Proceedings of the

Seventh Annual Symposium on

"GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING"

February 24, 1956

Sponsored by

N. C. STATE HIGHWAY AND PUBLIC WORKS COMMISSION

Conducted by

N. C. STATE COLLEGE

RALEIGH

Foreword

The Annual Symposia are informal meetings at which subjects of mutual interest to geologists and highway engineers are presented and discussed. The value of geological sciences and procedures to the planning and construction of highways has become recognized in many areas but the fullest efficiency of their use has not been developed. The exchange of ideas always aids in more rapid advancement and the avoidance of errors and it is to this end that the symposia were originated by Mr. W. T. Parrott, Geologist for the Virginia Department of Highways. The first Symposium was held in the Highway Department's Auditorium in Richmond, Va., in 1950.

Extreme latitude is given the Sponsors as to the arrangements and every effort is made to keep the Symposia simple to arrange and conduct. Central themes are avoided and diversity of subjects encouraged. Practical aspects are considered highly important and involved scientific treatises have no place in the Symposia. It is felt that these aims have been accomplished in all of the Symposia held thus far and the continued interest and attendance recommends that even greater efforts to advertise them are justified in the future.

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SYMPOSIUM ON GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING

RALEIGH, N. C.

FEBRUARY 24, 1956

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1956 Program Arrangements

The Symposia have always been held in a one-day programs that leave very little time for the personal contacts and development of acquaintanceships which, frequently, form the most profitable part of the conference or meeting. With this in mind there was an innovation at the Seventh Annual Symposium on Geology as Applied to Highway Engineering in the form of a Smoker the evening before the formal program to which all who registered were invited.

The refreshments were provided by the Superior Stone Company, Raleigh, N. C., and the Becker County Sand and Gravel Company, Cheraw, S. C. A large number of those who attended the Symposium were on hand, and most of the Speakers, thus making it possible for the Speakers to feel more at ease in their approach to their subjects and to fit the expressed interests of their prospective listeners at the next day's sessions. All declared the social hour to be a definite success. It was held at the Sir Walter Hotel in downtown Raleigh whereas the program was held in the Riddick Engineering Building on the North Carolina State College Campus.

The Welcome to State College was given by Chancellor Carey H. Bostian who was introduced by Dean J. H. Lampe, School of Engineering. The Response was made by Chairman A. H. Graham, North Carolina State Highway and Publics Works Commission.

Presiding over the morning session was Dr. Ralph E. Fadum, Head, Department of Civil Engineering, N. C. State College, and at the afternoon session the presiding officer was Dr. Jasper L. Stuckey, North Carolina State Geologist.

Between the morning and afternoon sessions lunch was served in the College Dining Room following which there was a color motion picture shown, "Relocation of U. S. 70 Between Ridgecrest and Old Fort, N. C." The film was produced by J. P. (Pete) Bourke, Roadway Photographer for the State Highway Commission and showed the construction of the spectacular highway across the mountain from clearing to final surfacing.

The Staff and facilities of the College Extention Division of North Carolina State College who performed the details of printing, publicity, registration and many other essential chores are hereby acknowledged with the thanks of the Sponsoring Committee.

Landslides and the Engineer

by
Robert F. Baker

INTRODUCTION

The role of the engineer in the solution of landslide problems has never been well defined. For many years, railroad and highway administrators have charged him with responsibility for their proper treatment, but have frequently relied upon a geologist or an engineering geologist for guidance in the solution of large-scale mass movements. Unfortunately, the theory for scientific treatment was very sketchy and investigators were forced to rely almost completely upon experience.

Geology produced excellent qualitative data, but a correspondingly accurate quantitative approach was not available. Since engineering is founded upon analyses that relate driving and resisting forces, it was natural for engineers to devote more and more attention to the development of a reasonably rational procedure based upon the fundamentals of engineering mechanics. Furthermore, the advent of large earth moving equipment, bringing with it a more favorable price for excavation, made slope stability a more frequent consideration.

With the increasing number of slope stability problems, the highway engineer was faced with a choice between a qualitative or empirical method and the highly theoretical mechanics approach. Almost without exception, assistance from the geologic viewpoint was requested. The degree of success was varied. When help came from the engineering-geology field, progress was generally good. However, for the academic or purely scientific geologist, the transition to the requirements for a practical solution came with considerable difficulty. In some instances, this latter condition led to the selection of engineers with a soil mechanics background. Within the soil engineering field, interest in slope stability was great, and numerous engineers became interested from a fundamental viewpoint.

