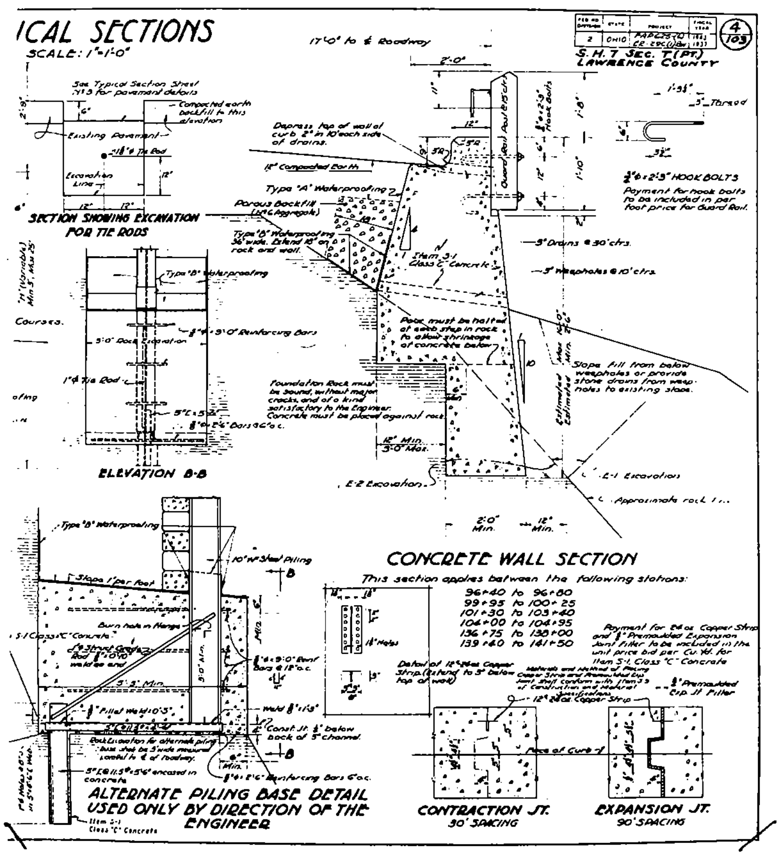


FIGURE 10

Concrete wall section used to support roadway shoulder near the face of a sandstone cliff - State route 7 near the Lawrence - Gallia County line.

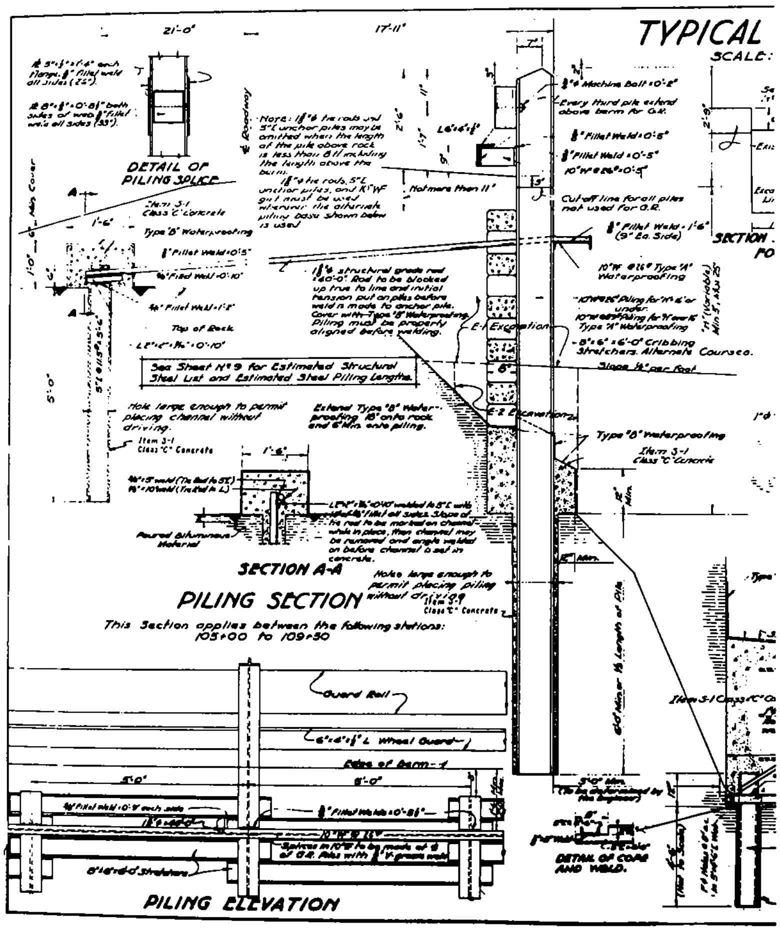


FIGURE 11

H-Beam piling section used to support roadway shoulder near the face of a sandstone cliff. At least 1/3 of the length of the pile is encased in concrete in a hole drilled into bed rook.

FIGURE 12

View of H-Beam piling installation used to hold the roadway shoulder near the Lawrence-Gallia County line on S.R. 7.

idered essential to make sure that the projected fill slope of $1\frac{1}{2}$ to 1 would intersect the shale bench. In securing the necessary information on the extent of the slide and depth to solid foundation core drilling, power auger and electrical resistivity methods were used.

The electrical resistivity work in which we were assisted by Mr. Moore of the Bureau of Public Roads in Washington, was very reliable as to location of shale and was particularly helpful on the rock fill through which drilling was practically impossible. Figures 8 and 9 show the benching and the reconstruction of the fill.

II. Concrete Walls

Concrete walls have been used to a limited extent to hold the shoulder of a roadway where it would be impractical to build a positively stable fill. One such wall is shown in Figure 10. The job from which this illustration was taken is on State Route 7 in southern Ohio near the Lawerence - Gallia County line. On this particular route the road was relocated in 1939 to get it away from the unstable talus slope on which it previously rested. The new location was up on one of the massive sandstone ledges of the upper Connemaugh formation. Due to the considerable excavation required to obtain the required roadway width throughout the length of the relocation certain sections were constructed as side hill fills which rested on the same sloping talus below on which the old unstable roadway lay. These fills proved to be a source of continual trouble. A plan was therefore developed for anchoring these shoulders to the rock ledge as indicated. Concrete wall was constructed where the distance from the shoulder to the solid rock was on the order of 5 to 16 feet.

III. H. Beam Piling

H Beam piling have been used extensively in landslide work in Ohio with considerable success. Figure 11 shows how these piling were used on the project just mentioned. Height of the H Beams above the top of rock ranged from 3 to about

25 feet. They were used in one section on this project instead of the concrete wall because of their greater economy and strength where the rock was located at greater depths. At least 1/3 of the length of pile was imbedded concrete in a hole bored into the bed rock. Piling were set in a single row on 5 foot centers. The space between the piling was faced with 6 foot cribbing stretchers placed in alternate courses. Where the length of the pile above the rock foundation was more than 8 feet anchor piles were set in rock on the opposite side of the road and the two rows of piling were tied together just under the finished roadway surface with a 1-5/8" steel tie rod. Figure 12 shows how the finished piling look from below.

H Beam piling have been used also to hold slides coming in from above. In most instances where H.Beams were so used the slide was one in which slipping material from above was causing an upheaval in the pavement sub-grade. The pile were usually set in a double row along the shoulder or under the ditch line and are drilled into the rock below. Above the solid foundation the piling were usually filled in between with large rock and often were tied together at the top by a concrete cap.

Well casings drilled into rock and filled with concrete have been used extensively in efforts to check slides, oftentimes fairly successfully. Driven timber piling are used even more frequently by our maintenance forces in an effort to bring slides under control. In the case of both well casings and of timber piling these piles are used without much previous investigations of the slip and often without much knowledge of the overall conditions involved. Driving piling is something which the Division maintenance forces are equipped to do and they are therefore often used to provide temporary relief. It is relatively seldom that such piling provide a truly satisfactory permanent cure.

IV. Relocation

Wherever possible potential landslide areas are avoided by shifting the line to more stable terrain on new location. This is one of the merits of careful study of soil and rock conditions for new construction work. In some cases potentially unstable conditions can be avoided by a slight shift in the line of grade. On the project on Route 7 mentioned previously near the Lawrence-Gallia County Line there were several sections where solid foundation for reconstruction of the existing road was at such great depth as to render impractical any correction such as digging out the slip to solid foundation, or of using walls or piling. In these sections some sacrifices were made in alignment and the road was shifted further into the hill and the solid rock (Figure 13).

V. Flattening Side Hill Fill Slopes and Counter Balancing the Toe

In some instances side hill fill type of slides have been brought under control by use of flatter fill slopes and by loading at the toe. This treatment is only successful when it is possible to load the toe further without producing an independent slide with the loading. We have used the method with some success only where the toe Loading would stand on a fairly level plain, and where other treatments seemed to be out of the question. The slide represented in Figure 13 was too large and the depth to any sort of solid footing too great (over 80 feet) to be economically treated by one of our "brute force" methods. However, the flood plain at the toe of the fill slope was of sufficient width to permit flattening of the slope and loading at the toe. The control methods used here were a combination of improved surface drainage, slight line shift into the hill flattening of the slope and loading of the toe.

VI. Benching and Flattening Back Slope

One of the most frequently occurring types of slide are those involving the slope above the road. Where there are no expensive properties affected near the top of the slope or other important roadways whose position must be kept secure, treatment of these slips is simple. Side slopes are flattened and if desired benches provided back of the ditch line or on the slope or both. Benches ranging in width from 12 to 40 feet have been used back of the ditch line, particularly in cases where there has been a tendency for the subgrade to heave in the pavement area. The bench back of the ditch is intended to provide both a resting place for further slump material from the slope above and also as an area of low surcharge so that if further movement takes place the heaving will be in the bench rather than the subgrade.

VII. Lighten Load

On the project between Wellsville and East Liverpool mentioned in some detail earlier in another section of side hill filling, slides developed during the construction. Not much actual settlement took place; however, a longitudinal crack developed along the center line of the fill. Investigation by core drilling showed no solid material within the limit of practicable excavation and reconstruction due both to the depth to rock and the proximity of the railroad.

H Beam piling set in rock would have had to be on the order of 45 feet in length and the installation cost would have been extremely high. However, it was possible to lower the grade about 10 feet at the point of maximum movement and thus lighten the load. Figure 14 shows the rolling grade that was resorted to to accomplish this load reduction. Whether this load reduction was sufficient to prevent further slipping remains to be seen. However, in view of the extremely high cost of other possible solutions it was considered reasonable to try the lighter load procedure even though it was admittedly not absolutely positive.

VIII. Drainage

Landslides are almost always aggravated by conditions of high rainfall and ground water seepage. In correcting slides we make every effort to improve drainage conditions. When complete removal and replacement methods are used we always accompany the treatment by suitable sub-surface drainage to protect the reconstructed fill from further entrance of sub-surface water. Due to the difficulty of locating the exact source of sub-surface water, we have not attempted much slide correction by sub-surface drainage alone. We have always made every effort

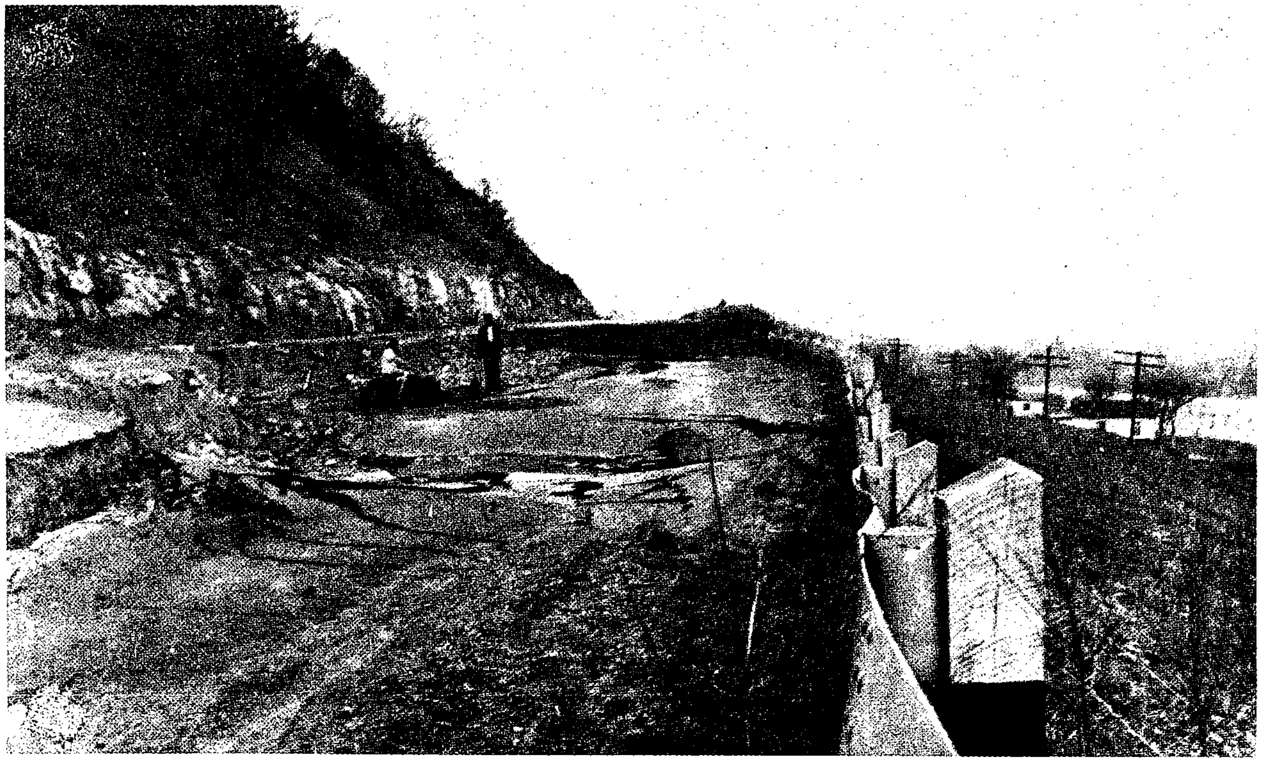


FIGURE 13

Land slide near the Lawrence-Gallia County line on S. R. 7. Depth to rock here was so great that neither benching to solid foundation or installation of H-Beam piling was considered practical. The corrective work used here consisted of a slight shifting of the line toward the hill and of flattening the fill slope and loading the toe.

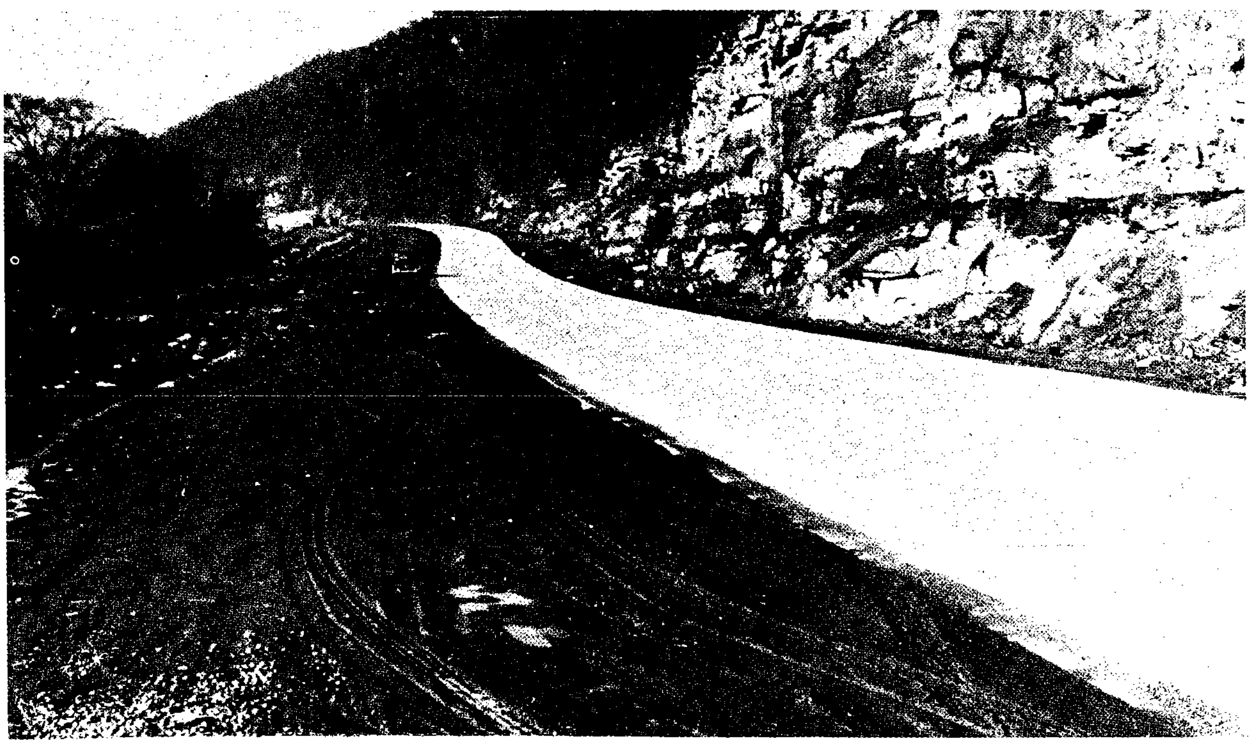


FIGURE 14

Grade line was lowered about 10 ft. at the point where the car is parked in order to lighten the fill loading here. Depth to solid foundation under the slide which developed here during construction was too great to make some type of more positive correction practicable.

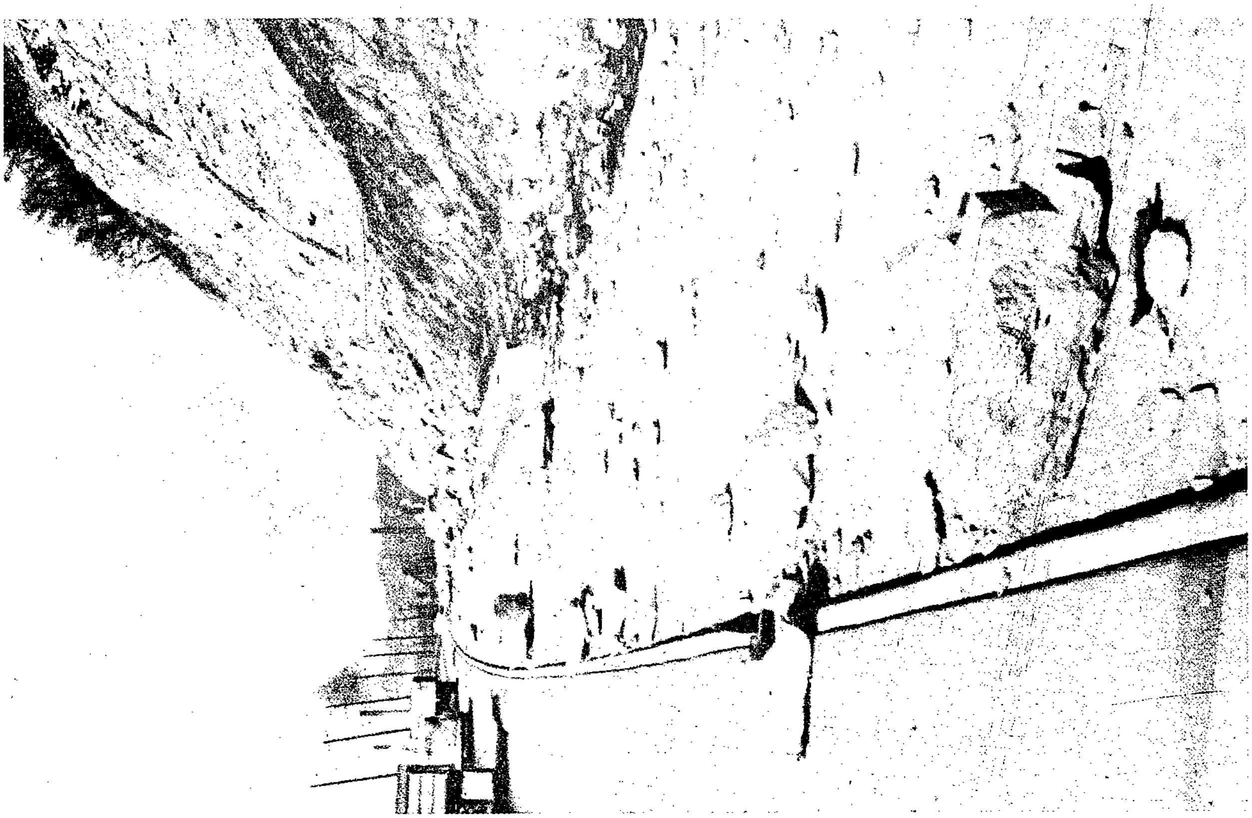


FIGURE 15

Rock fall Route 52 in Portsmouth. Corrective work consisted of steepening the lower part of the slope to provide a 30' bench back of the curb to form a resting place for the rock.

FIGURE 16

Rock Fall S.R. 7, Southwest of Marietta. The fallen rock come from a ledg about 30' thick which is underlain by about 12' of soft shale between the base of the sandstone and the road level.

to divert surface water from slide areas and usually on up hill slides where the material is not to be removed have made an effort to fill surface cracks and reshape the mass where necessary to assure rapid surface runoff. Intercepting ditches are used under special conditions above cut slopes to divert surface water.

IX. Rock Falls

Free falling and rolling rock constitute a serious problem on many miles of our highways. Figure 15 shows a section of recently built 4-lane highway on which so much rock fell that the City of Portsmouth refused to allow traffic to use the lanes closest to the hill. A supplemental contract was let to provide a bench width of 30 feet from the back of the curb to the face of the rock slope. This bench was sloped downward toward the hill to aid in holding rock from rolling out into the highway. The additional width was obtained by steepening the lower part of the slope to $\frac{1}{4}$ to 1. A guardrail was also provided to prevent the rock from rolling onto the pavement.

A section of SR 7 just southwest of Marietta was relocated about 13 years ago up onto the hillside to get it off of the sliding talus on which the old road lay. Figure 16 illustrates the rock fall of January 31, 1950, which is one of several which have blacked this road in the past. This sandstone ledge is about 30 feet thick and is underlain by about 12 feet of shale between the road level and the base of the sandstone. Joints and mud seams in the sandstone together with weathering of the underlying shale have permitted the development of this fall. Cutting of this slope back to a point where the road would no longer be seriously threatened by falling rock will be a very expensive operation.