With both geologists and engineers active in the studies of landslides, it has been increasingly more common to encounter uncertainty as to the most desirable organization for problems related to the two sciences. It is the purpose of this paper to discuss possible interrelationships, and to cite a typical landslide study by way of example.

RELATION BETWEEN GEOLOGIC AND ENGINEERING PRINCIPLES

In order to clarify the discussion that follows, certain terminology needs clarification. A landslide* is defined as "downward and outward movement of slope-forming materials composed of natural rock, soils, artificial fills or combinations thereof". Falls, slides, and flows, are the three principal types of landslides, and can be differentiated simply on the basis of the forces and resistance to movements*. Falls are masses that move down the slope in leaps and bounds, largely under the direct force of gravity. Slides are those movements for which a significant shearing resistance exists at the slip-surface, and flows are landslides for which little or no shearing resistance is available along the surface of separation.

A landslide problem will be used in a rather specific sense to differentiate between an academic or pure science approach and a specific set of conditions that require a practical solution. Geologic principles are defined as those data that describe, in a qualitative fashion, the history of the area, the development of the landslide, and the causes and contributing factors to movement. Engineering principles will refer to those factors that produce, in a fairly direct manner, quantitative values that delineate resisting and motivating forces as well as the economics that are involved.

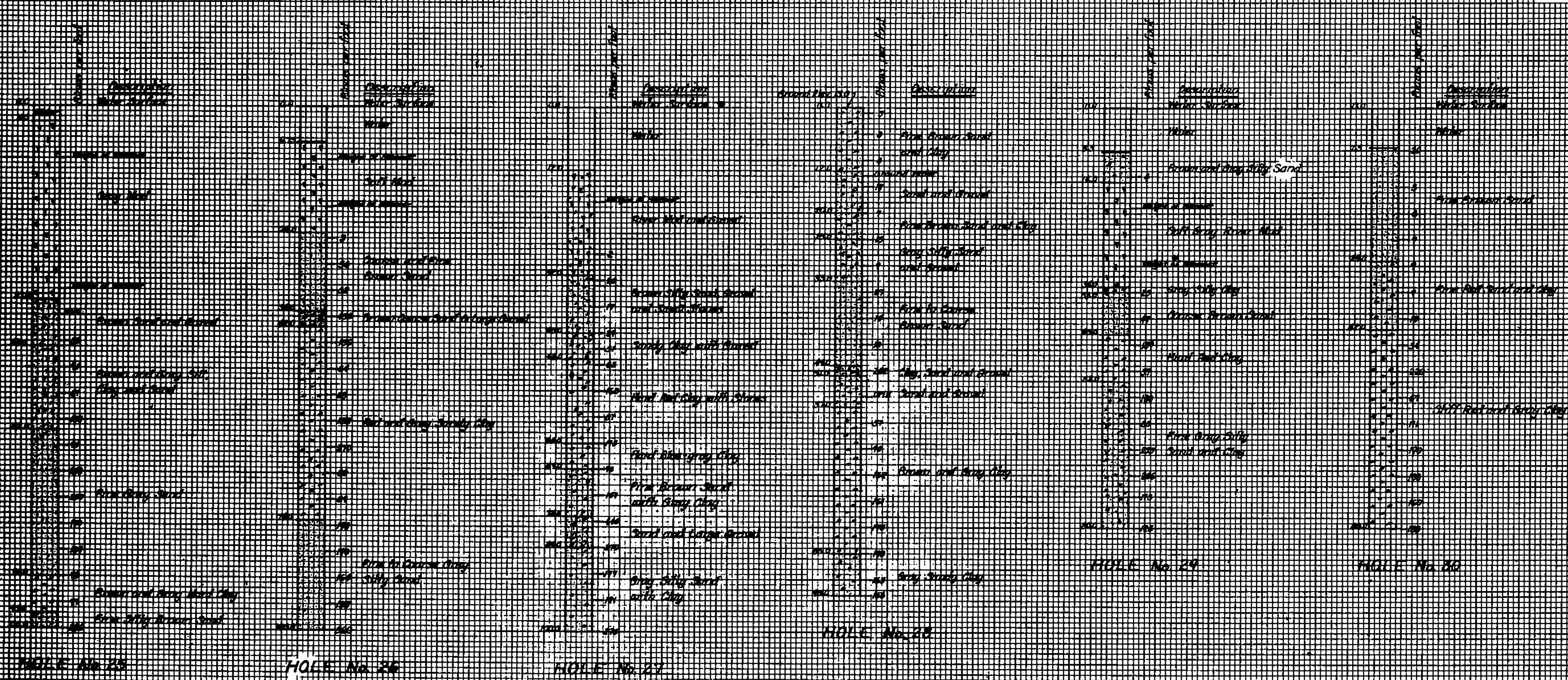
It is quite obvious that geology and engineering will overlap, particularly in practice. The borderline situations involve very argumentative factors. However, since precise delineation is not important for the purposes of this paper, no detailed differentiation will be attempted.

One perspective of the relative position of geologic and engineering principles in a landslide study is contained in Table I. The steps in the investigation were outlined, and the use of basic principles from the two sciences were considered. An effort was made to delineate between the degree of importance of basic principles at various stages in a study. Rather arbitrarily a "major" or "minor" status was assigned. There is very little real significance to Table I, although two conclusions might be drawn: one, geologic principles are extremely important in the investigational and analytic phases, while engineering principles are particularly necessary in the analytical and solution stage; and, two, neither of the two sciences should be ignored during any phase of the study.

* Definitions from "Landslides and Engineering Practice" a forthcoming book prepared by the Committee on Landslides Investigations of the Highway Research Board.

JONES POINT BRIDGE

PROJ. | SHEET 6 OF 8

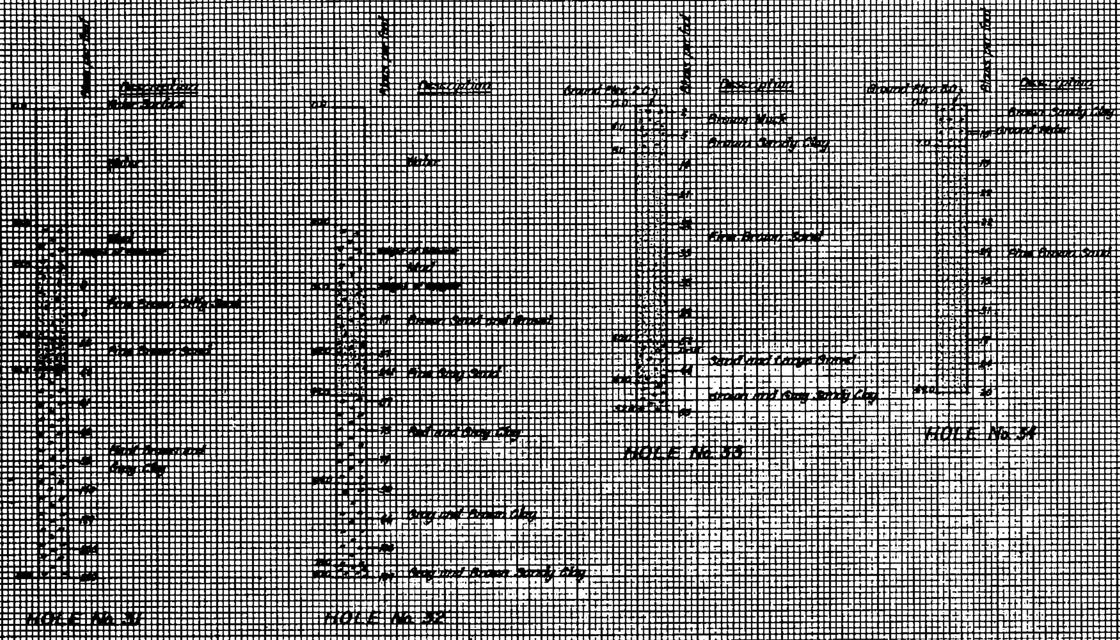


NOTE: Blows per foot refers to a 300 pound hammer falling a distance of 18 inches.

BORING LOGS

JONES POINT BRIDGE

PROJ. | SHEET 7 OF 8



NOTE: Blows per foot refers to a 300 pound hammer falling a distance of 18 inches.