We have rather regular and methodical patrol systems set up to pick up papers and rubbish along the highways. Perhaps some similar patrol of the slopes above the road by an agile crew would protect us from at least some of the danger of falling rocks.

THE APPLICATION OF GEOLOGY TO HIGHWAY SUBDRAINAGE IN KANSAS

By
John D. McNeal, Geologist
State Highway Commission of Kansas

The relationship between geology and the occurrence and movement of ground water is so well-known that it is perhaps surprising to find so little use being made of geology in the solution of subdrainage problems in highway construction. Comparatively few of the very many articles which have been written on highway underdrainage describe or even mention the geology of the location. Even more revealing is the fact that of the 16 states which employed highway geologists in 1949 only 9 used them in the solution of subgrade water problems.

Of course one of the major reasons that this application of geology to under-drainage has had so little recognition in the field of highway engineering is the very newness of the field of highway geology. Even today, few states have a staff of engineering geologists large enough to handle the many underdrainage problems which most states have. A second reason is undoubtedly the apparent obviousness and deceiving simplicity of highway subsurface water problems. Certainly, water is not an unfamiliar or difficultly recognizable substance and its removal or control by subdrains has been practiced almost as long as the practice of engineering or for that matter, agriculture has existed. Why is geology necessary to provide such an obvious answer?

In the little more than ten years during which we of the geologic staff of the Kansas Highway Commission have studied subdrainage problems, we have discovered the following answers to that question, at least in so far as it applies to our state.

We have found that:

1. Subdrains installed without regard to geology but located in wet or seepage areas noted either on preliminary surveys or uncovered during construction

are very often placed in the wrong location or remove only part of the water. Such drains do not prevent failures.

2. Subdrains are most economically installed if they can be shown at their proper location and depth on the engineering plans and included as a bid item, preferably in the grading contract. This is not possible if one must wait until springs are discovered by excavation before designing the drain.

3. Under Kansas conditions as many as 70% of the subsurface water conditions which become evident as failures in the finished road occur where no free water could be detected during the preliminary survey or during excavation. Nevertheless, such locations can be predicted in advance of construction by the proper application of geology.

4. Prediction of troublesome subsurface water conditions in advance of construction often permits their control by means less expensive than subdrainage.

5. Subdrain designs fitted to the geology of the area generally permit a reduction in the number of feet of drain required.

Fifteen years ago subdrainage was a subject seldom discussed by personnel of the Kansas Highway Commission. Such discussions as were held will not bear repeating. Mr. L. L. Marsh,[1] formerly Engineer of Maintenance, Kansas Highway Commission, in a report to the Committee on Maintenance and Equipment of the AASHO described past performance of subdrains in Kansas as follows:

"In the past we have attempted to install various types of underdrains by a hit-or-miss method. This method is not satisfactory as we have several types of installation throughout the State where one of ten will accomplish the desired results."

"At this time we are concentrating on study of the necessity of drains and have turned that work over to our Geology Department. We have confidence that future results will be much better, due to the time and study given each special problem. Studies by the Geology Department in connection with studies by our soil experts are necessary. We have at last found this out."

Proof of the above statements can be found in performance. In the period since 1945 all subdrains installed on the Kansas Highway System have been the result of geologic investigations and have either been designed by members of the geologic staff or in accordance with their recommendations. Projects constructed during this period have been almost entirely free of failures caused by subsurface water. Of the hundreds of underdrain installations made not more than half a dozen have failed to function in preventing road failures. Prior to the application of geology to subdrainage it is doubtful that half of the installations were beneficial. Later investigation has shown that many older drains actually were detrimental. In addition, we have found numerous failures on older projects which are due to subsurface water but for which no drainage was provided.

The special study which Mr. Marsh mentions of each particular problem still continues. We find it helpful to approach each project and each location as though it were distinct and unique from all others. However, we have found that it is possible to fit most subsurface problems into a general system of classification.

Actually, of course, we might use several systems of classification. One major division of subsurface water problems could be made between those which are associated with true water table conditions and those which occur where no water table exists. The latter examples would include many of the cases where temporary flows of gravity or vadose water enter the road structure as well as many of those aquifers which occur where the upper limit of saturation falls in impermeable rocks. Such a classification has its chief usefulness in distinguishing between those cases where draw-down or agricultural subdrainage would best apply and those for which interception subdrainage generally provides the best solution. This system, however, is not too applicable to Kansas conditions where no more than 5% of our water problems are associated with true water table conditions.

Other major categories could divide those situations in which the source of trouble in the road structure arises from free water from those where water under tension is responsible. Here again, the system also provides a division on the basis of treatment. Where free water feeds directly into the subgrade, drainage will usually be required. The treatment for water under tension (that is, capillary water, etc.) may often be effected by a raise in grade, increased base and surface strength or various types of capillary breaks.

The system of classification which best fits Kansas needs is one based on occurrence. As major subdivisions we have

I. Bedrock aquifers

II. Mantle aquifers

III. Induced water problems

Bedrock aquifers in Kansas may be divided according to lithologic type into four principal groups: limestones, sandstones, shales and coals. The seepage from each of these rock types may be further described as permanent, seasonal, or ephemeral depending upon the duration of flow. While perhaps two-thirds of our water problems would fall some place in this classification, most of these occurrences of subsurface water are familiar and merit little discussion in themselves. We would like, however, to use a few actual examples of limestone, sandstone, and shale aquifers to illustrate the principal types of underdrain installations which we use. Figure 1 shows a typical limestone aquifer. In this view, this seepage must be classed as seasonal because the flow shown exists for only a few weeks of the year. Note that the movement is confined to the relatively thin zone of the limestone. If it had been possible to show both sides of the cut, it could be seen that the flow is directional as no seepage has been detected from the other backslope.

The predominance of these aquifers exhibiting movement which is generally downdip, but may occasionally be updip, makes possible interception of the flow either by pipe underdrains or by blanket underdrain. The second figure illustrates the principal features of the pipe interceptor underdrain. Here the water is moving in the limestone in a direction parallel to centerline. The intercepting trench is cut entirely through the aquifer and one foot into the impermeable shale beneath. Thus, the water is cut off well back of the intersection by grade of the water-carrying zone and no subgrade softening can occur. Figure 3 shows the actual installation of a drain of this type. The second type of interceptor, the blanket underdrain, is used for thicker water-carrying zones which could not economically be cut by a single trench. It is illustrated in the next figure (Figure 4). In this case water moving along each of the shale breaks in the limestone is collected by the 12 inch blanket of selected, permeable aggregate and led to the pipe longitudinals and laterals at the edges of the blanket. Complete interception is again provided by cutting the end lateral well into impermeable material below the lowest water-carrying zone. These blanket drains have proved quite successful for the type of aquifer shown of for thick sandstones such as the one being intercepted in the next figure (Figure 5) which shows the construction of a blanket underdrain.

Frequently, a seepage area may require a combination of pipe interceptors and blanket underdrain. Figure 6 is of an example from our files. Here the principal water-bearing zone is the thick cherty limestone over which the blanket has been installed. Normally the shales and shaley limestones which occur below the cherty limestone do not bear water. At this location, however, weathering has opened the joints and spread the bedding planes in these underlying horizons. Therefore at some distance back of the outcrop part of the water carried by the cherty formation escapes downward through the shales to spread out again toward the surface wherever resistance to its downward movement is encountered. Not until it reaches the shale, labeled 12 on the figure, does its downward movement stop. Therefore, we have used a series of overlapping laterals below the end of the blanket to complete the interception. The lowest of these "stair-step" laterals, you will note,

FIGURE 5

Installation of a blanket underdrain over a water carrying sandstone.

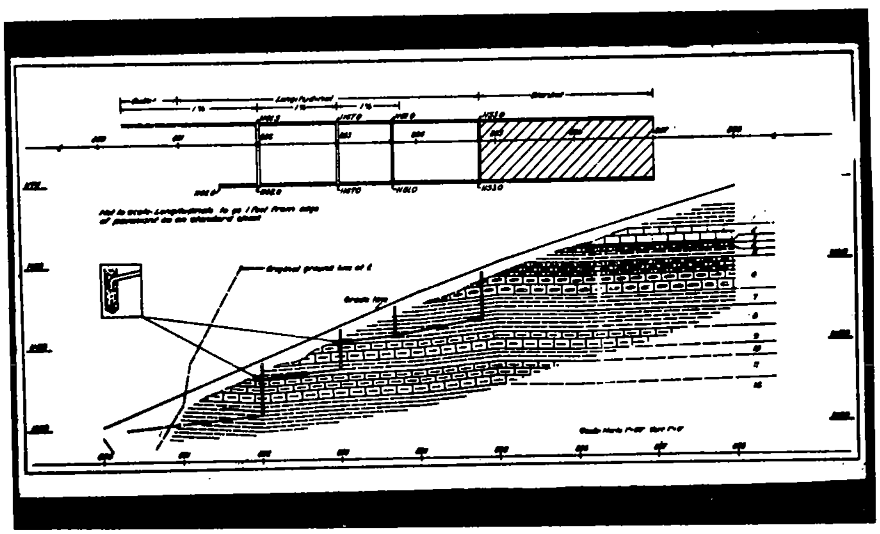


FIGURE 6

A combination of pipe and blanket underdrains often proves most economical where the water is under very little head and shale intervals separate aquifers of varying thickness.

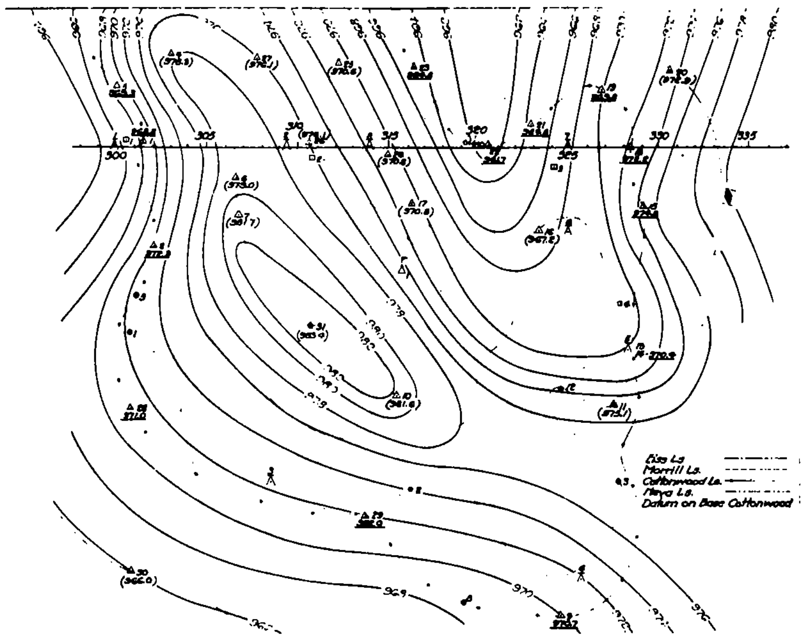


FIGURE 7

Structural contour map along a proposed improvement. These maps enable the geologist to determine the catchment area for an aquifer and the direction of flow of the water.

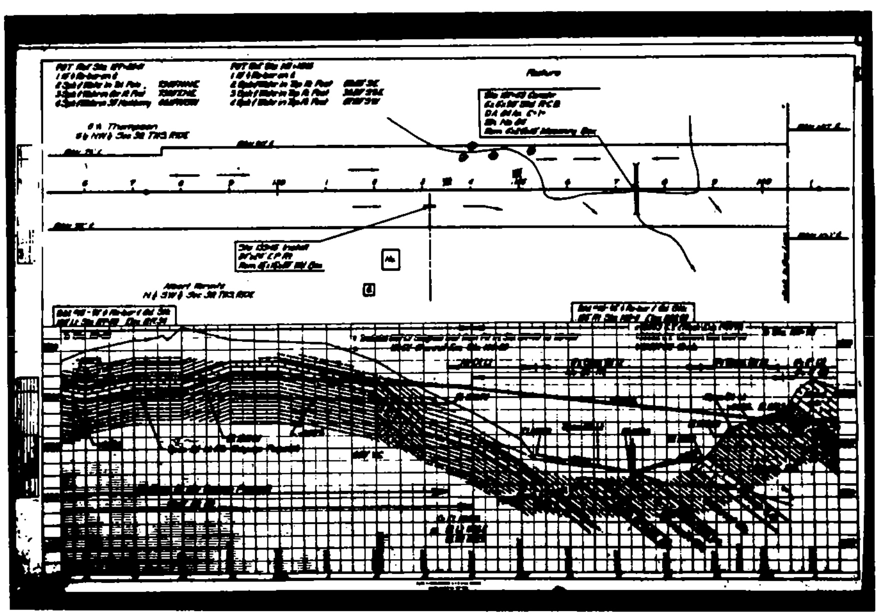


FIGURE 8

Standard plan and profile sheet from the files of the Kansas Highway Commission (slightly modified for purpose of illustration).

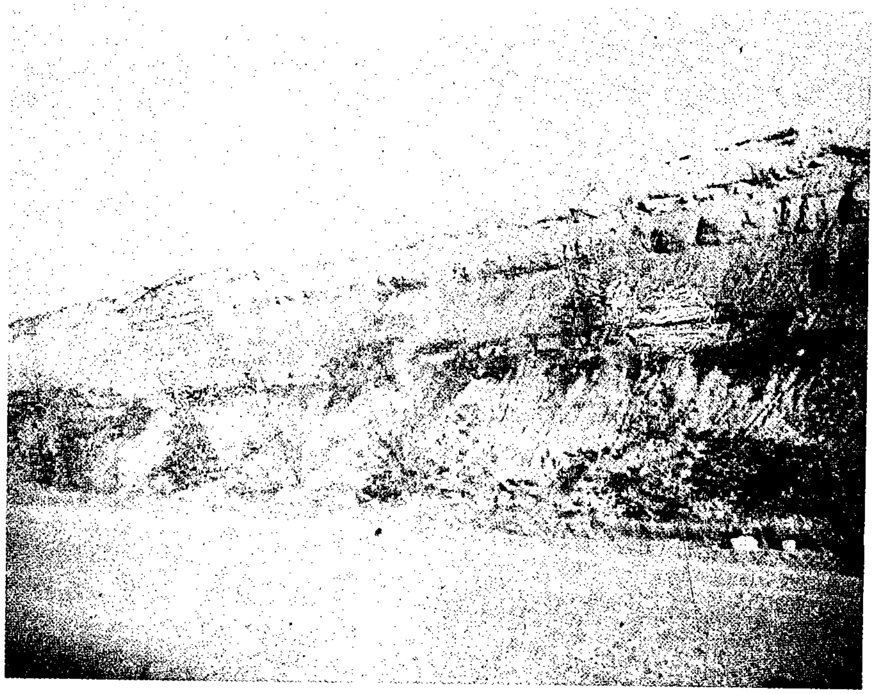
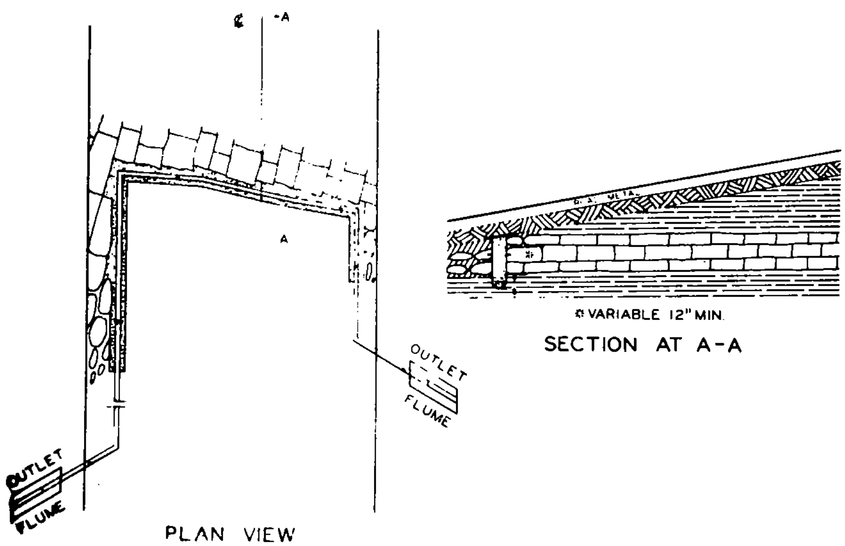


FIGURE 1

Typical seasonal seepage from a limestone aquifer, Lower Pennsylvania rocks, Wabaunsee county, Kansas. The water movement is confined to the darkened zone above and below the limestone in the center of the figure.



PIPE UNDERDRAINS
(DIAGRAMMATIC)

FIGURE 2

Pipe interceptor underdrain used in Kansas for cutting off thin aquifers such as that shown in figure 1.

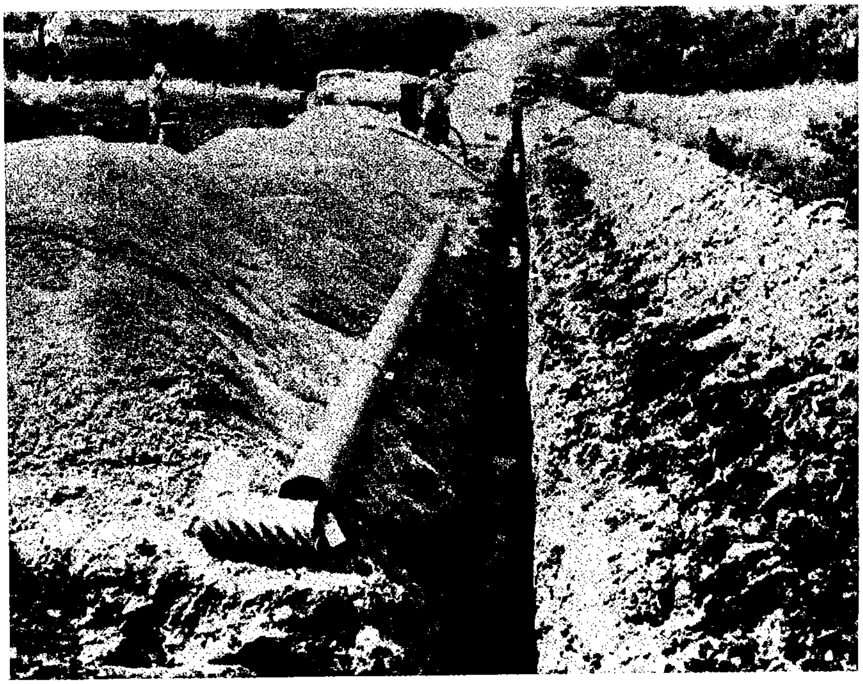


FIGURE 3

Installation of a pipe interceptor underdrain on a Kansas highway.

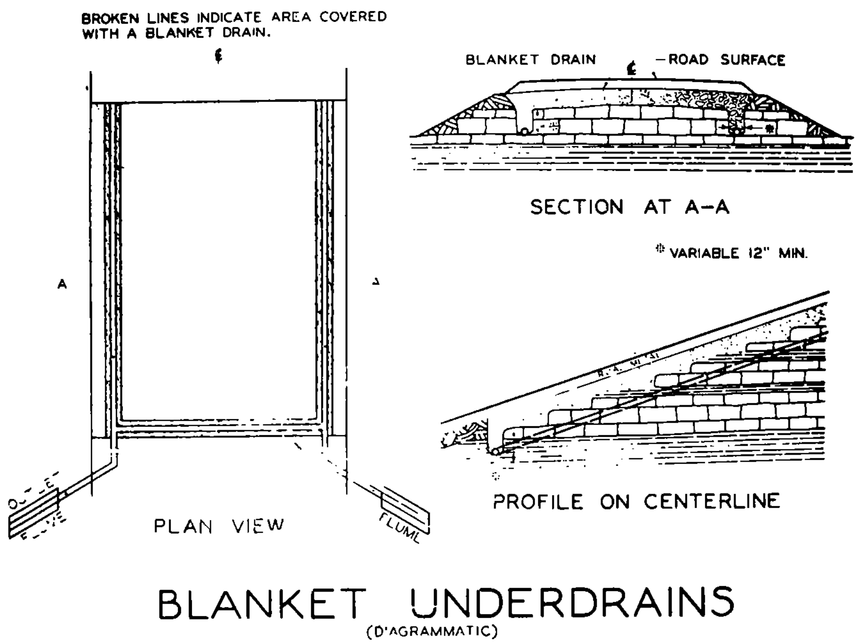


FIGURE 4

Blanket underdrains are used for intercepting water moving in thicker aquifers such as sandstones or the limestone-shale sequence shown.

is cut into the unweathered shale on zone 12.

The recognition of conditions such as that just described and the identification of seasonal or intermittent aquifers are among the most difficult problems which face the highway geologist making a subsurface hydrology study. A successful solution to these problems is necessary if drain failures and road failures are to be prevented.

Their solution requires a knowledge of botanical taxonomy and ecology, weathering processes, animal habits, meteorology, hydrology and petrology, as well as general geology. Fortunately, common sense and keen observation may be used in place of the formal knowledge mentioned above. Just as one may predict the lithologic factors of a covered formation by study of surrounding exposures and the general geology of the area, so one may predict the hydrologic characteristics. Basic to the entire problem, of course, is a detailed knowledge of the geology of the project. We are very fortunate in Kansas in having a sufficiently large geologic staff to prepare a detailed picture of the geology of every project. Thus, before conducting the subsurface hydrologic study, the geologist has the basic information shown in the next three figures.

First, (Figure 7) he has a structural contour map of the area along and adjacent to centerline. From this map and other data, a geologic profile (Figure 8) along centerline is prepared which shows not only the attitude and lithology of all formations but also their relationship to the grade of the proposed improvement. Finally, to complete a three dimensional picture of the underlying geology (Figure 9), the geology is shown on the cross sections included with the engineering plans.

If one knows the formation which will be cut by the grade line at a particular location and its dip, it is then simply a question of determining the water-bearing characteristics. Lacking visible seepage as evidence of <u>permanent</u> seepage, the geologist must rely on other signs to predict <u>seasonal</u> seeps. Among the signs to be observed are:

1. Vegetation. Either the species or the rankness of growth may be a valid clue, but one must be able to identify the dead winter plant as well as the living plant in the summer. Plant indicators will vary with the locality and should be determined by preliminary reconnaissance. In Kansas young cottonwood trees are a useful indicator in many areas. Figure 10 shows an example. The line of cottonwoods marks the base of the Little Kaw limestone. This limestone exhibits visible seepage for only a few weeks of the year in this area, but the amount of water is sufficient to cause failures.

2. Evaporites (Figure 11) are often an important indication of seasonal seeps.

3. Deeply impressed cattle tracks in an area where there is no evidence of the ponding of surface water also indicate seasonal seeps. On the other hand, animal burrows in or below a suspected horizon may prove that it does not carry water.

4. In test holes and at outcrops the presence of certain secondary minerals and concretions may indicate water movement. In shales subsurface water weathering produces softening, changes in color and changes in structure which may be detected even when no water is moving.

5. When existing ro ds are to be resurfaced or otherwise improved, a condition survey will give reliable indications of all types of seepage. Similarly, condition surveys of other roads in the area when correlated with geology give useful information.

All such signs must be used with caution. Not all road failures can be attributed to subgrade moisture; and rank growth of vegetation may be associated with soil type as well as with soil moisture.

The signs of seasonal seepage become even more important when we consider mantle seepage, for relatively few mantle aquifers carry permanent flows. We use the term mantle in connection with seepage to include not only residual soils and

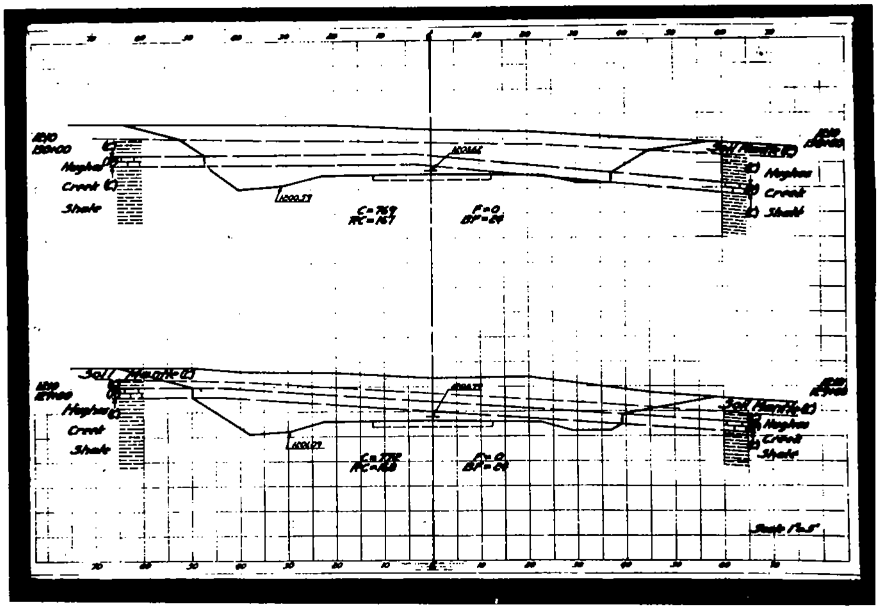


FIGURE 9

Standard engineering cross sections. Geologic information on every Kansas highway project is furnished to the designer and the contractor in the manner shown in this figure and in figure 8.

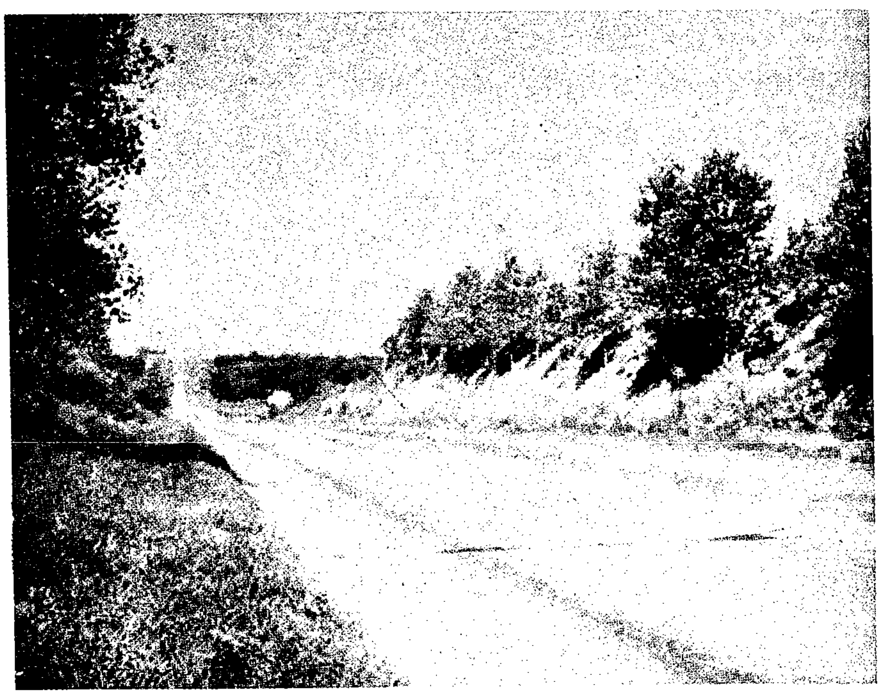


FIGURE 10

The line of cottonwood trees along the backslope marks a seepage zone which seldom shows a movement of free water but has proved damaging to roads. Trees and other vegetative indicators are very helpful to the geologist in detecting seasonal seepage.

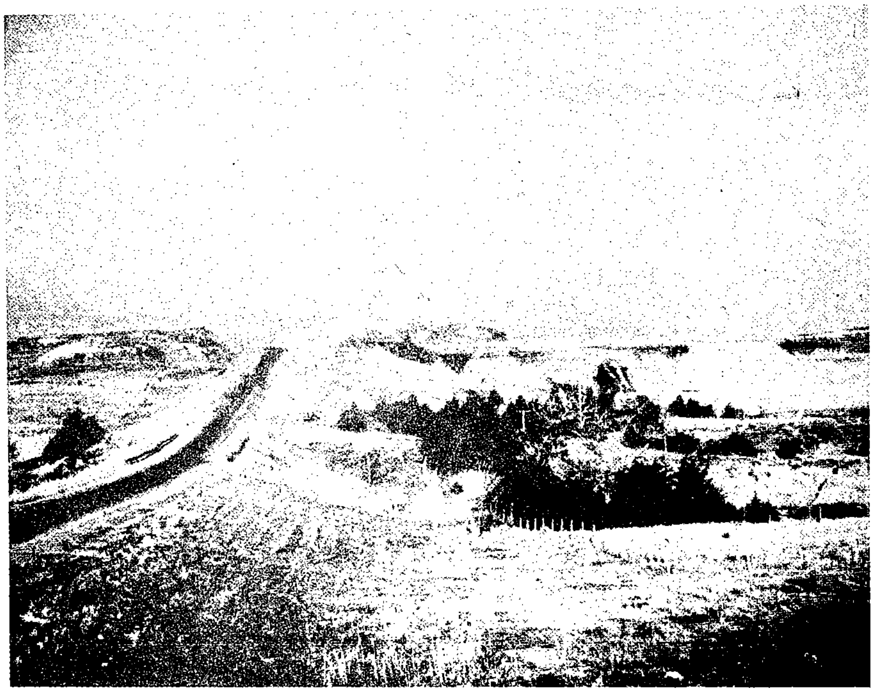


FIGURE 11

The irregular lines of white salts seen at right center of the figure are useful indicators of seasonal seepage in the Kansas Permian Red Beds.

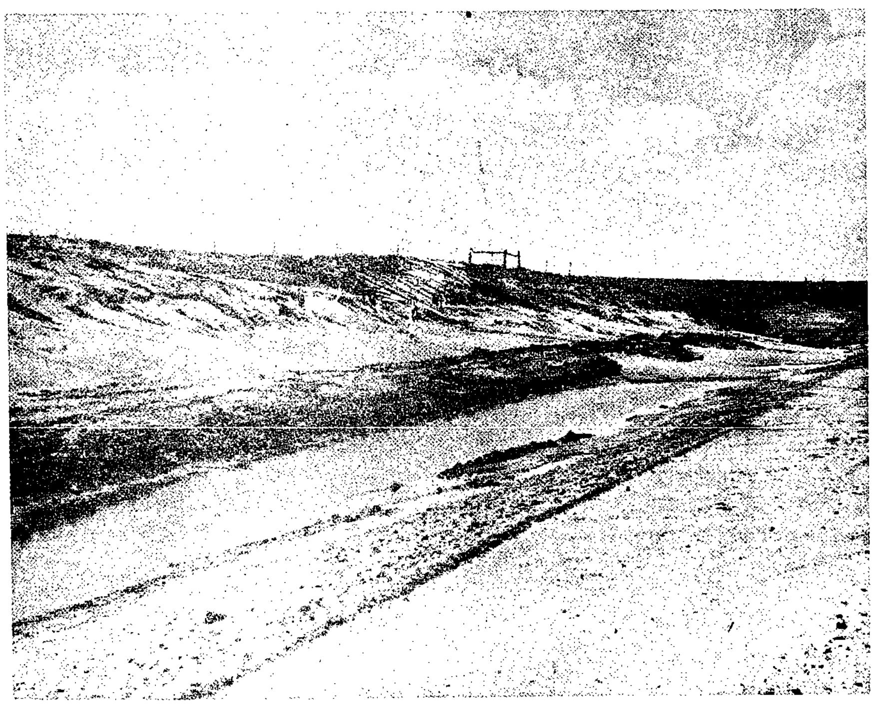


FIGURE 12

Permanent seepage from the contact between unconsolidated Pliocene sediments and Permian Red Beds. Interception at this location will be provided by the ditch which is cut well into the relatively impermeable Red Beds.

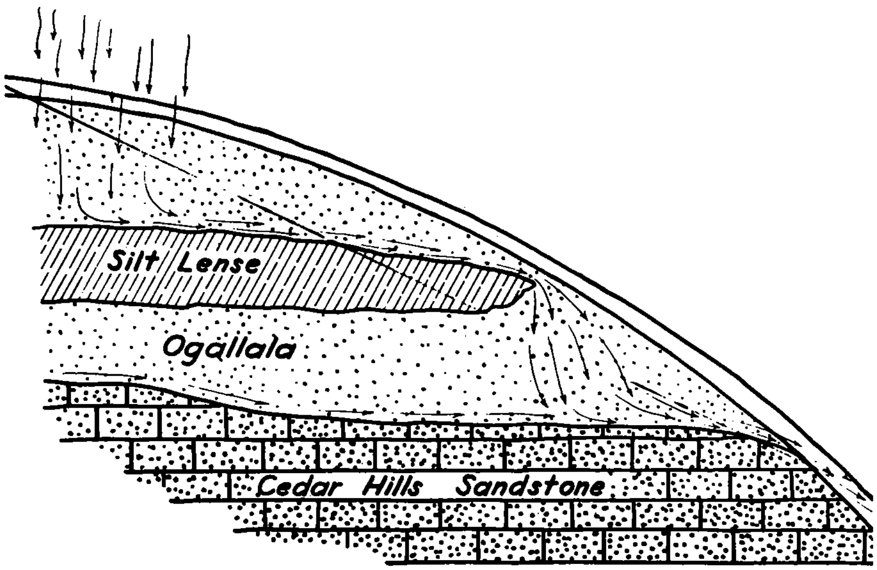


FIGURE 13

In unconsolidated sediments silt and clay lenses decreased permeability deflect and concentrate the downward movement of subsurface water giving rise to contact seepage. The flow of water is often of very short duration.

FIGURE 14

Failure in a sand-gravel county road caused by contact seepage.

FIGURE 15

Contact seepage at a weathering contact in consolidated rocks (Greenhorn formation, Cretaceous).

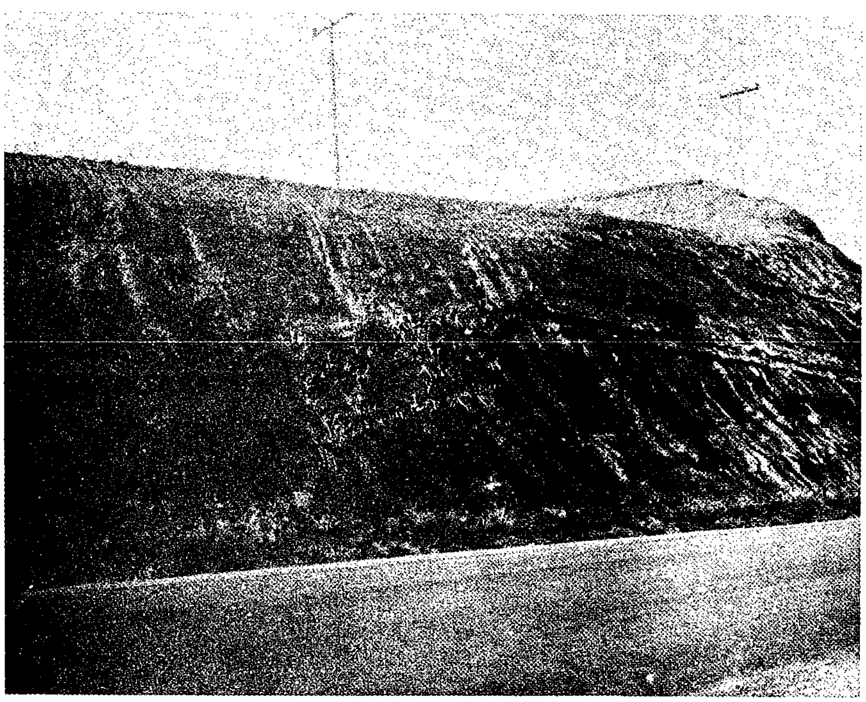


FIGURE 16

Buried erosional scar in Permian Red Beds of southwest Kansas. Many of the highway problems in this area are caused by water moving under the road in such scars.

recent deposits but also all unconsolidated or partially consolidated sediments whether of Recent, Pleistocene, or even Tertiary age. In this mantle the water which is encountered in road construction in Kansas except in the deepest cuts is vadose water or soil water. Therefore, the quantities found vary closely with the periods of precipitation and with evaporation and transpiration. In general, any zone regardless of its own permeability may become the lower boundary of an aquifer if its permeability is less than that of the material above. In the mantle one must give particular attention to structural permeability as well as to textural permeability.

Contact seeps most commonly occur at the mantle-bedrock contact. Figure 12 shows the strong seepage that is frequently encountered at the contact between unconsolidated Pleistocene and Pliocene sediments and the underlying Permian Red Beds in south central Kansas. In this case, interception will be provided by the ditch. A similiar condition is seen in the diagram of figure 13 where an additional contact seep is found on the top of one of the silt lenses so common in the sandy Pliocene sediments of the western third of the State. Even though the movement of water shown in Figure 13 persists for only a few days after heavy rains, the results can be clearly seen in the next slide (Figure 14).

As it is seen from the next figure (Figure15), contact seeps can occur where the contact is simply between weathered and unweathered rock. This condition is common in the chalky Cretaceous limestones and shales of northwestern Kansas where the depth of weathering is indicated by a change in color from white or tan to dark blue.

The buried erosional scar, such as is seen in the backslope of Figure 16, is only a variation of the contact seep. The particular scar seen here would not have been difficult to detect in advance of excavation since the Permian Red Beds into which it was cut are exposed on the surface on either side of the small buried valley. In the next figure, Figure 17, there is no indication of the scar at the surface. Becuase of its small size it would have been luck if this feature had been detected by test drilling. The loess-limestone contact was essentially dry except in this buried depression. In areas where such scars are known to exist, a constant depth, electrical resistivity traverse will often reveal their presence. Treatment at this location was simple. A single longitudinal interceptor on the upstream side was the only drain required.

In the Pleistocene loess deposits which mantle much of the northern and western sections of Kansas, seasonal seepage is frequently encountered at the top of the Sangamon buried soil. Figure 18 will illustrate this condition. The dark bank across the center of the exposed slope is a vegetation line marking the buried soil. We have never found seepage along the outcrop at any time of the year; however the known hydrologic characteristics of this buried soil guided the location of test holes well back of the exposure. These tests did find a high moisture level at the top of the old soil. A close-up (Figure 19) of the location shows the result when it was decided that drainage was not economically advisable. At the location shown interception by a lateral drain would have been easily accomplished. Generally the topography of these old soil zones follows the present topography and, thus, they are not intersected by the grade line but will be found in the backslope as a zone paralleling the ground surface. Figure 20 illustrates the result.

Induced Water Problems

Buried soils, even tight B horizons of modern soils, may also become a factor in induced water problems. By induced water is meant that which is introduced into the road structure as a direct result of the construction. Even where conditions of rainfall, evaporation, and transpiration are such that a zone, such as a buried soil, does not carry water before construction, a flat ditch may serve as a catchment area to turn the zone into a aquifer. Flat or plugged ditches are the most common but not the only source of induced water.

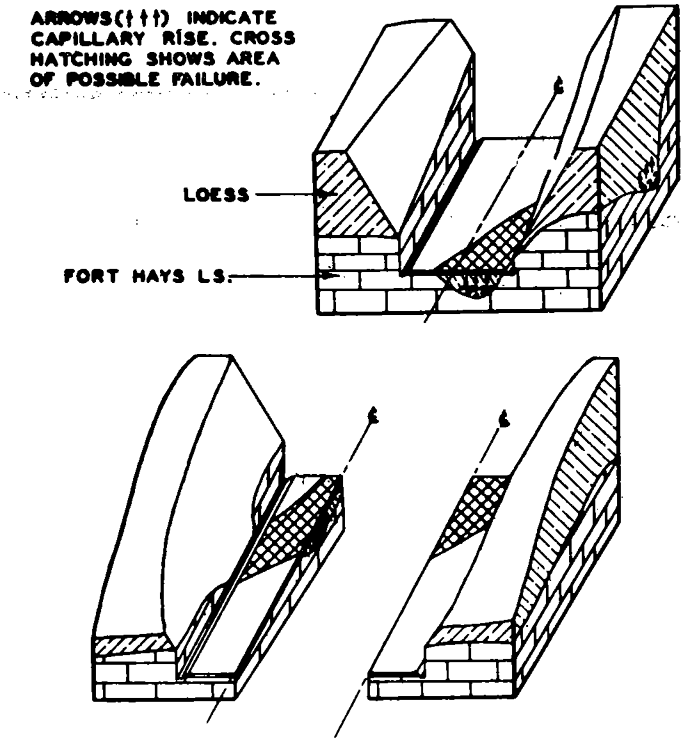


FIGURE IV
DIAGRAM SHOWING RELATION BETWEEN POSSIBLE FAILURE AND BURIED EROSIONAL SCAR.

FIGURE 17

Deeply buried, loess filled erosional scar.

FIGURE 18

The vegetation **line near** the center of the cut bank marks the position of a buried soil. No seepage is found at the outcrop but the soil zone controls the movement of sub-surface water. See figure 19.

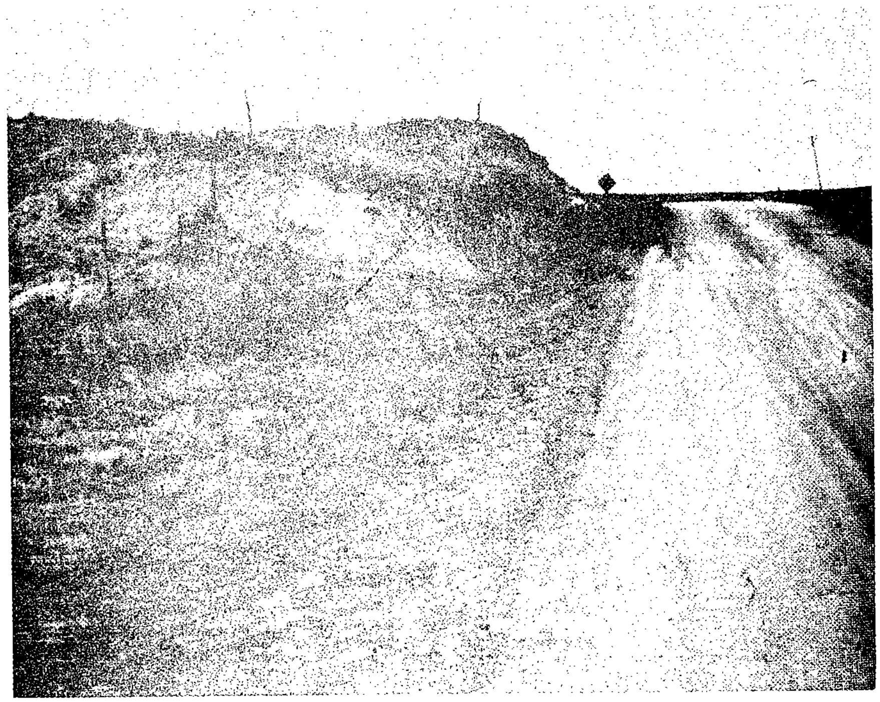


FIGURE 19

The road failure occurs above the buried soil zone seen in Figure 18. Back of the outcrop water is encountered at the top of this zone in wet seasons.

FIGURE 20

The backslope slide was caused by water moving along a buried soil zone which occurs near the middle of the backslopes.

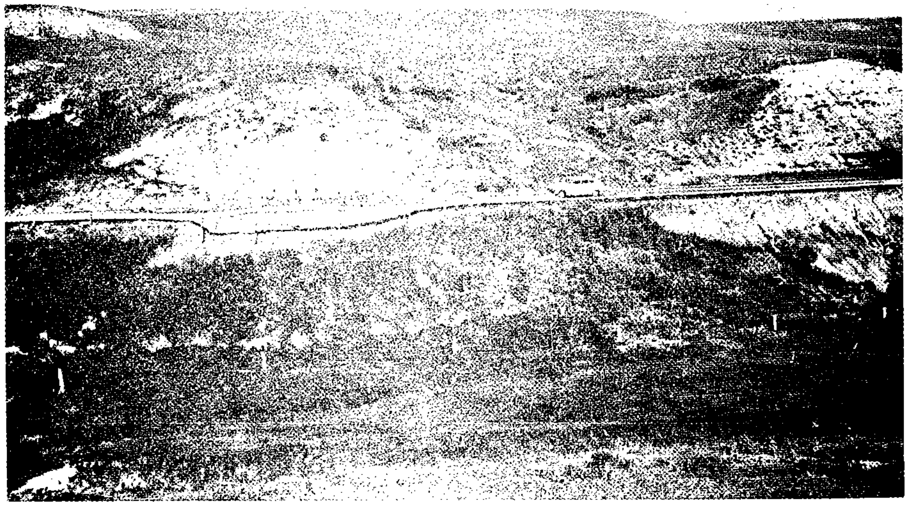


FIGURE 21

This fill slide is the result of induced water from ponded ditches. See Figure 22.