BORING LOGS

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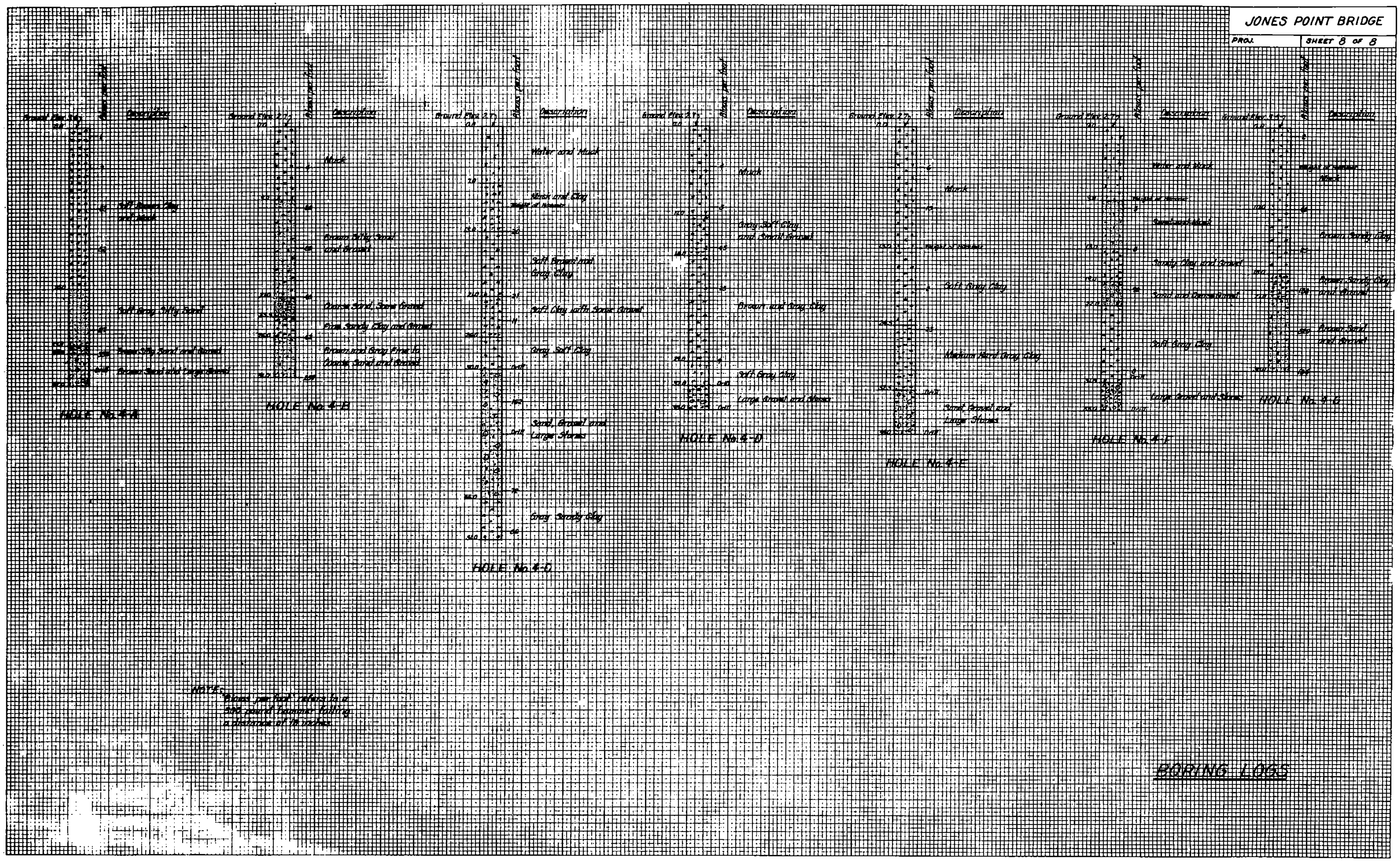

JONES POINT BRIDGE
PROJ.
SHEET 8 OF 8
Ground Elev.
Blows per foot
Description
Muck
Water and Muck
Soft Brown Clay and Muck
Brown Silty Sand and Gravel
Coarse Sand, Some Gravel
Soft Gray Silty Sand
Gray Soft Clay and Small Gravel
Soft Brown and Gray Clay
Soft Clay with Some Gravel
Brown and Gray Clay
Gray Soft Clay
Soft Gray Clay
Medium Hard Gray Clay
Large Gravel and Stones
Sand, Gravel and Large Stones
Gray Sandy Clay
Brown Sandy Clay
Brown Sand and Gravel
HOLE No. 4-A
HOLE No. 4-B
HOLE No. 4-C
HOLE No. 4-D
HOLE No. 4-E
HOLE No. 4-F
HOLE No. 4-G
NOTE: "Blows per foot" refers to a 300 pound hammer falling a distance of 18 inches.
BORING LOGS

If one accepts the alignment given in Table I, the engineer would be responsible for the development of the solution. In smaller organizations, this would be tantamount to delegating total responsibility for the study to the engineer. For larger companies or agencies, the work could conceivably be divided into two parts, with a geologist in charge of the investigational stage and an engineer heading the analytic work. If an engineering problem is involved, undoubtedly the recommendation would be reviewed by an engineer with a good geologic background. Under any arrangement, competent personnel from both a geologic and engineering viewpoint are a necessity if the best answer is to be obtained.

TRANSLATION OF THE RESULTS OF GEOLOGIC STUDY INTO ENGINEERING DATA

There is a wealth of data available for scientific analysis in the geologic literature. Case histories are numerous, many of which contain a great deal of detail. To the engineers who are introduced for the first time to landslides, this literature has little value in its present form beyond broadening the vision and, in some instances, furnishing experience which permits more rapid proficiency for empirical solutions.

However, there are two major difficulties related to rapidly and accurately absorbing the data in the literature. One lies in the diversity of classification, terminology, and technical details that are reported, and the second is related to the human element which tends to (1) produce results of the successes and not the failures and (2) promote premature discussions of a given analysis and treatment.

As a result of the first mentioned difficulty, an engineer is required to read and observe extensively before he learns what a "debris avalanche" really is, and whether or not the type occurs with frequency in the region in which he is interested He may read of the dangers of "serpentine" or the "Conemaugh Shales" but not be able to translate this knowledge into quantitative values. If he then neglects considerations of driving and resisting forces, he may become completely "lost" as to how to deal with these troublesome formations beyond avoiding the areas, if possible, or philosophically paying for damages that result. The availability of experience and descriptions related to these critical formations permits translation into "relative" stability values and into sources of restraint and motivation. With study, good quantitative data should be forthcoming.

As to the human elements involved with reporting landslide experiences, a follow-up study of as many of the problems as possible would produce some data as to reliability and permanence of the treatment method. Additional efforts might be needed to learn the details related to unsuccessful treatments.

Another phase of the problem of translating geologic data into engineering factors lies in the determination of the quantitative implications of results dealing with the historical, structural, geomorphic and physiographic aspects. Certainly some reflections of these factors will be present in the forces that are acting upon the landslide mass. The relationship may be in the shearing resistance of some of the materials, or it may be significant in terms of hydrostatic pressures. Even with our lack of knowledge for a complete translation, some estimates of the influences on the mechanics of the movement are possible. The full value of the data derived through the pure geology sciences will not be realized until a method is available for its conversion into quantitative effects upon resisting and driving forces.

EXAMPLE OF A LANDSLIDE INVESTIGATION

The example of a landslide investigation that is discussed in the following pages is not a model for such problems. It was selected because it represented the most complex and extensive series of studies undertaken by the author while working with the West Virginia Road Commission during the period 1949-1954. The adequacy of the recommendations will not be considered, since construction work on the project has not been completed. The general technique used for the studies is the primary source of interest, although the absence of a performance history prevents an analysis of the interpretation of the data.

The problem arose during the planning stages for approximately three miles of new location for U.S. 60, East of Charleston near the town of Bell in Kanawha County, West Virginia. The proposed location was along the back edge of the Kanawha River terrace. Since a toe-excavation of the talus deposits of this area will invariably lead to a landslide, the study was assigned in October, 1953 to the Department of Soil Mechanics by the State Construction Engineer, who at that time was Mr. George W. McAlpin. The study was completed in June, 1954 and a total of $13,104.98 was estimated as having been spent. Less than one-half mile of the proposed location was situated so as to be free of landslide problems. Figure 1 shows the West end of the project which contained the most critical sections. This mile of the project will be discussed in more detail in the following.

With such an obvious large-scale slide problem, one might wonder why a better location was not selected. A long-range study had been made in previous years. The presence of the railroad (Figure 1), a large industrial plant between the railroad and the Kanawha River, as well as rugged, irregular slopes to the North made all other locations unfeasible. Certainly the three miles under consideration were the most troublesome of the forty-mile stretch that was involved.

The preliminary investigation included a review of a set of plans prepared in 1948. Airphotos, available from the U. S. Department of Agriculture, were studied, as well as county geologic maps prepared by the W. Va. Geological Survey. After a brief field examination, the project was divided into five areas, based on similar conditions as to landslide potential. The detailed field investigation that followed included auger and core boring, electrical resistivity measurements, accurate locationing of exposed bedrock, extension of cross-sections (sometimes as much as 500 feet transverse to centerline and 200 feet higher than centerline), accurate locationing of electric towers and pipe lines, and other miscellaneous items. The office study consisted principally of assimilating the data, plotting of cross-sections, and analyzing the problem from a treatment viewpoint.

In addition to the author, the technical staff that was available to work on the problem included: a civil engineer who coordinated all activities and was in direct charge of the analysis; a geologist who assisted in the planning of the field studies, conducted part of the electrical resistivity tests, prepared a general geologic report, and reviewed the analysis and the recommendations; and a young civil engineer who directly supervised the field investigation. A special series of electrical resistivity tests were conducted by Mr. R. W. Moore, U.S. Bureau of Public Roads.