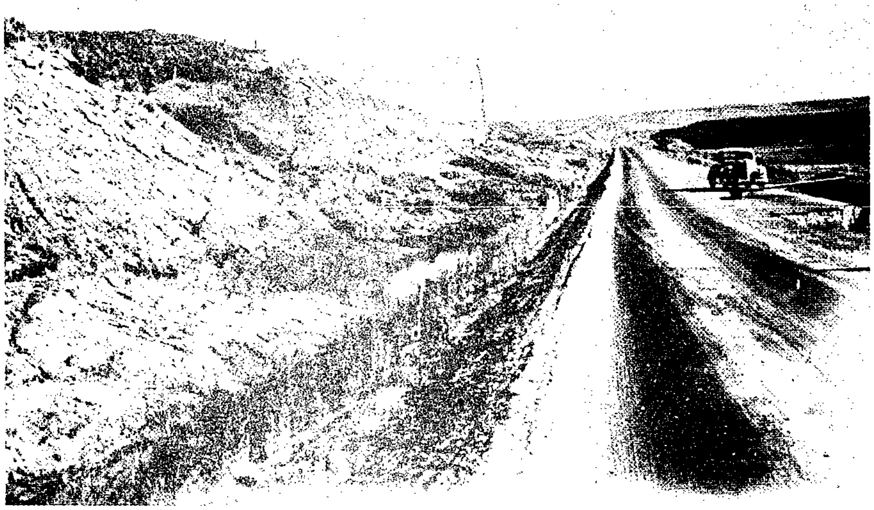


FIGURE 22

Flat ditches, unstable backslopes and seepage zones in the backslope combine to cause ponded ditches. Induced water from this ditch has been responsible for the fill slide seen at the extreme right of the photograph.

The fill slide seen in Figure 21 is a good example of a result of induced water from flat, ponded ditches. Note the strong, seasonal seepage which feeds into the ditch from at least two higher zones.

A closer view (Figure 22) will bear this out. Slumped material from the backslope has plugged the ditch at a point opposite the car and the ditch gradient is too flat to promote self-cleaning. Problems of this type cannot always be predicted in advance of construction but many of them can be foreseen. In this particular example the presence of strong seepage above the road level, coupled with the flat grade and known performance of the backslope material, should have led the geologist to recommend both special ditches and backslopes as a preventative. These measures were effected by the maintenance department and have proved successful.

Other induced water problems may result from overbreakage. If a four or five foot massive limestone is cut in the upper foot by a flat grade line which parallels the dip of the bed, the result is essentially that of digging a canal and then constructing a highway down the center of it. Water from either ditch will move into the lower area of excavation under the center of the road and softening by capillarity soon results in road failure.

Overbreakage is a predictable engineering-geologic characteristic of a formation. Therefore, the geologist is able to furnish information to the designer which will permit either a change in grade line to avoid both the heavy overbreakage and resulting water problem or the installation of protecting longitudinal interceptors if a change in grade is not feasible.

Whether the particular subgrade moisture problem with which one is faced is the result of induced water or arises from bedrock or mantle seepage, its solution will be most effective if based on a detailed knowledge of the geology of the location. Certainly, this statement is true for Kansas conditions. It is believed to be equally true for other climatic and geologic provinces. The particular problems discussed here will not be common to all state highway systems. For example, many of the subsurface water conditions which can be observed along the Chesapeake and Ohio railroad west of Charleston appear quite dissimilar to those found in Kansas and their correction might require a different type of subdrain installation. For any example, however, the relationship between subsurface water occurrence and the geology of the bedrock or mantle rock of the location is an established fact. Designs based on this fact will be most effective.

Reference

1 / Worthington, E.L., Performance experience of subdrains under and adjacent to pavements. Report of Subcommittee No. 4 of the Maintenance and Equipment Committee, American Association of State Highway Officials. No date given.

SOME ENGINEERING PROBLEMS ASSOCIATED WITH THE PREGLACIAL MARIETTA RIVER VALLEY

By

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INTRODUCTION

The engineer is often called upon to give counsel on problems involving an understanding of geologic phenomena. In recent years the recognition of ancient and often buried valleys has taken on important engineering significance. These valleys have long been recognized by the earth scientist; however their significance and importance to engineering works has not always been apparent. In turn, the engineering investigation frequently develop important data which can be used to supplement and detail the available information.

The character of the sediments in old valleys has important bearing upon the foundation conditions for heavy industrial sites, buildings, dams, highways and railroads. In addition, these sediments may be important in connection with water-supply needs - particularly when such valleys occur in rock areas where high-grade, water-bearing strata are not commonly encountered.

It is the purpose of this paper to review the literature on the ancient Teays and Marietta Rivers and to present certain data which show the general engineering characteristics of the Marietta River Valley in at least one location. It is a further purpose to illustrate a correlation between some geologic and engineer-data which were used to obtain the most satisfactory solution for a specific problem involving the design of a water-supply reservoir.

ACKNOWLEDGEMENTS

The authors wish to express appreciation to the City of Jackson, Ohio, the Jackson Iron and Steel Company, and the firm of Jones, Henry and Williams, Toledo, Ohio, for permission to use the engineering data contained in this report.

Particular credit is due Mr. L. G. Williams for his untiring efforts in helping obtain much of the foundation information and for help in the planning of the exploration programs.

GEOLOGIC CHARACTERISTICS OF THE TEAYS AND MARIETTA RIVER VALLEYS

Of the many ancient and buried valleys, none is more interesting and complex than that developed by the preglacial stream known as the Teays River; more recently referred to as the Mahomet-Teays (38). Geologists have been aware of the existence of parts of this ancient valley for over a hundred years. One of its earliest recordings was by Dr. S. P. Hildbreth (13) who in 1838 described fossil, fresh-water shells found in coarse sand and gravel beds in Barlow Township, Washington County, Ohio, and postulated their origin as a ponded valley, now extinct. In the early eighteen nineties, Leverett and Foshay in describing the drainage systems of the Mississippi River and its tributaries noticed many pecularities inconsistant with the general pattern of drainage development (21,9). They attempted to explain these on the basis of drainage systems existing prior to the "Ice Age".

Meanwhile, two men were gathering information in two widely separate sections of Ohio which led to a confirmation of the work of Leverett and Foshay. From field studies in west-central Ohio and eastern Indiana, Bownocker (1) postulated the existence of a preglacial channel based on his studies of the thickness of glacial drift. Working in southeastern Ohio and adjacent West Virginia and Kentucky, Tight (31) recognized and studied a series of highlevel, alluvial deposits within broad valleys having underfit streams or no streams at all. These observations together with available water-well data formed the basis for his theory of the precursor of the upper Ohio River valley.

The work of Tight and Bownocker has been further strengthened by the more recent studies of Happ (12), Stout (25), and Ver Steeg (34) who have been able to collaborate and extend the earlier ideas through the use of better equipment and more complete data.

As late as 1943, the lower reaches of the Teays were believed to coincide with the lower Wabash Valley of Indiana (7), but in 1946 Fidlar (6) discovered evidence near Lafayette, Indiana, which pointed to a westward continuation of the old valley from Tippecanoe County, Indiana, to eastern Illinois. This has since been correlated with the work of Horberg (14, 15, 16) in Illinois and the drift studies of Wayne and Thornbury (39), and McGrain (23) in northern Indiana so that it now appears that the preglacial Mississippi River was a tributary of the old Teays or Mahomet-Teays as Wayne (38) prefers to call it.

It is interesting to trace the course of this ancient drainage artery, reconstructed by these men and point out some of the important aspects with respect to engineering. The accompanying map (Fig. 1) is not complete in detail but will serve for orientation purposes.

The headwaters of the Teays River are believed to have risen in a portion of the Piedmont Plateau in northwestern North Carolina and western Virginia. Its upper reaches appear to correspond somewhat to the present New, Gauley, and Kanawha Rivers, to which it adheres through south-central West Virginia. Near Charleston, West Virginia, the Teays depart from the course of the present Kanawha River, to cut across western West Virginia and thus join the present Ohio River at Huntington. The old valley corresponds to the present Ohio River valley from Huntington, West Virginia, to Wheelersburg, Ohio. At this point its course swings sharply to the north to cross southern Ohio to a point near Chillicothe. Here the surficial evidence of the valley ceases as it passes beneath the terminal moraine of the Pleistocene ice sheets. A major tributary of the Teays was the Marietta River. It headed near New Martinsville, West Virginia and joined the Teays a few miles west of Jackson, Ohio. See Figures 1 and 2.

The upper portion of the Teays valley, just described, may be observed with little difficulty. From its headwaters to Charleston, West Virginia, extremely deep channels have been cut. This is particularly true in the vicinity of Gauley Bridge, West Virginia. If this portion of the old valley has ever been filled to an appreciable degree by water-deposited materials as is known to be true from Charleston to Chillicothe, there remains little if any evidence today. In other words it would appear that this section of the old Teays has operated almost without interuption or major change since before the Pleistocene Epoch.

Below Charleston, the course of the Teays may also be observed but in a different character. In an area of exposed bedrock hills, the valleys are broad, high, and comparatively flat, and the area bears a subtle resemblance to basin and range type of topography. In reality, these valleys were once the channels of the Teays River and its tributaries. Today they are in part filled with alluvial material to a comparatively high level.

The material which fills the greater portion of the old Teays valley from Charleston to Chillicothe is predominately a "laminated silt" (often with clay-like characteristics). These terrace sediments, known as the "Minford silts" in the Teays valley, are micaceous and in some cases are highly plastic. Locally they may be as much as 80 feet deep, though they generally average from 20 to 40 feet in depth. They are known to be radioactive (29); a point which tends to bear out their Piedmont origin. It is believed that as the early continental ice sheets advanced, the old Teays was effectively blocked or dammed below Chillicothe giving rise to a condition of ponding. These ponded waters eventually overflowed low divides and thus established new drainageways which cut valleys below the former drainage level. Later the Illinoian and Wisconsin advances added about 100 feet of outwash sand and gravel in that section of the valley marginal to the ice sheet.

The Minford "silts" contain little coarse material although occasionally thin seams of sand appear. The presence of gravels below the "silts" is far from common though locally they exist and in one case boulders from 8 to 10 inches in diameter have been recorded (32p. 52). They have that well-rounded appearance due to stream action and are firmly cemented sandstone remnants. Quartz and chert pebble gravels are also known to exist at the base of the silts but again only locally.

Sand deposits usually overlie the bedrock of the old valley floor and grade upward into silicious silts which do not appear to be related to the overlying Minford silts.

It is not uncommon in making excavations in the Minford sediments to encounter beds of muck or muck-like material. Entire tree trunks have been uncovered together with large quantities of organic matter (32 p. 59). Such material may represent organic accumulations in slackwater areas or in oxbows.

With the exception of infrequent exposures or where it coincides with present drainage, the balance of the Teays River valley lies buried beneath the drift materials of the Pleistocene glaciers. North of Chillicothe, Ohio, the valley trends northwest until it makes a broad bend which brings it into Adams County, Indiana, from the northeast. Again bending to the northwest it continues this until it crosses the Wabash River northeast of Peru, Indiana. The valley lies north of the Wabash River at this point and trends in the same general pattern until it merges with the present Wabash valley near Logansport, Indiana, and coincides with it to a point south of Lafayette, Indiana. Here the valley makes a sharp departure from the present drainage system by turning westward and passing into Illinois. Crossing Illinois in a torturous series of curves through Champaign, Decatur, and Lincoln, it joins the present Illinois River valley south of Beardstown, Illinois.

The existence of this portion of the old valley is recognized largely from the records of the many wells which have been driven in the area. More recently the assumptions based on these records have been strengthened through the use of geophysical methods such as seismic and electrical resistivity surveys.

For the most part, the valley from Chillicothe North is buried beneath glacial material varying from a few feet to as much as 450 feet. This irregularity of depth is quite striking in places. Drillers of the old Trenton oil field, southeast of Ft. Wayne, Indiana, were very familiar with these irregularities and often found the depth to bedrock in adjacent wells to vary as much as 100 feet. There is recorded in Adams County, Indiana, a vertical difference to bedrock of as much as 250 feet in two wells less than a quarter of a mile apart, (7). One may readily see the engineering problems which might arise under these conditions, e.g., securing foundations for bridges.

The buried Teays valley has been an excellent source of water supply in many localities and the deeper portions of the buried topography are commonly sought for that purpose. This is due to the general granular texture of the fill material. Commonly, coarse and fine materials are mixed heterogeneously but seams of clean gravel are known to lie on the floor of some of the valley. It should be pointed out however that the presence of a buried valley is not always a good water source. McGrain found for example that the direction of flow was significant. "...... where the former stream flows in a southerly direction coarse sands and gravels are generally present in quantities. Where north flowing, the valleys are generally filled with fine-grained sediments" (23). This is understandable when one considers the nature and cause of the filled valleys in southeastern Ohio.

ENGINEERING CHARACTERISTICS OF A PORTION OF THE MARIETTA RIVER VALLEY

Jackson, Ohio is the county seat and is located very near the geographic center of Jackson County. It is not coincidental that some of the main arteries

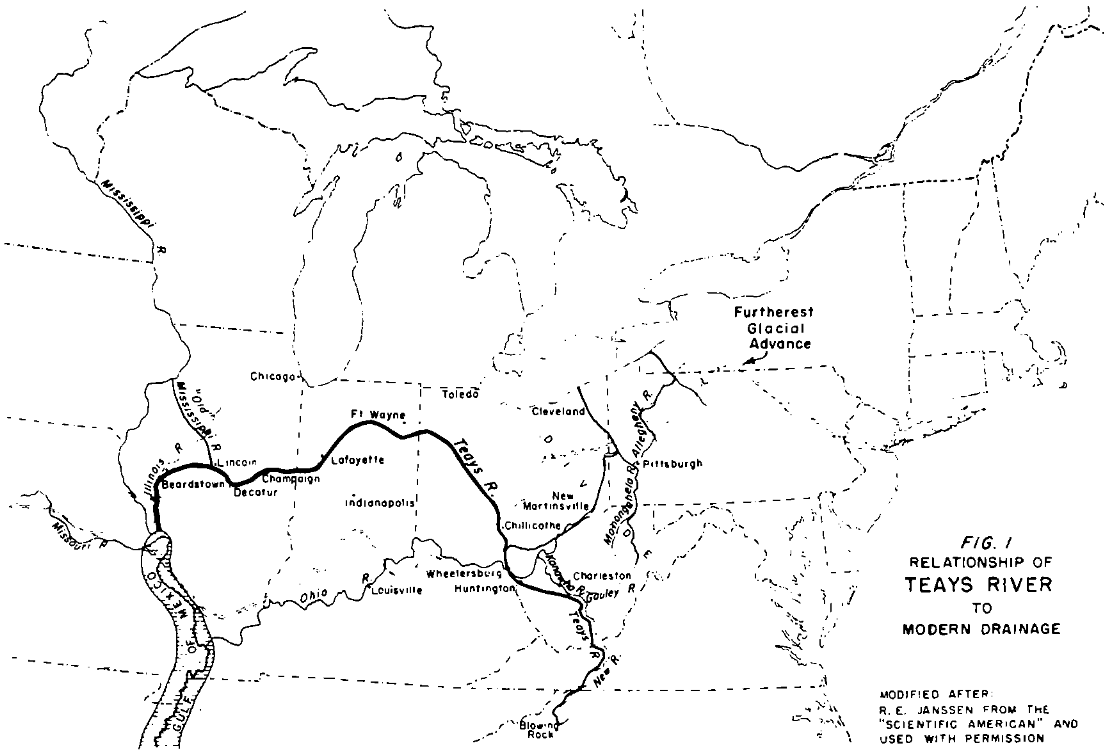

FIG. 1
RELATIONSHIP OF
TEAYS RIVER
TO
MODERN DRAINAGE

MODIFIED AFTER:
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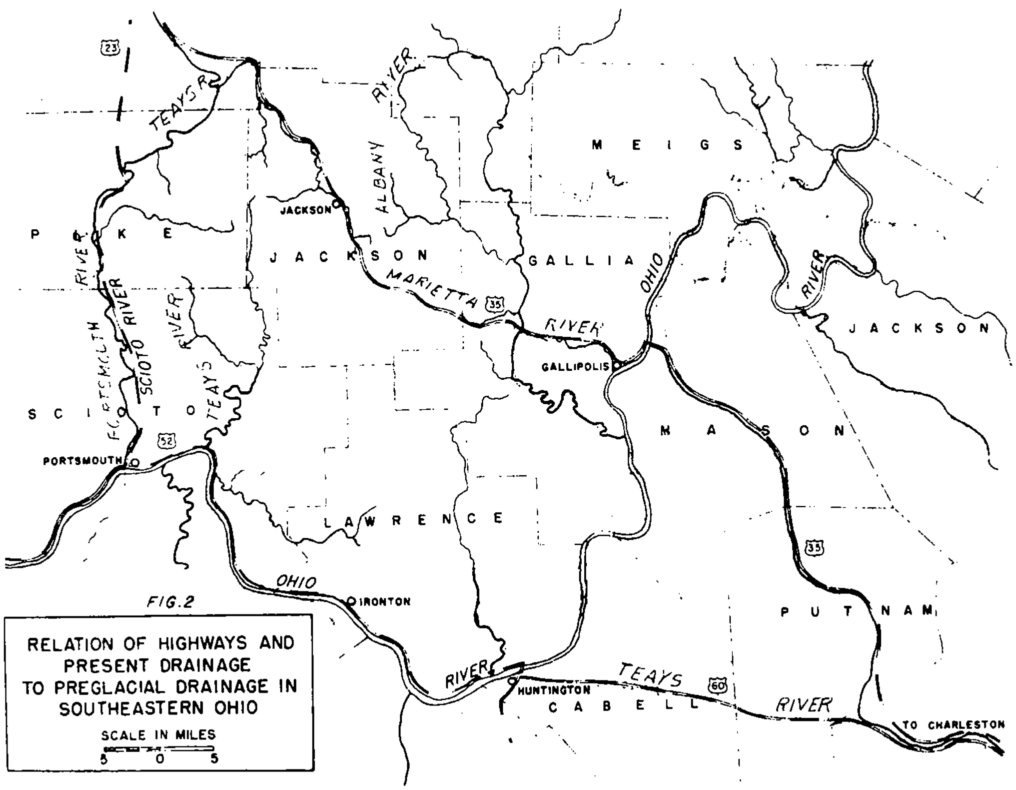
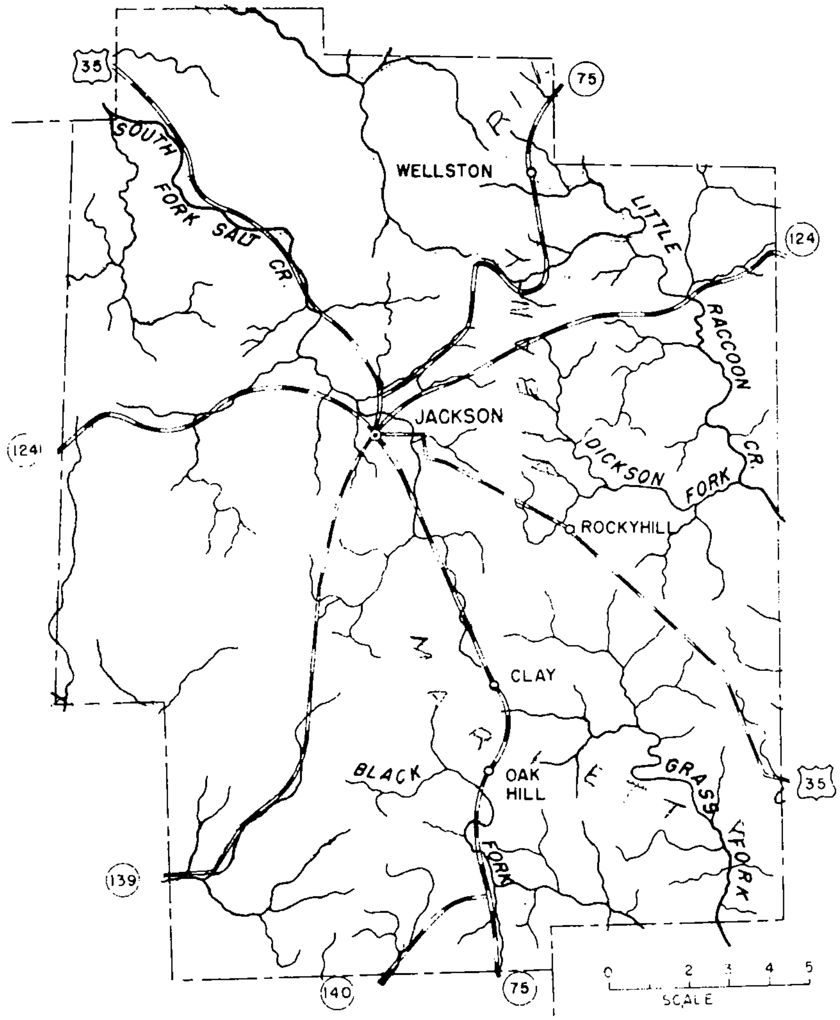


FIG. 2

RELATION OF HIGHWAYS AND PRESENT DRAINAGE TO PREGLACIAL DRAINAGE IN SOUTHEASTERN OHIO



RELATION OF PRESENT HIGHWAYS & DRAINAGE TO PREGLACIAL DRAINAGE IN JACKSON CO. OHIO

FIG. 3

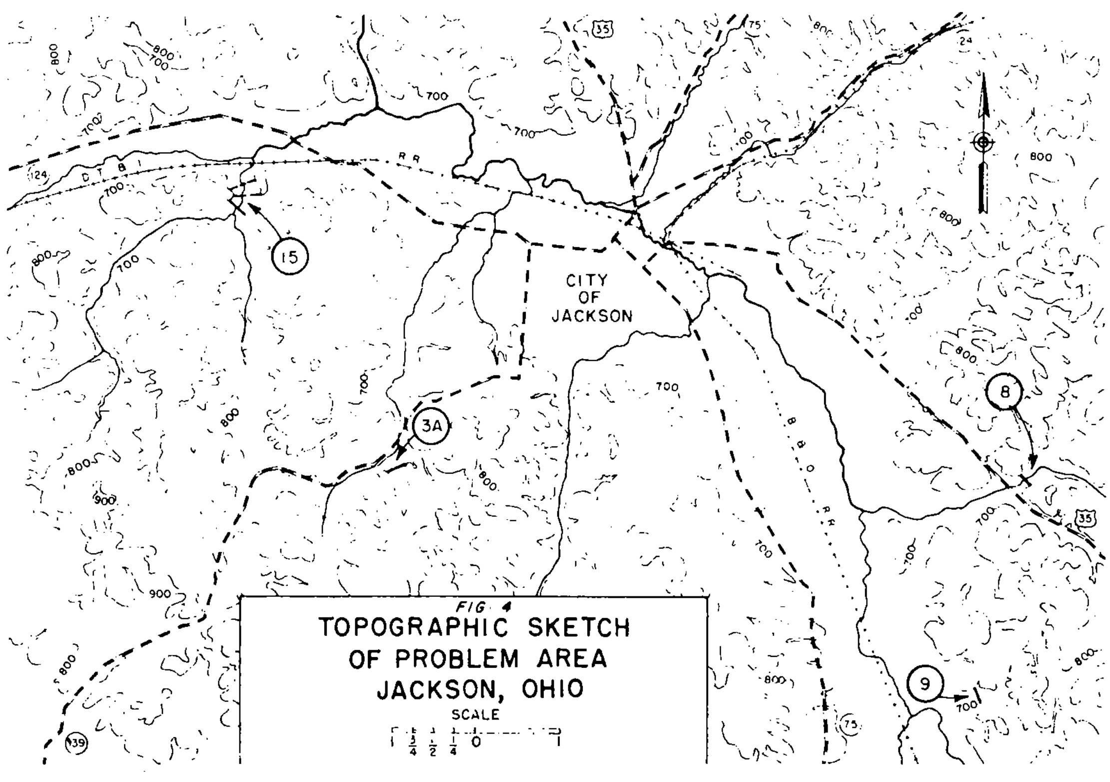


FIG. 4

TOPOGRAPHIC SKETCH OF PROBLEM AREA JACKSON, OHIO

SCALE

of transportation (the Detroit, Toledo, and Ironton, and the Baltimore and Ohio Railroads, and U. S. Highway No. 35 from Chillicothe to Gallipolis) follow, to considerable extent, the valley of the preglacial Marietta River (Figure 2). The City of Jackson is located in the center of this old valley and, unfortunately from the standpoint of water supply, at about the point where the water-shed in the valley divides, with the eastern portion draining to the Ohio River and the western portion of the old valley draining to the Scioto River (Figure 3).