Due to the rugged terrain, several miles of access roads for the drills were cut by a bulldozer obtained on a rental basis from the Maintenance Division. The Department's own auger drill, core drill, and electrical resistivity equipment were used. Samples and rock core were taken, tested and classified in the laboratory. Approximately 119 auger holes, 10 core drill holes, 46 electric resistivity locations, and 3 miles of dozer roads were involved. The approximate total footage was 1500 feet by auger, and 300 feet by core drilling.

Geologic principles were applied to advantage throughout the study. During the early stages, the general geology of the area was defined. Core borings were located so as to produce the geologic column shown in Figures 3 and 5. The location of the Warfield Anticline near the middle of the project indicated a dip of two feet in one thousand feet toward both ends of the project. Geomorphic principles aided in the identification of existing and potential slides and flows, and greatly facilitated the selection of drilling sites. Interpretation of electrical resistivity results, as well as aid in the locationing of test sites, was also governed largely by geologic aspects. The following quotation from the geologic report is of interest: "The presence of the massive sandstone explains the behavior of the hillside with reference to the landslide problem. In the past, the sandstone has been slower weathering than the shale, thus producing a bench. In many instances, the soil mantle moves down and spills over the sandstone as water saturates the material. Subsurface water can be anticipated at an elevation of 700 to 710 near the coal and underclay. Other coal seams can be anticipated above an elevation of 800 and near 770". This statement, plus similar data, had a great deal to do with the final recommendations that were made.

During the early stages, the primary engineering work consisted of cross-section survey; layout of dozer access roads, accurate locationing of drill holes, power lines, etc. and general coordination of equipment and personnel. For the analysis, an engineer studied various methods of treatment, but the influence of the geologic factors received due consideration.

Some of the recommendations will be of particular interest. It might be pointed out that essentially three dangers existed as follows:

(1) Undermining of upper layers, thus producing slides, flows, and falls onto the roadway.

(2) Undermining of electric towers and pipe lines.

(3) Slides developing below the road so as to encroach upon the railroad.

From a monetary sense, the potential damage to the utilities were by far the most troublesome to analyze and to correct.

In Figure 2, typical cross-sections are included to illustrate two principles that were employed. Where excavation could be avoided (over a fairly extensive stretch), the grade line should be raised (as shown for Sta. 369+50) even though the cost of a crib-wall might be required to protect the railroad. This solution was recommedned for a total length of 2500 feet near the West end of the project. One major landslide (near Sta. 367+00) could thus be avoided, or at least, the condition would not be aggravated. At Sta. 390+00, bedrock was located near the planned grade line at center-line. Benches were recommended to intercept the debris produced by rock weathering and the relatively thin soil mantle that is likely to "spill over" the sandstone. The latter design was recommended for extensive areas.

In Figure 3, a slight variation in the benching of bedrock is shown (Sta. 407+50). Note that where an electric tower was founded on bedrock no special consideration was required. For Sta. 378+00, extensive benching in soil was recommended to prevent future slides onto the roadway. Stability analyses were used to estimate the height between the benches, and the width of the benches. This area was near a section where a grade change was recommended so the heavy excavation does not represent waste material. The soil in the slide was adequate for embankment, according to laboratory tests.

Figure 4 shows an interesting type of solution that was applied. In the lower left corner, a three-sided "box" is indicated around certain towers where undermining was feared from one side or from beneath. The tower near Sta. 380+00 (See Figure 1) is in a good position to be undermined by the spread of the slide. The lines of piling that formed the "box" were to be drilled into the bedrock, and if slides did develop, the tower would be left on an "island".

Where these solutions were suggested, the soil was shallow (less than ten feet) and steep rock with a shallow soil mantle lay up the slope from the tower.

In the upper part of Figure 4, anchored piling is also recommended to isolate a critical pipe line area. Across this section, an embankment was called for. However, back of Sta. 408 and ahead of Sta. 412 excavation was necessary. It is believed that the line of piling will provide a shear plane, across which the movement will not spread. The exposed bedrock assured that the slide would not completely envelope the piling installation and sever the pipelines.

Perhaps the most expensive installation that would be threatened by the highway construction is shown in Figure 5. As can be seen in Figure 1, a slide exists on both sides of this large electric power installation. Earlier plans had called for a crib-wall as indicated in Figure 5. However, drilling showed that the towers were not founded on bedrock. Furthermore, probabilities seemed remote that a crib-wall would be able to withstand a movement as extensive as would be the combined unstable areas to the east and west (See Figure 1). No solution that was reasonably safe, was considered less costly than moving the towers, and such a recommendation was made.

In addition to these types of recommendations, the existing condition of the utilities with reference to potential slide damage was discussed. Numerous towers were in imminent danger of collapse. Others would be stable, regardless of the State Road Commission's activities. Finally, some would be endangered through the construction of the roadway. It was believed that the classification would assist in the legal aspects of the problems.