The need for a more adequate water supply for the City Jackson required an engineering analysis of the soil conditions in the vicinity of Jackson, Ohio. It had been determined previously that an upland reservoir type of supply would be used. Thus it became necessary to explore various sites in an effort to locate a dam in the most favorable situation. Of primary consideration were foundation conditions, availability of borrow for the proposed embankment, and a site with optimum water storage capacity.

Explorations

Topographic maps and contact aerial photographs (40) were used in the preliminary location stages to determine the best possible sites in the vicinity of Jackson. Several potential locations were available and after some reconnaissance work in the field, exploration procedures were established to determine the characteristics of foundation materials as well as the materials immediately available for use in the construction of the dam. Various procedures were used in making the explorations for the foundation and embankment-borrow areas. Equipment used consisted of 4- and 6-inch post hole augers with pipe extensions, while a four-inch continuous screw-type power auger was used to determine the depth to rock. In addition some resistivity data were obtained (45) for locating rock elevations not only in the foundation areas but also in proposed borrow areas. In addition, a diamond core-drill rig was used for several of the deeper holes to make certain that the true nature of the underlying rock was determined.

Samples of soil from each soil horizon encountered were visually inspected in the field. In some cases visual description was the only means utilized for identifying the materials. In most instances, however, samples were obtained for laboratory identification tests.

The soft strata underlying the major portion of site 15 was further investigated to determine its strength characteristics. This was done by obtaining "drive samples" by means of a split tube sampler (41) for laboratory determination of shearing resistances. The samples were encased in paraffin immediately on removal from the ground to insure that no evaporation of the soil water took place.

Testing

Identification tests consisted of standard Liquid and Plastic Limit determinations (42). Compaction tests using standard procedures were made on samples of the terrace deposits and other soils that were considered for borrow material (43). Unconfined compression tests were utilized for estimating the shearing resistance of the soft strata (44). The shearing resistance was assumed to be one-half of the unconfined compression strength of the soil sample. Due to the character of this soft material no definite shear failures were observed in these tests and therefore the ultimate compressive strength was arbitrarily selected as the unit stress at 20 percent strain.

Results of Tests

The complete set of data collected for the Jackson project are not included as a part of this paper. However, a number of examples have been chosen to illustrate the general trend of results which, in turn, present an over-all picture of the soil conditions in the vicinity of Jackson.

Data contained in the paper have been collected from four sites, Nos. 8, 9, 3A, and 15. Figure 4 shows the location of each. Sites 8 and 9 are located southeast of Jackson on the eastern side of the preglacial valley, while Sites 3A and 15 are located south and west of Jackson on the opposite side of the old

TABLE I

Log of Borings - Marietta Valley
Hole C-1, Site 9, Standpipe Hollow (E. of Jackson)
Elev. 675 (approx.)

Depth	Description
0-3'	Alluvium
3-12'	Very wet yellow silty clay
12-16'	Very wet sandy silt
16-20'	Sandy clay with organic material
20-24.5'	Blue sand with muck
24.5'	Sandstone

TABLE II

Results of Tests on Foundation Soils - Marietta Valley
Hole 13, Site 8 (E. of Jackson)
Elev. 675 (approx.)

Depth	Description	Field MC	LL	PL
0-2'	Br. sandy silt	17.8	27	18
2-7'	Same			
6'	Free water			
7-13'	Gray sand			
13-20'	Wet blue silt	32.9	23	21
20'	Blue silt	23.6	28	19
20'	Limit of drilling			

TABLE III

Log of Boring and Results of Tests
Lower edge of Site 15 - Hammertown Hollow

Hole	Depth	Description	Field MC	LL	PL
Hole B-3 Elev. 644.8	0-4'	Alluvium			
	4-29'	Sandy clay	29.4	30.3	21.9
	29-45'	Gravel			
	45'	Shale			
Hole B-6 Elev. 645.7	0-4'	Silty Alluvium			
	4-16'	Yellow and gray sandy clay		25.1	20.2
	16-32'	Blue sand with "muck"			
	32-42'	Sand and gravel			
	42-45'	Blue shale			
Hole B-16 Elev. 647.2	0-5'	Alluvium	28.0	23.9	18.1
	5-16'	Sand			
	16-21'	Blue silty clay	22.9	31.3	20.4
	21-30'	Same	22.3	31.9	19.8'
	30-35.5'	Fine gravel			
	35.5-38.5'	Sand rock	22.6	31.6	20.1
Hole B-17 Elev. 646.2	0-4'	Alluvium	27.4	32.8	21.0
	4-16'	Sand	-	-	-
	16-30'	Silty clay	21.0	22.4	15.7
	30-43'	Sand & gravel	-	-	-
	43'	Rock			

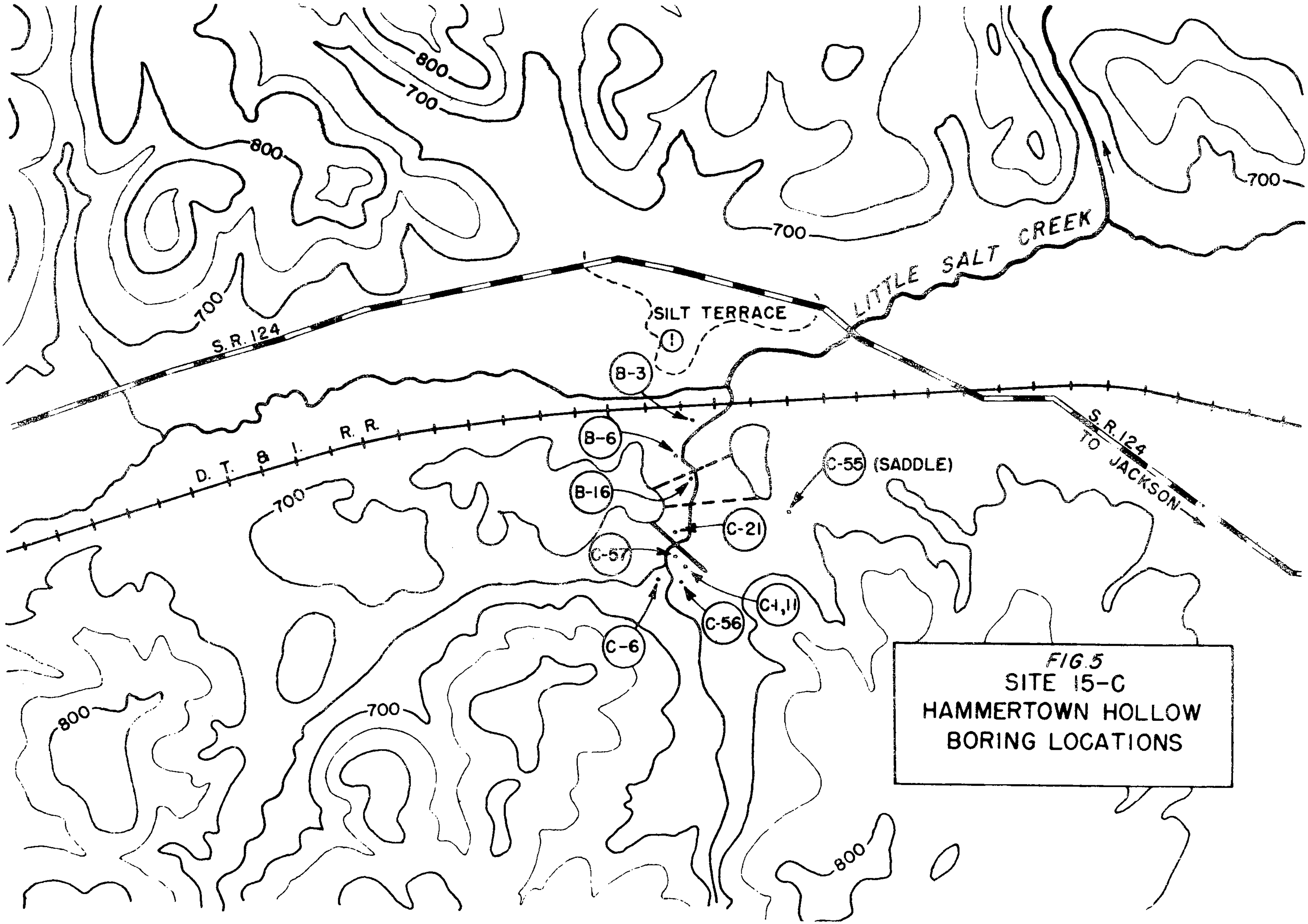
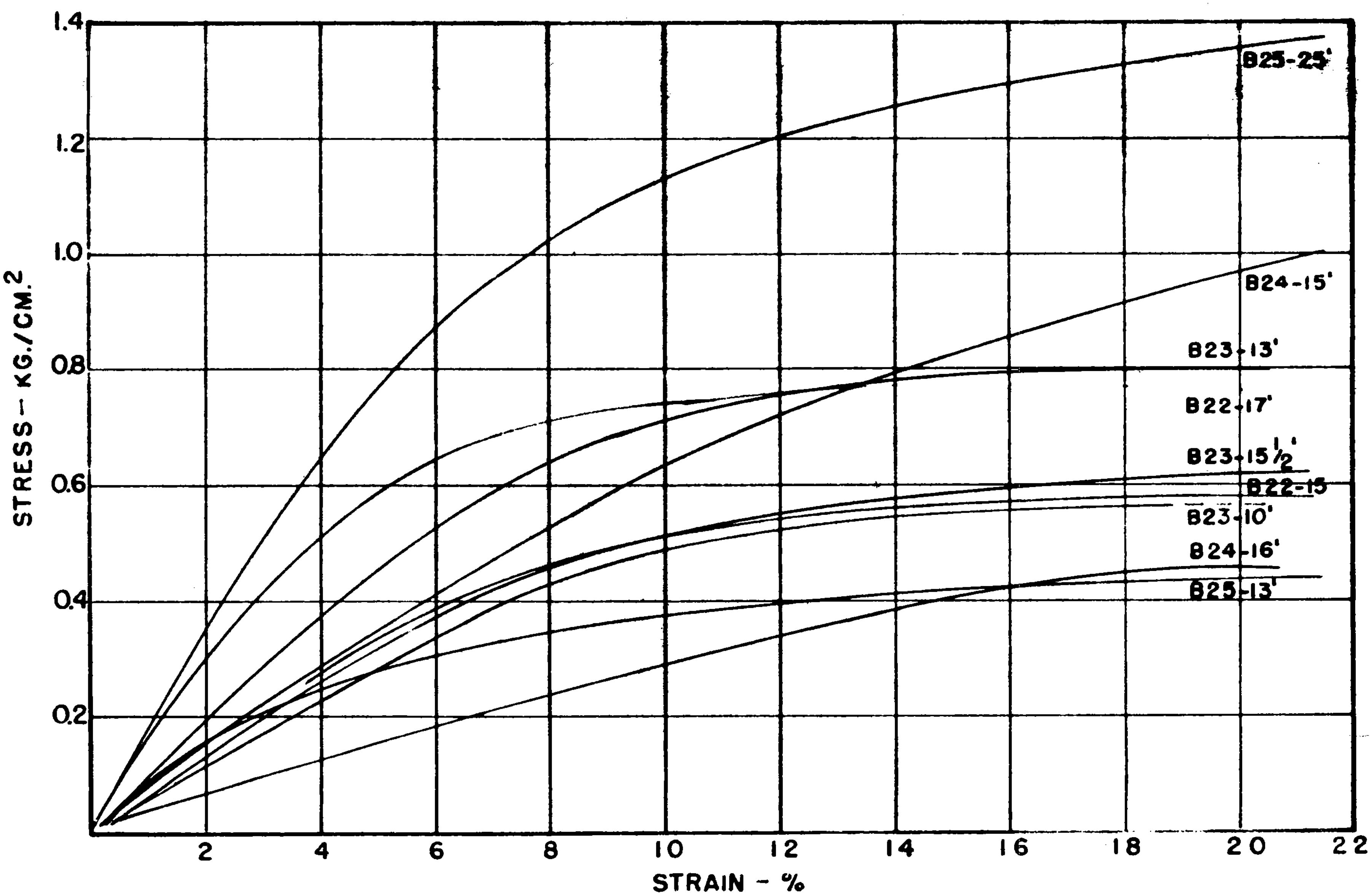


FIG. 5
SITE 15-C
HAMMERTOWN HOLLOW
BORING LOCATIONS



UNCONFINED COMPRESSION TESTS, SITE 15

FIG. 6

valley. The elevations of the present surface of the valley floor for Sites 8 and 9 are somewhat higher than those at No. 15. Inspection of Figure 4 will show that the former areas are at the upper end of the present drainage for the valley, there being a drainage divide just south of Site No. 9.

Some results of tests of samples obtained from one boring near the middle of the alluvial area in Site 8 are shown in Table II, while the log of borings for a similarly located hole at Site 9 is shown in Table I. Attention is directed to the similarity of the alluvial sediments--particularly the organic silt soil which occurs at considerable depth below the surface of the present valley.

The major portion of the engineering data were obtained in the general vicinity of Site 15--known as Hammertown Hollow. Figure 5 shows the location of some of the bore holes. Table III presents the log of borings and some results of tests on samples obtained from some of the drilling performed at the lower end of Hammertown Hollow, i.e., near the mouth of the Valley and adjacent to the pre-glacial Marietta River Valley. Table IV contains similar data from borings made a few hundred feet upstream, while Table V records the foundation information for the upper portion of the area under consideration. Figure 6 consists of results of unconfined compression tests on "drive" samples obtained from Holes B-22, 23, 24, and 25. Table VI contains an over-all summary of much of the pertinent data covering the foundation conditions at Site 15. Figure 7 shows pertinent profile information for the entire Hammertown Hollow area which was investigated. It is pertinent to note that the elevation of rock in the valley floor at Site 15 rises in the upstream direction and that the depth of "blue silty clay" as well as the bottom layer of granular material decreases in extent in the upstream direction.

Some of the physical characteristics of the terrace deposits are reported in Table VIII. The top portion of the table contains soil descriptions and some results of tests for Hole C-55 which is located in the so-called Saddle area just a few hundred feet northeast of the proposed "C" location of the dam. (See Figure 5). The lower portion of Table VIII reports similar data on a Minford-like terrace remnant located just opposite the mouth of Hammertown Hollow in the main portion of the preglacial Marietta River Valley. Figure 8 shows the laboratory compaction data fro the sample of terrace material at a depth of 5.5' to 8' for Hole 1 reported in Table VIII.

One additional table (No. VII) shows the results of tests and log of borings for Site 3A. The dam for this project was constructed during the summer of 1952 and the drill and test data in Table VII were obtained during the preliminary investigations. The holes are located approximately on the present centerline of the dam with the exception of Holes A8 and A9. Attention is directed to the fact that Holes A1, A2 and A7 report hillside data.

DISCUSSION OF RESULTS

The engineering data collected covering the general soil conditions in the vicinity of Jackson, Ohio include information on both the foundation conditions as well as some information on the characteristics of the terrace remnants of the preglacial Marietta River. Since the primary endpoint of the investigation was directed toward locating suitable sites for upland reservoirs, the data collected cover only a few locations, all of which are near the sides of the old valley. It would have been interesting to have had additional data covering the foundation conditions in more central locations of the valley - particularly data on elevation of the now-buried rock floor of the old valley. Also, some additional information on both the foundation as well as terrace soils in more widely-located situations would have been very desirable. Also additional engineering data on both the foundation soils and the Minford silts for locations in the Teays River Valley should prove interesting.

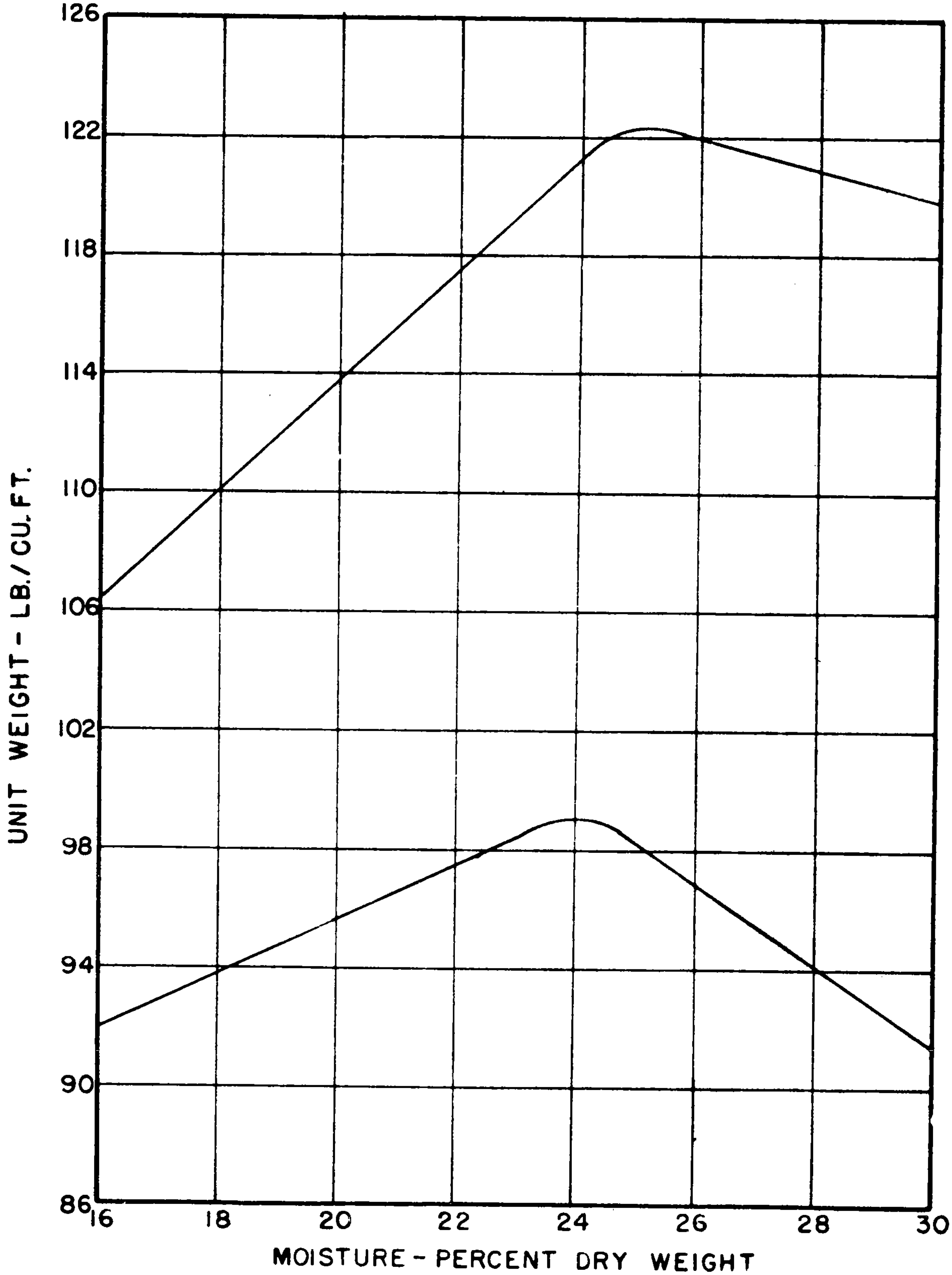
Foundations

Despite the limitations of the explorations, the engineering data

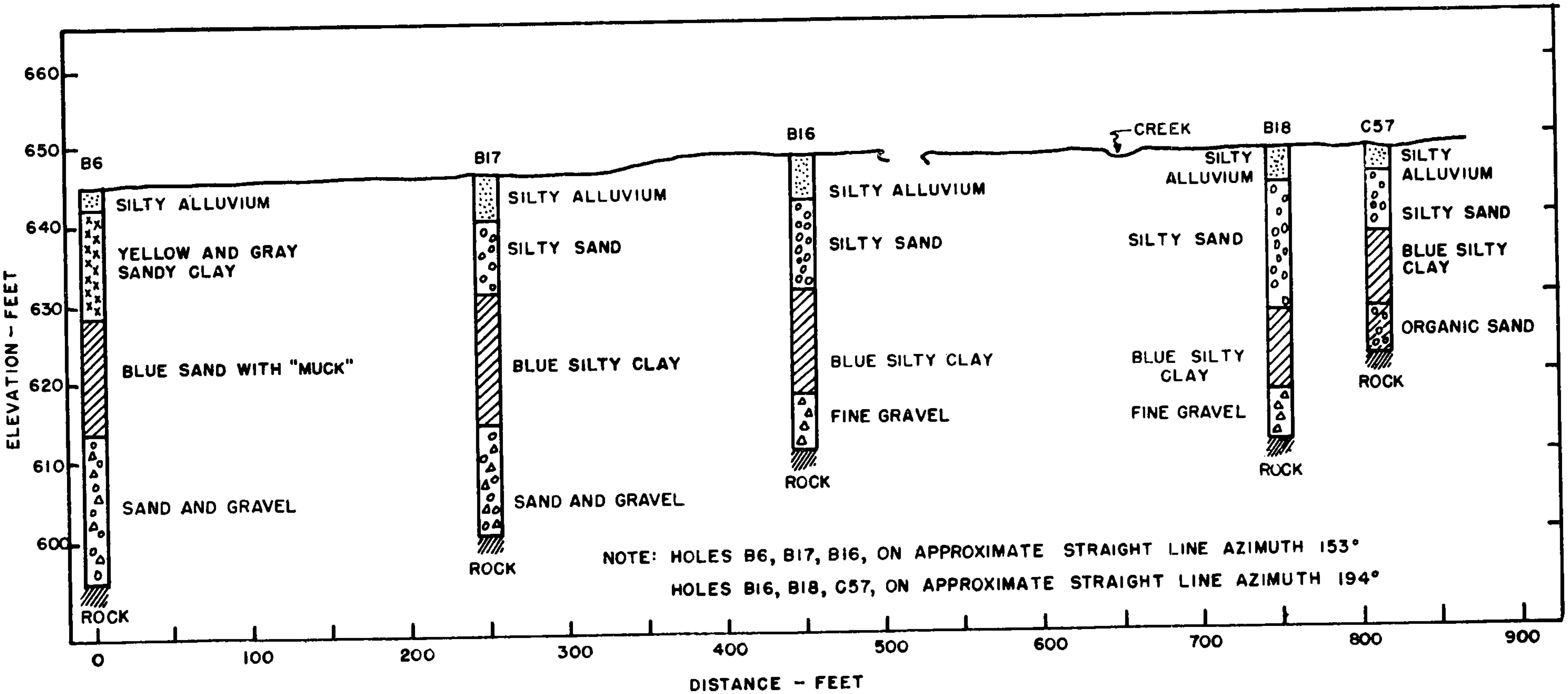
TABLE VI

Summary of Foundation Information
Site 15-C, Hammertown Hollow

Hole No.	Elevation	Sand Strata	Water	Weak Strata	Rock	Remarks
C-1,11	650.4	4-8'	7'	14-16' 19-21'	27'+	
C-2	647.7		9'	6-9'	33'	Drive sample at 14.5' S = .464 T/sq. ft.
C-6	648.1	6-10'	6'	15-20.5'	21'+	
C-7	648.5	4-6'	4-5'	7.5-15'	-	
C-57	647.6	6-11'	9'	20-25'	26'	Organic layer 20-25'
C-56	648.0	7.5-9'	7'	21-27'	-	Organic layer 19-25'
C-12	648.9		12'	12-17'	-	
C-17	646.4	3-6'	6'	6-9'	15'	
C-21	647.2	6-13'	6'		23'	
C-23	646.8		9'	16'	22'	Muck at 16'
C-25	646.7	5-7'	5'	14.5'	40'	
C-30	646.1		8'	15.21'	21'	
B-3	644.9				45'	Gravel 29-45'
B-6	645.7	16-42'			42'	
B-16	647.2	5-16'			35.5'	Gravel 30 to 35.5'
B-17	646.2	4-23'			43'	Gravel 30 to 43'
B-18	647.2	5-36'			36'	
B-19	647.5	5-31'			31'	
B-20	647.0	5-10'		10-24'	24'	Samples 10-16' S Min. = .13 T/sq. ft.
B-21	646.9	13-23'			23'	
B-22	647.3	8-10'	4-5'	10-15'		Sample at 15' S = .295 T/sq. ft.
B-23	647.5		5'			Samples 10-17' S Min. = .31 T/sq. ft. & .19 T/sq. ft.
B-24	647.6	8-10'				Samples 15 & 16' S Min. = .22 T/sq. ft.
B-25	647.0	16'				Sample 7-16' S Min. = .22 T/sq. ft.



PROCTOR COMPACTION CURVES
MINFORD SILT SITE 15
HOLE NO.1 5½'-8'
FIG.8



SOIL PROFILE, SITE 15, JACKSON OHIO

FIG. 7

TABLE VII

Results of Tests on Foundation Samples
Site 3A, J.I.S. Co. Dam Site

	Depth	Description	LL	PL
Hole A-1				
Elev. 162.1	0.3'	Brown silty clay	30	19
	3-4'	White clay	55	23
	4-9'	Gray silty clay	44	21
	9-14'	Gray clay	37	19
Hole A-2				
Elev. 144.9	0-5'	Tan silt	29	17
	5-11'	Clay	55	30
	11'	Shale		
Hole A-3				
Elev. 109.2	0-4'	Silt	23	17
	4-5'	Sandy silt	27	17
	5-11'	Sand and gravel		
Hole A-4 Center Line Dam Middle of Alluvium				
Elev. 106.4	0-5'	Alluvium	29	18
	5-19'	Silty sand		
	20'	Sandstone		
Hole A-5				
Elev. 104.9	0-7'	Sandy sand		
	7-10'	Fine sand		
	10-13'	Coarse sand		
Hole A-6				
Elev. 107.1	0.7'	Sandy silt	23	17
	7-10'	Sandy silty clay	28	17
	10-12'	Sand and gravel		
Hole A-7				
Elev. Hillside	0-4'	Alluvium		
	4-9'	Silty clay	37	19
	9-11'	Clay	42	24
	11-14'	Sand		
	14-17'	Silty clay (shale)?		
	17'	Rock		
Hole A-8 Upstream in center of Alluvium				
Elev. 106.3	0-4'	Alluvium	34	22
	4-22'	Fine sand		
	22'	Limit of drilling		
Hole A-9 Downstream in Center of Alluvium				
Elev. 107.8	0-4'	Alluvium		
	4-15'	Silty sand		
	15'-20'	No log		
	20-40'	SS & Shale		

TABLE VIII

Results of Tests on Terrace Soils
Site 15
Hole C-55 Saddle Location - Hammertown Hollow - Elevation 676.4

Depth	Description	LL	PL
0-1'	Top soil		
1-10'	Br. silty clay and sand	27.8	17.8
10-11'	Same	24.6	14.7
13'	Free water		
11-16'	Wet sand - some organic material		
16-18'	White silty clay	32.2	17.7
18-20'	Silt		
20'	Limit of Drilling		
51'	Predicted depth to rock by earth resistivity method-Power drill did not reach rock at 42'		

Hole 1* Preglacial Marietta River Valley opposite Hammertown Hollow El. 680.0

Depth	Description	Field MC.	LL	PL
0-2.5'	Tan silt clay	10.0	30	20
2.5-5.5'	Tan clay	12.0	52	24
5.5-8'	Mottled gray clay	14.5	52	27
8-12'	Clay with black partings	25.0	47	26
12-15'	Gray clay	24.0	54	24
15'	Free water			
15-18'	Tan sandy silt			
18-20'	Gray clay			

Hole 2**		Field MC.	LL	PL
	Terrace edge Gray clay	-	47	24
	Rd. Cut Salt Creek Valley		68	30

* In old terrace position along valley wall opposite the mouth of Hammertown Hollow.

** Sample taken on edge of terrace near Hole No. 1

collected define some interesting trends. The drill records of holes made in the alluvium in locations 8, 9, and 15 all show a soil profile of (a) surface soils representing recent alluvium and consisting of sands, sandy silts, and silty clays, (b) a buried layer of very sensitive, wet silts and silty clays containing some organic materials with thicknesses of as much as 16' and (c) an underlying layer of variable thickness of sands and gravels, which in turn rest on bedrock.

The sensitive silts have very low unit weights with correspondingly low strengths and unusual remolding characteristics. They are blue-gray in color and change to light tan color after drying either in the air or by oven drying. These materials are not always "organic" in the normal sense of the word in that they are not compressible; however, they are very sensitive and are subject to loss in strength after remolding. In some cases this material was erroneously identified as "muck" in the field but decayed vegetation was rarely noted in the samples.

The strength data obtained on the buried strata of sensitive silts and silty clays indicate the existence of an unusual material. Field moisture contents are invariably near to and sometimes above the values for liquid limit thus indicating an extremely weak material. It is probable, however, that the unconfined compression test values reported in Tables IV and VI and shown graphically, in part, in Figure 7 are conservative, i.e., the actual field strengths are greater than those indicated. This surmise is based primarily on the fact that it was necessary to obtain samples by the "drive" method for the determination of the unconfined, compressive-strength values which no doubt resulted in considerable disturbance of the samples. It will be noted in Figure 7 that shear failures in the normal sense, were never encountered, but that the samples were soft and showed appreciable increases in strength up to as high as 20 percent strain.

The present drainage in the Marietta valley divides at a point some three miles southeast of Jackson thus resulting in a minimum water shed to supply water to the underlying sands and gravels in this vicinity. It would be possible for water to exist in this strata under some head a few miles distant from Jackson as the overlying impervious layer would tend to confine water in underlying pervious strata. Additional data on both the elevation of the top of bedrock in the valley floor as well as the characteristics of the buried soils are needed to establish firmly the ground-water possibilities in other sections of the valley.

"Silt" Terraces

On the basis of the samples obtained from the terrace remnants, both above and below Jackson, the "silts" have very definite clay-like properties. An inspection of the second portion of Table VIII will show that, at least the material for 2.5' to 15' is very plastic with liquid limit values of about 50 percent and plastic limits near 25 percent. The laboratory compaction curves made on a sample from this same strata (Figure 8) also indicates a material with clay-like characteristics with a maximum dry weight of about 99 pounds per cubic foot and an optimum moisture content of 24.0 percent. The character of the deeper strata in these terraces is not known although a three-foot layer of non-plastic silt does occur as shown in the profile in the second portion of Table VIII. A layer of similar material two feet in depth also occurs at about the same elevation in the so-called Saddle location.

The contrast in the characteristics of the material from the Saddle as compared with the material in the major terraces is striking - with the Saddle area containing materials of sandy and silty characteristics while for the most part the soils from the terrace are quite plastic. The slopes on both sides of the saddle are long and they occur in low gradients. The need for additional geological investigations to determine the origin of the materials in the Saddle locations is definitely indicated. The location of at least one additional "saddle" is known thus indicating that this phenomena may be of extreme importance. These filled-in materials blend in well with the natural setting and are therefore not easily distinguished from other landforms. Their presence is extremely important to engineering - not only for water-supply reservoirs but also in connection with the location of highway and railroad transportation systems.

ENGINEERING AND GEOLOGIC SIGNIFICANCE

The results of this study stand as mute evidence that engineering programs planned for any portion of the Teays River Valley or its tributaries should be approached with extreme caution and planned with great care. Much information is available on the old Teays drainage basin and should be consulted at the out set of an investigation. However, nothing can compare with or replace the first hand information procured by an intelligently planned exploration program based on existing information.

The consistency of the profile, as indicated by drill and laboratory test data, of the valley-fill sediments in the Marietta River Valley, confirms in part the available geologic information. The sands and gravels occurring as the basal strata on the valley floor immediately above the bedrock were deposited by the river at a time when the velocity of flow was moderately high. The overlying organic sands and silts probably represent the period in the life of the river when the water had been diverted, the stream bed was stagnated, and the river bottom became swampy. The present surface mantle of sands and silts probably represents deposits which have accumulated during and after the glacial period.

The presence of sands and gravels sandwiched as they are between an underlying rock strata and an overlying semi-impervious silty layer constitutes potential sources of water. However, considerable data are needed with respect to the elevation of the underlying rock floor as well as additional information on the depth and lateral extent of the water-bearing materials.

The presence of the buried layer of very wet organic silts and sands constitutes a real hazard with respect to the location of any type of structure requiring firm foundation support. These soils are extremely sensitive and high embankments for dams, highways, and railroads cannot be supported without extra design precautions. Heavy industrial buildings will probably require piling to rock.

Additional information is needed with respect to the engineering characteristics of the materials occurring in the major terrace remnants of the old valley. Existing data indicate an extremely plastic material which is not particularly well-suited for use in high embankments. Academic interest indicates the need for some additional detailed testing of the terrace materials in both the Marietta and Teays Valleys to determine whether or not important differences exit between the two deposits.

More geologic information is needed badly with respect to the origin and the potential number of so-called saddle deposits. These saddles are important in the location of dams, since the sand strata constitute potential seepage areas. Also, the elevation of the tops of the saddles is such that they represent potentially undesirable spillway situations. The existence of these fill-in areas also could have a bearing on the location of certain highways and railroads.

In any type of construction where water-tightness is necessary, the water-bearing strata must be intercepted by use of cut-off trenches.

The work done at Jackson, Ohio indicates that the elevation of the rock floor rises in the bottoms of the gullies and small streams in the upstream direction of the tributaries, thus pinching out the sands and gravels and diminishing the thickness or even pinching out completely the layer of organic sands and silts.

Geologically, the engineering data obtained from this investigation provides another link in the overwhelming chain of evidence to support the concepts of preglacial drainage systems in southeastern Ohio. Investigations such as this

TABLE IV

Log of Borings and Results of Tests Made on Foundation Soils
Site 15, Area C - Hammertown Hollow

	Depth	Description	Field MC	LL	PL	Dry Density (#/cu.ft.)	Unconfined Compression (Kg/sq.cm)
Hole No. B-20							
Elev. 647.0	0-5'	Alluvium					
	5-10'	Sand					
	10-24'	Wet organic sand					
	24'	Sandstone					
Hole No. B-21							
Elev. 647.9	0-5'	Alluvium					
	5-13'	Yellow clay					
	13-23'	Blue clay and sand					
	24'	Sandstone					
Hole No. B-22							
Elev. 647.3	0-5'	Alluvium (silt)					
	4-5'	Water					
	5-8.0'	Stiff sandy clay					
	8-10.0'	Fine sand					
	10' -	Blue silt clay					
	at 15'		22.6	31	20	102.0	0.59
Hole No. B-23							
Elev. 647.5	0-5'	Alluvium (silt)					
	5-9'	Sandy clay					
	5'	Water					
	9-10'	Sandy silt clay					
	10' -	Blue clay (stiff)					
	at 13'		31.6			89.5	0.80
	at 15½'		30.1	45	23	90.6	0.62
	at 17'		27.6			80.0	0.77
Hole No. B-24							
Elev. 647.6	0-8'	Alluvium (silt)					
	8-10'	Fine sand					
	10' -	Blue clay					
	at 15'		24.3			100.0	0.95
	at 16'		27.1	33	19	98.0	0.45
Hole No. B-25							
Elev. 647.0	0-2'	Alluvium (silt)					
	2-7'	Sandy clay					
	7-16'	Blue clay					
	at 7.5'	Blue clay--stiff	24.6			102.0	1.35
	at 10'	Blue clay--soft	32.5	49	23	85.0	0.56
	at 13'	Blue clay--soft	35.4			84.0	0.45
	16'	Fine sand					

TABLE V

Results of Tests Made on Foundation Soils
Dam Site (15-c) - Hammertown Hollow

Hole C-56
Elev. 648.0

Depth	Description	Field MC	LL	PL
0-2'	Alluvium			
2-7'	Free water			
7-7.5'	Br. Silty clay	23.6	24.6	17.9
7.5-9'	Gray sand and clay			
9-10'	Bl. silty clay	30.1	34.8	22.1
10-13'	Same	32.4	31.5	22.4
13-19'	Same	17.1		
19-21'	Organic silty clay	36.7	40.2	22.8
21-22'	Br. firm Peat	82.8		
22-25'	Organic silt clay	40.8	40.9	26.6
25-27'	Silt with sand	43.9	29.6	21.6
27'	Limit of drilling			

Hole C-57
Elev. 647.6

Depth	Description	Field MC	LL	PL
0-2'	Alluvium	-	-	-
2-6'	Br. silt	-		
9'	Free water	-		
6-11'	Sandy clay	-		
11-13'	Bl. silty clay	26.7	34.5	22.7
13-20'	Same	34.1		
20-23'	Organic silty clay	30.4	31.6	20.9
23-25'	Organic silty & sand	31.3	33.8	21.9
25-25.5'	Same	28.1		
25.5-26'	Organic sand	-		
26'	SS Pebbles			

Hole Cl, 11
Elev. 650.4

Depth	Description	Field MC	LL	PL
0-4'	Alluvium			
7'	Free water			
4-8'	Sand - silt			
8-10'	Blue silty clay	23.5	25.5	18.4
10-11'	Same	23.6	28.5	18.8
11-12'	Same	21.8	23.4	16.1
12-14'	Same	21.8	21.7	15.0
14'-16'	Tough blue silty clay	17.2	23.2	15.1
16-18'	Same	18.7		
18-18.5'	Blue sand	-		
18.5-21.0'	Soft blue silty clay	27.8	28.5	17.8
21-27'	Same			
27-33'	Soft rock or gravel			
33'	Sandstone			

invariably lead to discoveries hitherto unsuspected which in turn must be further explored. This confronts the geologists with new problems and questions which must eventually be answered. In the last analysis mutual benefits result and the ends of both are achieved.

SUMMARY OF RESULTS AND CONCLUSIONS

The following conclusions are based on information developed during the investigation of the foundation conditions in the vicinity of Jackson, Ohio:

1. The general profile of the filled-in area of the Marietta River Valley—at least adjacent to the valley wall-consists of (a) about 15 feet of recent alluvium ranging in texture from sands to silty clays,(b) from 10 to 20 feet of wet, sensitive blue silty clay, (c) from 0 to 15 feet of sand and gravel, and (d) a rock floor consisting of shales and sandstones.

2. The existence of these strata confirms the geologic findings to the effect that the deposition of granular materials under conditions of relatively high river velocities, the development of the wet organic silts and sands during the drying-up period when the river bottom probably was in a swampy condition, and finally the glacial and postglacialdepositions of sands and silts, as alluvial deposits.

3. The organic silts are extremely weak and are not capable of carrying heavy loads without the use of unusual design features —high embankments for railroads, highways and dams will require very flat slopes. For heavy building foundations the use of piles to bedrock is indicated while in the construction of reservoirs, cutoff walls are necessary to minimize the seepage thru the underlying basic granular strata.

4. The character of the terrace deposits range from sands and silts through plastic clays. On the basis of the observations made at Jackson, it is indicated that the terrace remnants in the major valley are plastic silty clays, while other terrace remnants somewhat removed from the major valley may be sandy

or even silty in character. No conclusions were reached with respect to the deposition of the materials in the so-called saddle locations between upland areas and rock islands—although the deposition of such sediments could have occurred following a local change in the direction of flow of water.

5. On the basis of the data obtained at this site it is emphasized that to adequately evaluate the structural properties of the foundation conditions in these valleys, an understanding of the geological processes by which the soils were deposited is essential.

6. The need for additional geologic and engineering information, is emphasized. Elevation of the rock floor and the source of the materials appearing in the "saddle" and terraces of the Marietta valley are examples of some of the needed information which has been brought to light as a result of this investigation.

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THE LANDSLIDE PROBLEM

In The Construction and Maintenance of Highways in West Virginia

By

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A landslide is a fairly rapid movement of a superficial part of the earth's crust, caused by yielding to gravity. The mass in motion ordinarily moves diagonally downward, always to a lower position than that which it had formerly occupied.

In nature, a landslide will occur whenever a mass of rock or soil lacks adequate support. The material that moves may be either competent or incompetent, but the presence of a bed or thicker mass of incompetent material is almost always a factor of the occurrence of any serious slide. Landslides may take place both above and below sea level - In fact, submarine slides are frequent, and often very large. As our present interest is in the relation of landslides to highways, we shall confine our attention to those occuring on land.

The lack of support that is a necessary factor of all landslides may result form "natural" processes or from human activity, as is much more frequently the case when highways are damaged by slides. Man may very easily cause a landslide either by removing or by piling up material, and most of the slides with which we shall be concerned have been so caused. Because this is the case, as much or even more attention should be given to their possible avoidance as to their correction.

Various classifications of terrestrial landslides are possible. Perhaps the most natural is made on the base of the kind of material set in motion. This gives at once these two main causes:

I. Bedrock landslides

II. Superficial landslides

The names are self-explanatory. The former class includes most of the larger landslides. On the same base, it may be subdivided into:

1. Landslides consisting entirely of incompetent material and:

2. Landslides consisting partly of competent material that had rested upon the incompetent material, the failure of which has usually been the immediate cause of the slide.

Superficial landslides also consist of two groups, depending upon the material involved:

A. Soil slides

B. Fill slides

Classification of landslides on the base of immediate causes also gives us two main classes:

a. Caused by natural processes

b. Caused by the activity of man.

Many of the landslides with which we have to do here belong to the second of these classes. Various subdivisions may be made of this class. Perhaps as convenient a scheme as any divides it thus:

Landslides caused by:

Excavation

Filling

Overloading

Removal of forest cover

Alternation of the water level.

These groups will now be discussed individually, in so far as they affect highway construction and maintenance, and appropriate remedial or preventive measures will be suggested.

Landslides Caused by Natural Processes

Of the many landslides produced entirely by natural processes, only those immediately caused by erosion do much damage to highways in this State. The nature of our terrain is such that considerable stretches of our highways must closely adjoin streams, and, in many cases, have to rest upon alluvium. As is well known, any stream that flows through a valley just wide enough to contain narrow strips of alluvium besides the stream bed itself, constantly tends to change its course, cutting away the alluvium first on one side, then on the other. It may therefore remove the alluvium along a highway, then undermine the road itself, causing part or all of the road bed to slide down into the stream bed. Though less frequently a stream may also sufficiently erode incompetent bedrock below a road to cause a landslide.