SUMMARY

An increasingly larger number of highway landslide problems are developing due to the expanding highway and airport programs. The improvements in earth moving equipment have resulted in more and more large-scale excavations and embankments and this factor has also added to the quantity of slope stability problems.

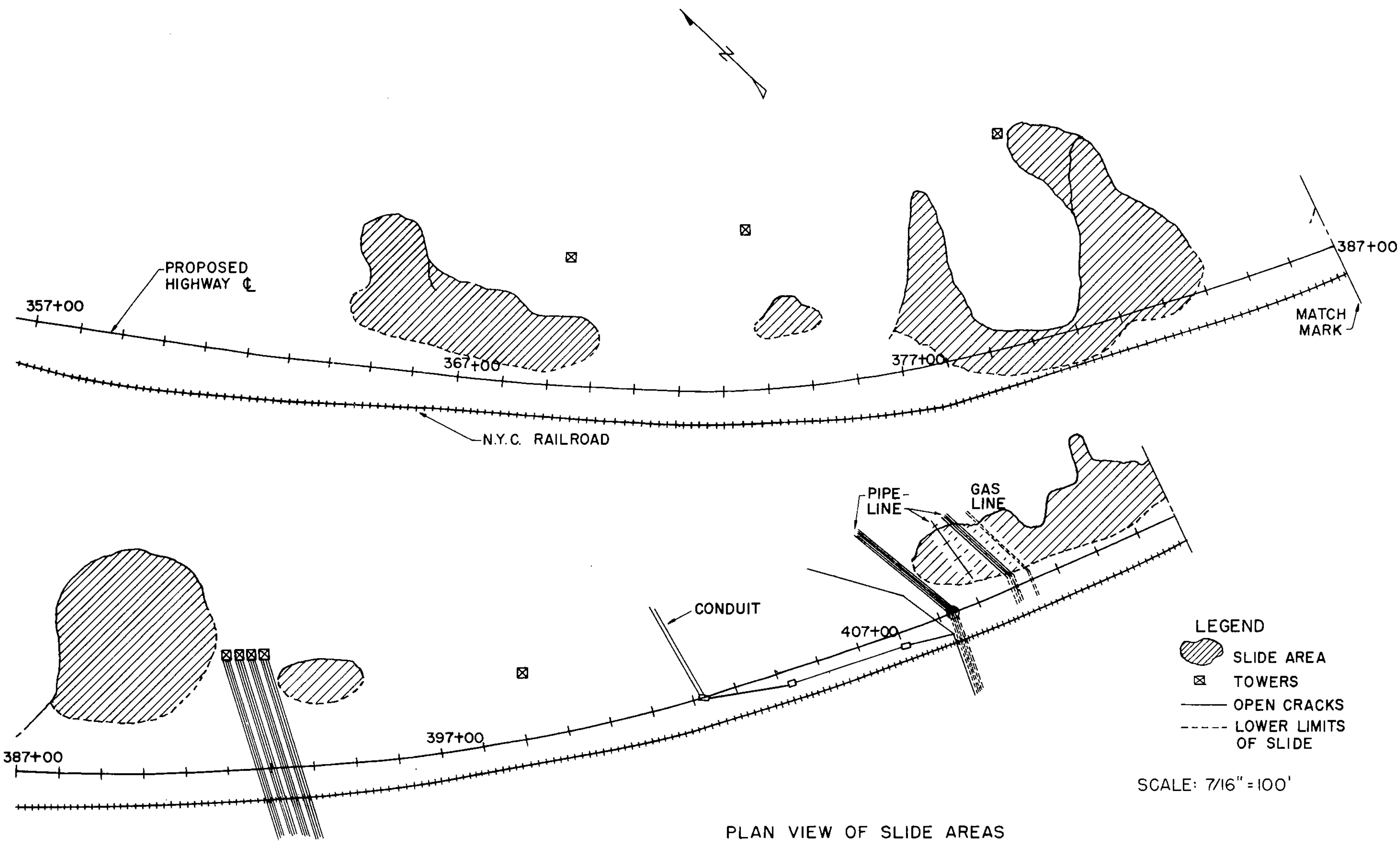
The most satisfactory approach to the selection of a treatment of a landslide will involve the application of both geologic and engineering principles. Furthermore, one can not safely ignore either of the sciences in any stage of the investigation.

A need exists for the translation of pure geologic data and terminology into quantitative results that can be related to the influence on resisting and motivating forces.