Highway damage caused by the erosion of alluvium is very frequent in West Virginia, and effective remedial measures are often hard to find, since any strong stream usually carries away quite promptly any fill material dumped in beside a threatened highway. The only good preventive measure is simply not to locate the highway on alluvium likely to be eroded within the probable life of the road. Should this not be possible the construction of a wall resistant to abrasive erosion may be used.

An example of a slip of this type is located on State Route 25, $\frac{1}{2}$ mile east of Dunbar, in Kanawha County. Here the Kanawha River is eroding the alluvium upon which the road rests, and refilling does not seem to have been very successful as a remedy.

Small but dangerous slips have developed on State Route 7 between Willeyville and New Martinsville in Wetzel County. Here Fishing Creek is actively eroding soft red shales upon which the road rests, and refilling does not seem to have been very successful as a remedy.

Landslide Caused by Excavation

Any excavation necessarily removes some of the lateral support of the material on one or both sides, and may provoke either a bedrock or a superficial slide. While superficial slides so caused are not usually very important, the bedrock slides may be serious, especially in terrain such as we have in this State. Throughout the western part of West Virginia, the surface rocks are nearly horizontal shales and sandstones of Pennsylvanian and Dunkard age. Many of the shales are soft red claystones, essentially weak rocks in which landslides easily form. On hillsides the zones of exposure of those rocks are usually represented by gentle slopes, which alternate with steeper zones formed by the sandstones. There is little slipping as long as the terrain is undisturbed, for the sandstones reenforce and support the shales. But a road grade up a hillside necessarily cuts diagonally across the shale zones and leaves masses of shale just above the cut without lateral support. As soon as these become saturated with water- often during the first winter after the cut has been made - they may begin to slump down into the road grade. The cut has also made the shale beneath it more accessible to water than it had previously been, and when it becomes thoroughly saturated, it too may start to slide, carrying the road bed with it.

A very troublesome slip of this kind occurred on Richwood Avenue in the City of Morgantown, where the street cuts through the Clarksburg red shale. After it had caused trouble for several years, this slip was finally controlled by means of the construction of a retaining wall based upon underlying more competent rocks. Large and small slips of this type are very common on country roads, in the western part of the State, where such expensive, though effective, control measures as those used in Morgantown are usually regarded as impractical. Instead, the road is usually filled across the slip, or relocated deeper in the hillside.

Where a road must be constructed through a region of steeply dipping beds, such as most of the eastern part of West Virginia, the shale zones may be very troublesome, or may cause no difficulty whatever. Much depends upon the attitude of the bedding relative to the relief. Motion in shales develops most easily along bedding surfaces, and in an area of steep dips, a slide so started may carry along large masses of overlying competent rocks. Steep dips usually result from deformation of the rock body, and the stresses that produced this have also often developed a system of conspicuous joints in the rocks affected. Such joints are also surfaces of weakness, along which slipping may take place quite as easily as on bedding planes.

If the road cut is made through beds that dip into the slope, there is much less danger of serious trouble with slides. Even here however, yielding on joint surfaces may result in considerable slipping. More frequently, blocks of the more competent beds exposed on the face of the cut fall intermittently into the road as they are undermined through disintegration of less competent interbeds. Alternate freezing and thawing greatly accelerates this process under the climatic conditions we have in West Virginia.

The slope of a talus fan nearly always coincides with the angle of repose of the material that forms the talus. If a road cut is made through such a fan, talus material from above nearly always slides into the road. The amount depends upon the depth of the cut and its position on the fan. As a rule, a slip of this kind is handled simply by removing the material as fast as it reaches the road. Such slips are quite common on West Virginia highways. An example is to be found on U.S. Route 52, just west of Clear Fork, McDowell County.

If part of a talus fan is removed from below a road that crosses it, the road bed itself is quite likely to start sliding. This is a more serious situation than the preceding, and permanent correction may require relocation of the road. The slide at Cheylan Bridge on U.S. Route 60, in Kanawha County, is an example.

Landslides Caused by Fills

Through typical West Virginia terrain, much of any highway grade necessarily consists of an alternation of cuts and fills. Usually it is not difficult to construct a stable fill, but unless this is done during construction, a slide very difficult to control may develop later, and we have many examples of such failure. It is to be assumed of course that the fill is so made that it will not deform under its own weight, combined with a normal traffic load. Even when this condition is met, the fill may move as a whole unless it has adequate support. This depends largely upon the attitude of the basal surface of the fill and upon the character of the material just below this. If the basal surface is practically horizontal, there will be no tendency for the fill to move as a whole upon it. In the case of an inclined basal surface, there is always a component of the weight of the fill that tends to push the whole mass down the slope. As the accompanying diagrams show, this component increases rapidly with the steepness of the basal surface when this slopes at an angle of 30° or more.

A deep fill of any considerable length should never be made upon the natural surface of the ground, though a comparatively short deep fill, such as must be used in crossing a narrow canyon-like valley, may be made with hardly any preliminary work on the ground surface. Such a fill is supported largely by the steep surface at its ends, and is unlikely to give any trouble other than that caused by compaction of the fill material itself. A long fill however receives practically no support from the surfaces at its ends, and, unless it is properly "anchored" to the original ground surface below, is very likely to slide down even a very gentle slope. The usual control measures - retaining walls or piles - are invariably more expensive than proper preventive work at the time of construction, moreover are often ineffective.

An expensive example of a slip of this type (and as yet uncorrected) is to be found on State Route 73, about six miles south of Morgantown, Monongalia County. Here the highway upon the fill had to be abandoned and replaced by a detour around the rather shallow valley crossed by the fill. A slip of the same kind has taken place in Wood County, on State Route 2, a mile N. of Parkersburg.

Slides Caused By Overburdening

The Pennsylvania stratigraphic column of West Virginia includes fire-clays and red shales that have practically no strength when dry and even less when saturated with water. Any heavy fill made directly upon the outcrop of such a bed is fairly sure to subside as the soft clay is squeezed from beneath it, and will probably also move laterally. Even where there is no fill, any road bed built across such shale or fireclay will be insecure, and probably will subside more or less under heavy traffic. Every highway leading out of Morgantown has at one time or another failed when it crosses either the Pittsburgh or Clarksburg red shales. Minor damage so produced is very common on our highways, and no doubt will increase as the traffic burden becomes greater. At the time of construction of most of our roads, neither the highway engineers nor anyone else could have been expected to forsee the enormous increase that had taken place in the weight of the trucks that use our roads. Of late years, many slips have developed on hillsides, where highways necessarily cross the outcrops of incompetent beds. In many cases, it is felt that such slips have been caused, directly or indirectly, by increased heavy traffic. One can hardly doubt this if he takes the trouble to stand by the roadside at such a locality and notice the road bed yield, perhaps several inches, under the weight of the heavier trucks that pass. Slides on U.S. Route 250 in Marshall County, between Littleton and Moundsville, have almost certainly been caused by heavy traffic. The only remedial measure taken to date has been to restore the grade by filling and resurfacing.

So far as existing highways are concerned, little can be done to prevent damage such as that just described. In new construction, special care should be taken to secure a good support forthe road bed where it must cross the outcrop of an incompetent bed. So far as possible, such zones should, of course, be avoided.

Landslides Caused by Removal of Forest Cover

Forests will grow and maintain a continuous soil cover on very steep slopes

- to about 45° under such climatic conditions as we have here. When the forest is removed from a steep hillside, there is usually no immediate damage. Within two or three years, however, the mattress of interwoven roots that held up the soil decays and the whole mass, down to bedrock, slides away. A covering of sod will not save a steep slope. Grass roots are short and weak, hence to not enter the crevices of the bedrocks, and anchor the soil as do tree roots. Fortunately, the forest cover is not seldom removed from large areas of steep slopes in this state.

Road Failure Or Damage Due To Differential Compaction

Many of West Virginia's highways are located on stream terraces or on tallus slopes paralleling our major streams. These terraces or talus slopes of unconsolidated materials are superimposed upon an irregular and uneven topography, sloping toward the stream. The earth mass is subject to a fluctuating water level with alternate wetting and drying, and with the removal by sapping of the fine clay and silt particles together with particle rearrangement, hence decreasing the volume. This not only results in differential compaction and settling but thrusts downslope on the suballuvium topography result in an uneven rolling or "wash-board effect" roadbed.

This condition is almost ever present along our highways paralleling our rivers and is perhaps the most difficult problem we have to contend with. To date there is no satisfactory solution to this problem. Measures presently used are refilling the sunken areas.

Examples of this type are found throughout State Route 2, paralleling the Ohio River. Similarly along the Kanawha and Monongahela Rivers.

Remedial Measures

Procedures adopted for control of the slides hitherto mentioned have already been discussed. Many other methods have been employed, more or less successfully, in efforts to control landslides, or to minimize the damages to highways that they have caused. The first procedure that should be undertaken is to determine the geological conditions that are responsible. Not always, but generally, this requires the aid of an experienced geologist, and one who is familiar with the geology of the area. Each slide should be examined individually, and an effort made to select the procedure most likely to succeed in controlling it, provided that this is not too expensive to be justified by the prospective value of the highway. It is clear that a very expensive piece of repair work may be fully justified on a main highway that carries a heavy traffic load - An example of this is the new road through the Benwood "Narrows", just south of Wheeling - though such expenditure on a minor highway would be indefensible.

The usual measures adopted in dealing with landslides that damage highways may be classified under three headings:

Removal of slip material

Stabilization of slip

Relocation of road, avoiding slip.

The first mentioned of these is of necessity the first measure taken when a road is blocked by a slide coming in from above. Unless the slide is large, it is usually all that is required, and even in the case of a large slide, it is often better to remove all the material likely to block the road rather than to make an attempt - perhaps unsuccessful - to stabilize the part of the slip that can be left above the road.

If a slip involves the roadbed itself, some method of stabilization will likely be required, unless it is found more practical to relocate the road so as to avoid the slide. However, a slide may soon come to rest spontaneously, and in such a position that there seems to be little danger of further movement. In this case, especially if the roadbed has undergone only minor subsidence, the most practical method of repair may consist simply in refilling to the road grade and resurfacing. If the slide moves again, this will have to be repeated, but even so, it may be much cheaper than any measures that might succeed in stabilizing the slide at an earlier date.

The fact that so many different methods of stabilizing landslides have been tried is good evidence that it is difficult to succeed in this task. The first measure usually thought of consists in driving rows of piles deep into the subsoil below the foot of the moving mass. This may be successful, but if the slip is large it will probably fail. Unless the piles are close together, the slip material will simply flow through between them. Otherwise, the pressure of the slip will either break off the piles or push them over and carry them along with the slide. In many parts of West Virginia, one does not have to travel far to see, far below a road, the remains of rows of piles that had been driven quite close to the road in an unsuccessful attempt to stop a slip. The use, instead of piles, of lengths of pipe, usually reclaimed from wells or pipe-lines, placed in holes drilled for them, then filled with concrete, is more likely to succeed. Here too however, the slip material may flow through between the pipes unless they are placed very close together, or the spaces between them bridged in some way.

The construction of a retaining wall is an expensive but <u>usually</u> successful method of stabilizing a landslide. False economy should be avoided, in building such a wall, for if it fails under the pressure of the slide, it will represent nothing more than total waste of time and money. To guarantee success, the wall must rest upon a bed of competent rock, and must be thick encugh to withstand the direct thrust of the slide above it. A wall that is too thin will invariably be toppled over by degrees, as it yields to the slide.

Attempts at draining the slide itself are often unsuccessful. The slipping material often consists largely of clay, from which water will not adequately drain. Good drainage channels made just above and around a slide are more likely to be useful, as they help prevent the entry of more water into the moving material thus tending to stabilize the slide.

If a slide is moving down into a narrow valley, it may be easy to check it by means of filling a section of the valley to above the foot of the mass in motion. A somewhat similar procedure is sometimes successful when a fill has started to move. In this case, the foot of the fill is held in place by means of a long gently sloping secondary fill made against it.

Preventive Measures

Various precautions intended to diminish the probability of landslides along highways have already been mentioned. In many cases it is not easy to have these taken, since they usually increase the first cost of the road. The guiding principal throughout is simply to avoid, so far as is practicable, terrain that is likely to slip. Of course if a zone of outcrop of a thick incompetent red shale lies across the route of a highway, it must be crossed. However, the road-bed should rest upon such a shale for the shortest possible distance, and especial care should be given to the road foundation here, in order to lessen the possibility of traffic starting a slide.

Highways in valleys should not be constructed upon talus or alluvium likely to be undermined by streams in the near future. In passing through a rugged area of steeply dipping rocks, location of a highway on slopes likely to develop bedding surface slips should be avoided, as should deep cuts through brecciated zones.

Fills, large or small, should under no circumstances be made upon the natural surface of the soil, nor upon a sloping surface of any kind. Care should be taken to secure adequate drainage through and under the foot of any large fill. The fill itself should be made of proper material, and sufficiently wide at its base, to insure against its spreading under its own weight or under traffic. A fill is almost always made of material taken from cuts nearby, and the lithology of our upper Paleozoic section is such that, in the western part of West Virginia, this material will nearly everywhere be suitable for filling.

After a road has been constructed, a few minor precautions in its care and upkeep may help to prevent slips. Especially through shale zones, drainage should always be kept open along and across the road. The drainage of fills should be watched carefully, especially when the fill is not very old.

In conclusion, I would very strongly recommend the utilization of the specialties of the trained geologist with that of the highway engineer combined in dealing with all problems of landslides and earth subsidence.

LANDSLIDES AFFECTING WEST VIRGINIA ROADS

By

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Webster defines a landslide as "The slipping of a mass of earth or rock on a mountain slope or behind a sea cliff." Such movement is caused by the weight of material itself, any super-imposed load, the weight of contained moisture and reduction in stability caused by the softening effect of excess moisture.

The size of a landslide may vary from a minor fall in a cut slope to a large mass many feet in thickness and covering a large area. Movement may be very rapid, as in avalanches, or may be slow, as in creeps, where movement is only perceptible over considerable periods of time.

TYPES OF LANDSLIDES

Numerous authors, among whom are Dr. George E. Ladd, Dr. Chalres Terzaghi, and Dr. C. F. Stewart Sharpe, have classified landslides into a variety of types. I understand the Landslide Committee of the Highway Research Board has recently reached agreement on a classification to cover all types common in the United States. Dr. Sharpe's classification is the most complete of any with which I am familiar. All of these classifications are the result of a great deal of study and are valuable to the engineer in the study of the cause and treatment of individual movements.

The following simple classification is sufficient for this discussion:

1. Bedding Plane Slides: Slides of this type occur in folded or inclined structure and are most common where the strata is not of great thickness and where more or less jointing is present. Movement is caused by removal of support, either by excavation in the construction of the roadway or by erosion.

2. Slides of Unconsolidated Material. This type of landslide occurs in soil formed by decomposition of the local rock, transported material, talus, or

combinations of two or more classes of these materials. The material is more or less porous and readily absorbs water, either from the surface or from seepage planes in the underlying strata. Movement may occur on the rock which underlies the material involved, or impervious soil which furnishes a plane for movement, or on a shear plane within the soil. The causes of this type of slide are:

(a) Removal of support by undermining the toe either by excavation or erosion.

(b) Superimposed fills which load soil beyond stability.

(c) Raising of the moisture content either by increasing the supply of water or by blocking its outlet. Under certain conditions capillary water may saturate a soil mass enough to cause instability.

(d) A combination of (a), (b) and (c).

3. Settlement of Fills. Movement of this type is caused by:

(a) Compaction of fill material by natural processes, after placing.

(b) Loading of the foundation on which the fill is placed beyond its supporting capacity.

(c) Construction of slopes steeper than permitted by shear characteristics of the material involved.

The settlements of artificial fills are not usually true landslides; however, such movement frequently has many of the characteristics of true slides. The foundation material, if overloaded, may squeeze out in sufficient volume to produce movement having slide characteristics. Slope readjustment may also have landslide characteristics.

PREVENTIVE MEASURES AND CURES

Each landslide is an individual problem to which few generalities can be applied. The following methods of approach have been successfully used in numerous cases here in West Virginia:

Bedding Plane Slides. The control or cure of bedding plane slides, once started, is difficult. In most cases the only remedy is removal of all the material involved in the movement. If the volume of material is too great to be moved economically, relocation of the roadway is the only alternative. Retaining walls or other barriers are of little practical or economic value in the control of extensive slides of this type and small ones can be removed for less than the cost of control.

The only safe method of preventing movement on steeply inclined bedding planes is by avoiding sections that are likely to be undermined by erosion and by locating the road and establishing the grade in such a manner that support will not be removed by excavation. A careful examination of the geological structure should be made in all folded or tilted areas, in advance of final location surveys, to determine where danger points are located. Alignment and grade should then be adjusted to prevent the removal of material that would cause undermining of the tilted strata.

Slides of Unconsolidated Material. Before making any attempt to cure or control a slide of this type, determination should be made of (1) its areal extent and depth, (2) the nature of the underlying structure, (3) the location of any seepage planes or underground springs, (4) the location of any perched water table which might supply sufficient capillary water to reduce soil stability, and (5) the type of soil and its shear characteristics.

From an analysis of the above information the causes of the slide can be determined and control me sures having good chances of success can be designed. Without such information the measures tried will very often fail and the money expended for them be wasted.

It is frequently difficult to obtain the information necessary to intelligently design slide control measures. The selection of methods depends largely on the conditions encountered at the site of the individual slide. Some of the common methods will be mentioned here, but the engineer must depend, for the most part, on his own ingenuity to select and improvise methods suitable for the individual case. Since a number of the important factors controlling the slide are hidden, some method of subsurface exploration is required.

Test pits and bore holes are commonly used. Their location and the material encountered should be carefully noted. A sufficient number of openings should be made to locate the source of underground springs or seepage planes and to provide a reasonably accurate cross-section of the underlying structure.

Test pits furnish the best information because visual inspection is possible, but if the depth is great and the area large the expense involved may be prohibitive.

Bore holes are sometimes made with core drills or with churn drills. Neither type of equipment is very satisfactory because water is generally used in the drilling operation, making the location of seepage planes and springs difficult. Power driven or hand operated augers provide more satisfactory information since the elevation of seepage planes and the type of soil at different elevations can be determined.

Geophysical survey methods involving the use of seismographic and electrical resistivity equipment are sometimes used for the determination of underlying geological structure at considerable depths.

Examination of the geological structure in cuts or other exposed areas near slides often furnish good indications of the location of seepage planes within the slide. Sometimes geological sections can be found in old reports which show structure now hidden by fills or slides. Such information will often save a great deal of exploration by indicating the approximate location at which certain strata will be found. The service of a competent geologist or use of the limited knowledge of geology that most engineers possess will always pay off.

The value of retaining walls, cribbing, piling and other barriers for the control of extensive slides is uncertain, unless thorough investigation has been made to determine the extent, cause, and pressure to be expected.

With this information available it is possible to design control measures. In many cases it will be found that properly placed drainage, removal of part of the material involved and the construction of proper slopes will stabilize the slide and eliminate the necessity for the barrier. In other cases economical control can be secured by the construction of a barrier after placing drainage to remove excess water.

The importance of locating the source of underground water and of placing underdrains in impervious material cannot be over-stressed. The placing of drainage at the toe of a landslide is of little or no value.

Blasting of Slides. Blasting of the underlying strata has often been recommended, and in a good many instances, tried as a slide control measure. In some situations it has been successful, but it is uncertain and may result in increasing the movement. Such measures should only be considered in cases where no other remedy can be found.

The State Road Commission of West Virginia has corrected hundreds of landslides within the past several years, and recent inventory lists five hundred forty-two (542) additional slides on primary and principal secondary roads The majority of slides included in this inventory are in the western half of the State. The Charleston District reported 180 - 33% of the total. The Weston District 78, and the Moundsville District, after an intensive correction program carried on for the past ten years, reported 50. The smallest number reported by any district was 6, in the Princeton District. Four counties, Morgan, Pocahontas, Raleigh, and Wyoming, did not report a single landslide.

Most of the landslides were in unconsolidated material, the principal exceptions being in the eastern panhandle, where most of the few reported were rock slides of the bedding plane type.

Records of the total cost of landslide control on West Virginia Roads have never been kept. This is due largely to the fact that many smaller ones are corrected as a part of routine maintenance, and also, many are corrected as a part of grading operations on new construction and the costs included in such items as unclassified excavation and drainage items.

A few examples will suffice to show that the cost of control is very heavy. The following rather serious ones were corrected by the removal of sizeable quantities of material to stabilize the slopes by load reduction:

(1)	1948	U 313 (2)	Morgantown-Star City	$70,000.00
(2)	1949	F 306(7)	Neptune-Belleville	119,000.00
(3)	1949	F 247 (5)	St. Marys-Alternate U.S. 50	113,000.00
(4)	1951	F 313 (4)	Star City-Osage	58,000.00
(5)	1952	FI 187 (17)	Huntington-U.S. 60	45,000.00

These slides were all in unstable areas where removal of support caused movement which could only be controlled by reducing the load in a sufficient amount to compensate for support furnished by the material removed by the construction of the grade.

Within the past several years the Maintenance Division of the State Road Commission has done extensive work on the control of landslides on existing roads. During the years of 1950, 1951 and 1952, special authorizations in the amount of $632,400.00 were issued for the correction of slides scattered through a large part of the state. The majority of these slides, as mentioned before, were in the western part of the state. In the Huntington District authorizations were issued for the correction of 26 major slides, the total amount authorized being $223,840. In the Keyser District, which includes the eastern panhandle of the state, only one authorization was issued, this in the amount of $1,790.00. The Princeton District did not have any authorization for slide control. In addition to the work done by special authorization, numerous minor slides were corrected by routine maintenance forces. A few examples would indicate the scope of work done on some of the more serious slide areas.

W. Va. 3	Lincoln County	$22,940
W. Va. 14	Lincoln County	26,910
U.S. 52	Mingo County	43,950
Alt. U.S. 50	Pleasants County	49,350

Where proper studies were made of the sites, underground conditions and soil types, the results of the work done by the Maintenance Division have been almost uniformly good.

The Maintenance Division has utilized various corrective measures including drainage, unloading of slopes by removal of material, barriers of piling, cribbing or retaining walls and combinations of these methods. Drainage has paid the largest dividends in satisfactory correction for each dollar expended.

The relatively slow movement of the type of landslide classified as a creep sometimes encourages maintenance forces to postpone definite corrective measures and to attempt to maintain the road surface by periodic patching of subsidences or frequent removal of material that crowds the roadway. Such treatment is never satisfactory and usually more expensive over any considerable period of time than correction. Numerous examples could be cited where landslides have been troublesome over a period of years, then after proper study, have been corrected at moderate cost. One slide involving 900 feet of W. Va. Route 2 had caused subsidence of the roadway since its construction. It was patched from time to time for about ten years before any definite correction was made. During the summer of 1943, $100.00 was spent for investigation of sub-surface conditions and the cost of placing adequate drainage to remove sub-surface water and to replace the surface was $6,400.00. The cost of maintaining the surface for the three months period preceding correction was $4,300.00. Thus the expenditure of approximately 50% more than three months maintenance cost provided effective corrective measures and reduced maintenance costs to normal. The cost of maintenance during the three months period mentioned was, of course, not representative of the costs over the ten year period since the rate of movement in this particular case had increased rapidly during several months prior to the time that correction was undertaken. Another good example was a slide on W. Va. Route 7 which had been troublesome for a number of years. It moved slowly on to the roadway from a fairly steep slope and it was necessary to remove a considerable amount of material three or four times a year in order to keep the pavement open for traffic. An examination of the site indicated that this slide was caused by a spring a few hundred feet above the road. The water from this spring had no adequate outlet and was being absorbed in the slide area. After clearing the material from the roadway in the summer of 1943, a diversion ditch was cut around the slide area which effectively drained the water away from this area. The area was then smoothed so that surface water would not be absorbed. This involved three days work by a grader and no further movement has occurred to date.

A thorough understanding of landslide movement and the conditions causing or affecting it is required to approach the problem of control. Given this understanding, the engineer can undertake the design of control measures with reasonable assurance of success.

CONTRIBUTIONS BY THE U.S. GEOLOGICAL SURVEY TO HIGHWAY ENGINEERING RESEARCH

By

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1 / Publication authorized by the Director, U. S. Geological Survey

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Indirectly, the U.S. Geological Survey contributes a great deal of research in highway engineering. This contribution is in the form of basic data that are used by others in planning, design and construction of highways and in the solution of specific research problems. It also contributes more directly to research in many ways, though the basic data just mentioned are of more continuing and widespread use.

The primary functions of the Geological Survey are to map the topography and geology, to investigate and conserve the mineral resources, and to measure the water resources of the nation. The resultant data are used in an almost infinite variety of ways by innumerable groups and individuals throughout the country; all are used extensively by highway engineers.

Our well-known topographic quadrangle maps shown the shape of the land surface in three dimensions. Such maps are now available for about 28 percent of the entire country and are being published at a rate of more than 750 quadrangles per year. They are invaluable in preliminary planning of highways and highway systems, and they provide many of the facts needed for preliminary estimates of cut-and-fill quantities, and for location of tunnels, bridges, and other sturctures. Among the useful by-products of the topographic mapping program are readily available aerial photos covering thousands of square miles, and accurate triangulation and levelling nets —in part by the U.S. Coast and Geodetic Survey and in part by the U.S. Geological Survey — throughout the United States.

The Geological Survey maintains long-term records of stream flows on most of the country's major streams and hundreds of its minor ones. These records are essential basic data for every engineer who deals with water or who, in designing bridges and other structures, has to predict future stream flows, floods, or channel changes. Our records of underground water resources are perhaps of less immediate value to the highway engineer than those of surface waters. Much of the information in Survey reports on ground water can be used, however, by the highway engineer in predicting and avoiding troublesome conditions caused by ground water. Moreover, the geologic maps prepared as a necessary part of nearly every ground-water investigation have at least as much value to engineers as the geologic maps prepared for any other special purpose.

The Survey's contributions are by no means limited to records of water resources, surface or underground. Its many years of research in this field have resulted in development of most of the principles that govern the accumulation and movement of water in soils and rocks; these principles form the basis of most present-day research by soils engineers and others who are investigating specific problems related to highway design.

The basic facts shown on geologic maps, when properly interpreted, can yield as much valuable information to the highway engineer as topographic maps or records of water resources.

The geologic map shows in three dimensions the kinds and shapes of rock bodies that form and lie beneath the land surface. These are the facts around which the engineer must build. Moreover, these rock bodies commonly are the parent materials of the overlying unconsolidated sediments — soils in the engineering sense regardless of depth or thickness -- shown on the map. The usefulness to the soils engineer of a knowledge of the distribution of various kinds of bedrock and unconsolidated materials is apparent.

Even though the U. S. Geological Survey and most State Geological Surveys have been mapping geology for many years, it is unfortunate that only about 10 percent of the United States is covered by maps on scales useful to engineers. The rate of mapping is increasing, but it will still be some decades before the mapping of the entire country is finished. Completed geologic maps however, are of immense value to the highway engineer in planning and designing roads, in searching for construction materials, in making preliminary estimates of quantities of rock and common excavation, and in many other ways.

The Geological Survey's direct contributions to research in highway engineering would make an imposing and creditable list, even though these contributions are less important in the long run than the total of the three kinds of basic data described above — topographic maps, water resource records, and geologic maps.

Not many of us, even within the Geological Survey, know that almost 60 years ago this organization published the results of research on the application of geology to highway engineering. In 1895, when asphalt paving was used only in a few large cities and macadam highways were just beginning to replace mud and corduroy, N.S. Shaler published a work on "...geology of the common roads of the United States." In this report he described the kinds of rock materials that were most suitable for road building and classified the entire United States with respect both to availability of materials and to foundation conditions. In the following year he published a second report, this one on "The Geology of the road-building stones of Massachusettes." This report is notable as one of the first attempts to correlate the results of physical laboratory tests on rocks with their geologic origin — and with actual performance on the highway.

Unhappily, this excellent early start was not continued, except sporadically. During the intervening years, however, the Survey has contributed heavily to the enormous fund of knowledge that we now have. It pioneered, for example, in the investigation of clay mineralogy, which only recently has come to play such an important part in the work of the soils engineer. It pioneered, too, in the development of the geophysical methods that are now used so widely in the search for oil and ore--and that are applied increasingly by highway engineers.

It is gratifying to us that in the State where Shaler pioneered in highway geology so long ago, we now have, and have had for 15 years, a cooperative program with the Massachusetts Department of Public Works. The ultimate aim is to prepare adequate geologic maps of the entire State, but the order of priority in mapping is based largely on highway needs. A large proportion of our work, moreover, is devoted to detailed geologic mapping of strips along proposed new highways or relocations. After the highway engineers have used and acted on the facts thus assembled, the strip mapping is enlarged to cover entire quadrangles.

We also have somewhat similar but less elaborate cooperative programs with several other state Highway Departments. These programs range from county-by-county mapping of materials resources, through exchange of geologic data for laboratory testing of our rock and soil samples, to general or specific advice on the application of geologic knowledge to highway engineering.

We have investigated and mapped a great many landslides, partly in the course of ordinary geologic mapping and partly as special studies. The most notable of these is a continuing investigation of the hundreds of active slides around the shores of Lake Roosevelt, behind Grand Coulee Dam. These investigations are not only contributing to an understanding of landslide processes, but their results are applied almost daily to problems of relocating highways, docks, buildings, and other structures.

As a result of the Survey's widespread interst in landslide problems, it is taking a leading part in the Highway Research Board's Committee on Landslide Investigations. This Committee is compiling a volume on "Landslides and the Highway Engineer." We hope that this volume will contain most of the information that the engineer needs to know to recognize and correct landslides.

In summary, the Geological Survey contributes basic data, in the form of topographic and geologic maps and water records, that can be used by highway research engineers in solving many of their problems. Subject to appropriations and the will of Congress, we hope and expect to continue to provide such data in ever-increasing quantity and quality. We hope also to continue to contribute directly to research in this field. As in the past, however, we feel that much of this research should be of a pioneering nature. That is, we should endeavor to show the way toward new ideas in science, toward new techniques, and toward new uses of basic data. Development and application of those ideas, techniques, and uses should be turned over to others.

DESIGN OF DEEP ROCK CUTS IN THE CONEMAUGH FORMATION

By
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Introduction

It is a pleasure to be able to take part in this Symposium on Geology as Applied to Highway Engineering although I may be somewhat out of place because the examples which I shall use are those derived from my experience in the design and construction of deep cuts for spillways and railroads and not for highways. In some ways I approach the problem of cut design from a different point of view than those in the highway business and yet the fundamentals of the design are common to both of our viewpoints. The major differences lie in the purpose for which the cuts were made and the greater depths of the cuts. My examples of design will be drawn from cuts 200 feet to 300 feet in depth. Cuts of these depths in the Conemaugh formation were without precedent when we designed the Youghiongheny spillway cut in 1941 and required considerable investigation. I shall discuss certain elements of the design of Youghiogheny cut as well as the East Bow Cut on the Conemaugh Division in Pennsylvania and the Tunnel 10 cut on the Panhandle Division in Ohio of the Pennsylvania Railroad. I hope my observations will be helpful to you in the highway field where cuts in the 200 to 300 foot range are now becoming necessary with the higher requirements of modern highway design.

These remarks will deal with four points in the following order:

1. The characteristics of the rocks forming the sedimentary cycles in the Conemaugh formation.

2. The effect of subsurface drainage on the stability of rock cuts.

3. The design of rock cuts.

4. Construction of rock cuts.

It has been known for some 20 years that the rocks of the Conemaugh formation in Western Pennsylvania, West Virginia and Ohio were deposited in a somewhat variable but not entirely dissimilar succession of units each one of which is composed of several different beds of rock. These units have been called sedimentary cycles or cyclothems. With an average thickness of 60 feet, several cycles may be found in a deep cut and the problems of design associated with each bed in a cycle will therefore be repeated with the repetition of the cycles.

A cycle in the Conemaugh formation usually, but not always, is composed of the following rocks, form the base upward: sandstone, shale, fresh-water limestone, under clay, coal, shale, marine limestone, and shale. The sandstone may be hard massive rock. The shales and under clay may be soft to medium hard and thin bedded to massive. The limestones range from pure limestones to nodular limy shales or clays. The sandstones, sandy shales, and carbonaceous shales may be jointed and fractured. The weaker members may be cut by numerous, random, slickensided surfaces. The coal is generally blocky and the limestones usually have a fracture system. None of these joint systems necessarily parallels the system of the next member. The rate and degree of weathering vary with each rock type. Localized zones of weathering are found at the base of the pervious members. Inherently soft materials are found at the top of the impervious members underlying pervious members. The result is that, from the standpoint of design, the rock section is heterogeneous. Sampling of each bed or type of bed is required to produce data upon which to base the design of that portion of the slope controlled by the characteristics of that bed.

Generally the sandstones, jointed carbonaceous shales and sandy shales, limestones and coal are somewhat permeable with the sandstones carrying the most water. The indurated clays or clay stones and the under clay are relatively impermeable.

The behavior of these rocks in cuts below the base of weathering is summarized as follows:

a. Fresh sandstones and siltstones will stand out in slopes which are not steeper than their joints, provided that the underlying rocks are stable.

b. Carbonaceous and other shales coarser than clay sizes behave somewhat like the sandstones. If overlain and protected by sandstone, they are usually jointed with the sandstone and will assume its slope.

c. Coal seams, if overlain by carbonaceous shales or sandstone and underlain by fire clays will tend to assume a slope parallel to their joints.

d. The indurated clays (clay stones) are separable into three types.

1. The under clay of the coal seam is commonly a gray very weak, poorly cemented nonresistant material which will hold no slope and will flatten out to whatever slope the opportunity affords to 1 on 5 flatter. If cut in a steep slope, it will tend to undercut, leaving the coal overhanging.

2. Fine grained, soft, red and gray indurated clay is somewhat more resistant than the fire clay immediately beneath the coal seam and, in general, will take a slope of 1 on 1 although it will tend to undercut the overlying resistant beds. A rate of erosion of 0.5 foot per year has been measured on such material where it was cut vertically beneath a resistant sandstone.

3. Massive, sandy, well-cemented, indurated clay has been observed with slopes as steep as 5 to 1, but is commonly not stable on slopes steeper than 2 to 1. This material fails through overstress along vertical fractures and tends to preserve a steep slope which retreats slowly.

e. The clay shales are mainly compaction shales and assume slopes of about 1 on 1 unless extremely well jointed and cemented in which case steeper slopes develop.

f. The nodular limestones generally break in the planes separating the nodules and behave similarly to sandstones in cuts.

We can restate this as follows: Rock falls will develop in cuts through the resistant beds: sandstones, limestones and jointed cemented shales. Ravelling will occur in the indurated clays and soft compaction shales.

Effect of drainage on stability of rock cuts.- As we all know, the stability of a given earth slope may be dependent upon drainage, for instance the downstream slope of an earth dam. It is less apparent that rock slopes may be similarly dependent upon subsurface drainage for their stability. There is one example of a landslide in the bedrock of the Conemaugh formation which seems to have been due to lack of subsurface drainage which I shall describe.

The Brilliant cut, on the Pennsylvania Railroad, Conemaugh Division at Washington Bourlevard and Allegheny River Boulevard, Pittsburgh, Pa., is the case in point. It was excavated on an average 1 on 1 slope across the end of a hill to a depth of 200 feet. Immediately back of the top of the cut a vertical joint opened slowly over a period of years, was plugged with concrete but continued to open. In March 1941 a slump slide of approximately 100,000 cubic yards occurred with the open joint as the rear boundary. The toe of the slide heaved the near tracks indicating that the base of the slide was curved in section similar to the classical failure in soil. I believe that this slide was caused by the hydrostatic pressure in the open joint which would normally have been relieved by drainage along the horizon of the Harlem Coal but the outlets of this drainage were then frozen. Probably the hydrostatic pressure had been increasing annually as the rear joint opened and deepened progressively. It is conceivable that the opening varied in width annually and that the slide block rocked on its base weakening the indurated clays (clay stones) which formed its foundation until the shearing stress imposed by the hydrostatic head of over 190 feet caused the failure. I believe that this slide points the finger at the necessity to consider subsurface drainage in cut deisgn.

Design of rock cuts

There are 5 elements, exclusive of cost, to be considered in the design of a deep rock cut in the Conemaugh formation. These elements, in the order in which we shall discuss them, are:

1. Purpose
2. Slopes vs. rock types, with or without weathering
3. Number of sedimentary cycles
4. Drainage
5. Berms

The purpose for which the cut is to be made will fix the general considerations governing the treatment of the remaining elements in the design. For example. If the purpose of the cut is to strip a bed of coal so that it may be loaded out by shovel with the excavation to remain open only for a very short time then the cut slope may be nearly vertical regardless of the type of rock in the high wall. Or if the cut is to border a rarely used spillway, rock debris may be tolerated on the spillway floor provided no major slide or failure of the cut occurs. But if the cut is for a limited access thruway then neither debris on the road or a failure of the cut can be tolerated. Similarly a railroad cut must be designed for stable slope which would yield little or no debris to fall on the road bed. The purpose of the cut therefore fixes the general design criteria and implies the desired behavior of the finished slope.

Slopes versus rock types, with or without weathering

The several rock types in the Conemaugh formation differ greatly in their physical properties, as you know. The greatest difference lies between the sandstones and the indurated clays (claystones) or fire clays. The sandstones are at least 5 times as strong in compression as the indurated clays and in point of resistance to weathering many, many more times stronger. Not quite to infinity for most of the indurated clays will withstand one cycle of wetting and drying. In shearing strength the sandstones average at least 3 times as strong as the average of the indurated clays. That is those indurated clays that don't fail in the shear blocks before the load is applied. These variations in strength and durability are reflected in common outcrop or lack of common outcrop on the hillsides and in variation in slopes in old cuts in quarries, roads and railroads. With the numerous examples of the normal slopes which develop in time on the several rock types one would be foolish to design a slope without reference to the rock type to be exposed on that slope. Therefore the rock type will determine what slope may be designed safely at any given place on a cut.

Now we may be conservative and design the slope on 1 to 1 regardless of rock types and come out with a pretty good cut. In so doing we may have moved a large yardage of rock for no gain since we might have designed a satisfactory cut using steeper slopes on the more resistant rock. Let us assume we are beneath the weathered zone and in fresh unweathered rock. In that case we could seriously consider using the following slopes:

a. 4 on 1 or 1/4 to 1 on massive sandstones; siltstones; carbonaceous shales coarser than clay size; and coal if protected by a resistant rock cover.

b. 2 on 1 or 1/2 on 1 on sandy shales or cemented silt shales or cemented indurated clays.

c. 1 to 1 on fine, non-carbonaceous shales with consideration being given to even flatter slopes.

d. 1 to 1 on other rock.

Supposing we are dealing with weathered rock, the slopes should be flattened to a general 1 to 1 slope in the shales, siltstones and sandstones unless the latter are massive. In the indurated clays design slopes of 1/3 to 1 or 1 on 3 may be required.

Number of cycles. This may be a somewhat complicated way of expressing the thought that the final shape of the slope may consist of a series of steep rises and gentle steps over a succession of alternating resistant and non-resistant beds. In the Conemaugh formation such would not be uncommon but it would not be mandatory. Let us assume a two cycle condition below the weathered zone. We would then have 2 sandstone beds each overlain by a succession of shales, indurated clays coal and carbonaceous to sandy shales with a stray thin limestone or two thrown in for good measure. If the cycles were identical the several slopes on each would be identical with the corresponding slopes on the other. The design would be repeated. We can see that shortly in one of the photographs.

Drainage. The water in the ground behind the face of the rock cut has considerable influence on the behavior of that face under certain conditions. If free drainage is afforded by previous layers or by fracture channels of ample capacity, no head will be built up behind the face. However, if undrained fractures lie behind the face and become water filled, hydrostatic loading conditions develop against the mass of rock lying outward from the fracture. It is then not a problem of surface ravelling and rock falls that confronts the designer. It is rather the provision of adequate subsurface drainage for the prevention of mass failure of the triangular section of rock lying outward from the water filled fracture. Most rock cuts are shallow and the maximum hydrostatic pressure which can develop is well below the shearing strength of the rock but the higher rock cuts in the 200 to 300 foot range might permit the development of heads exceeding the shearing strength of the weaker shales and clays. That seems to have happened at Brilliant. Therefore these deep cuts should be examined from this standpoint: Is natural drainage present? If not, subsurface drainage should be provided.

Berms. Berms are used by many designers as catch alls for rocks and ravellings from the slopes above. As such they are of great value. But not only do they catch rocks they also serve as ponding and infiltration areas for surface water coursing down the slopes. This water may find entrance into the rock behind the slope and with ineffective drainage below, serve to build up the head in the vertical fracture system and to no good end, either. Therefore the berms should be pitched to drain unquestionably, unless lying on or capped with impervious material, and systematically cleaned and maintained.

The location of a berm appears to be a matter wide open to discussion, if the design of highway slopes near Pittsburgh is representative of current thinking on this subject. Some of them are on shales; some are on sandstones; some are high above the road bed and others not far above the ditch line. There seem to be no fixed criteria governing the placing of berms, either with respect to vertical or geologic position.

Rather than undertake a critique of other people's designs without all the facts which were available to them, let me comment on three cuts in the Conemaugh formation for which I have had some responsibility. These comments will state my position on the design of deep cuts in the Conemaugh.

East Bow Cut. The East Bow on the Conemaugh Division, Pennsylvania Railroad is a 200 feet high, side hill cut on a main line double track freight railroad constructed under the direction of the Corps of Engineers 40 miles east of Pittsburgh. Most of the cut is thru the red indurated clays and shales of the middle Conemaugh formation, however, the top of the cut is formed by about 20 feet of weathered and fractured sandstone. The slope is 1 to 1 from a 7 foot wide track ditch to the top of the cut. This is nearly uniform soft rock cut now about 6 years old.

Youghiogheny spillway cut. The Youghiogheny dam of the Corps of Engineers is a 180 foot high earth dam with a gated tunnel and a 400 foot wide uncontrolled spillway located near Confluence, Pa. about 80 miles south east of Pittsburgh. The spillway cut required considerable investigation in its design because it was without precedent both in its location adjacent to an earth dam and in its proposed height of over 300 feet. Much of our basic knowledge of rock behavior in cuts was developed in this investigation. The purpose of this cut dictates over all stability but minimizes the necessity for preventing rock falls and ravelling. Therefore, the slope is constructed as steep as the rocks would permit and without berms to catch runoff. There are two unweathered sedimentary cycles with steep slopes on the sandstones and carbonaceous shales, and flatter slopes on the softer shales. Horizontal drain holes at the bases of the pervious layer of the sandstone were drilled landward to beneath the top of slope to drain the full triangular section of the cut. The cut is now 10 years old.

Tunnel 10 Cut. The Tunnel 10 cut on the Panhandle Division, Pennsylvania Railroad, about 30 miles west of Steubenville, Ohio, is a 200 foot deep thru cut on a main line double track railroad. This cut required overall stability plus a minimum of ravelling and rock falls. The adjacent tunnel portal excavation, over 70 years old, permitted approximation of the rock weathering and drainage conditions. The cut has three berms, the lower two of which are located beneath sandstone beds to catch the sandstone falls and the upper one perched high on the face to catch the soil slump. Drainage is prevalent thru the upper sandstone and is directed to the ends of the cut along the middle bench on top of an impervious indurated clay. The steep slopes in the sandstones and the two benches are omitted in the deeply weathered rock of the east end of the cut where a uniform slope of 1 in 1 is used.

May I suggest that during construction the intercepting ditches at the top of slope be constructed prior to commencing the main cut and maintained in working order along with the berms throughout the construction period. Scaling and subsurface drainage measures if required should accompany the excavation and not be left until the last although the scaling may be greatly reduced if the powder man minimizes his shots along the slope lines

My general principles in cut design may be summarized as follows:

1. In so far as practicable the cut should be explored thoroughly by core borings using M series core barrels and by careful geologic study both of the cut area and the adjacent terrain to ascertain the geologic conditions including the rates of weathering of the several rock types to be exposed and the ground water conditions.

2. The design of the cut should then be based on those findings plus the purpose of the cut.

3. This design should be subject to modification as the excavation proceeds so that the cut as finally constructed conforms to the actual geologic conditions.