The example cited does not represent a model or ideal approach. Interrelating the principles of geology and engineering did produce a better understanding of the existing conditions, and more confidence can be placed in the recommendations.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the work done by staff members of the Department of Soil Mechanics of the West Virginia State Road Commission in the conduct of the investigation that is reported. Mr. Louis H. Trigg had overall coordinating responsibility and was in charge of the analyses. Mr. Clifford L. Goans contributed in a major way as the geologist for the study. Mr. Clayton W. Taylor was in direct charge of the field investigation, and was largely responsible for coordinating the dozer and drilling operations. Mr. Charles Karr was in charge of the auger drill crew, and Mr. Jack Gibson supervised the core drilling. The study was also improved by the fine work done by the crew headed by Mr. R. W. Moore of the U. S. Bureau of Public Roads.



PLAN VIEW OF SLIDE AREAS

FIG. 1

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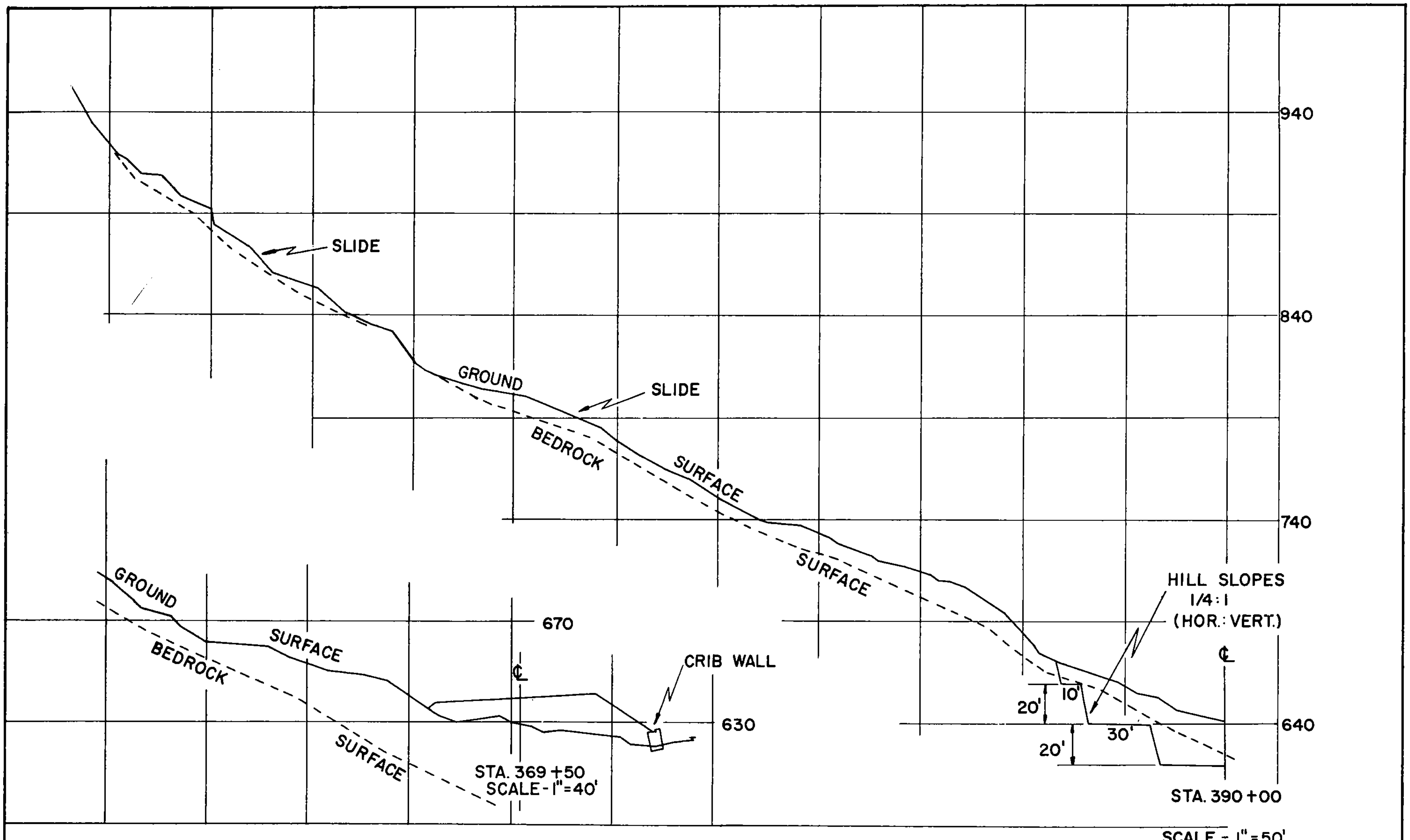
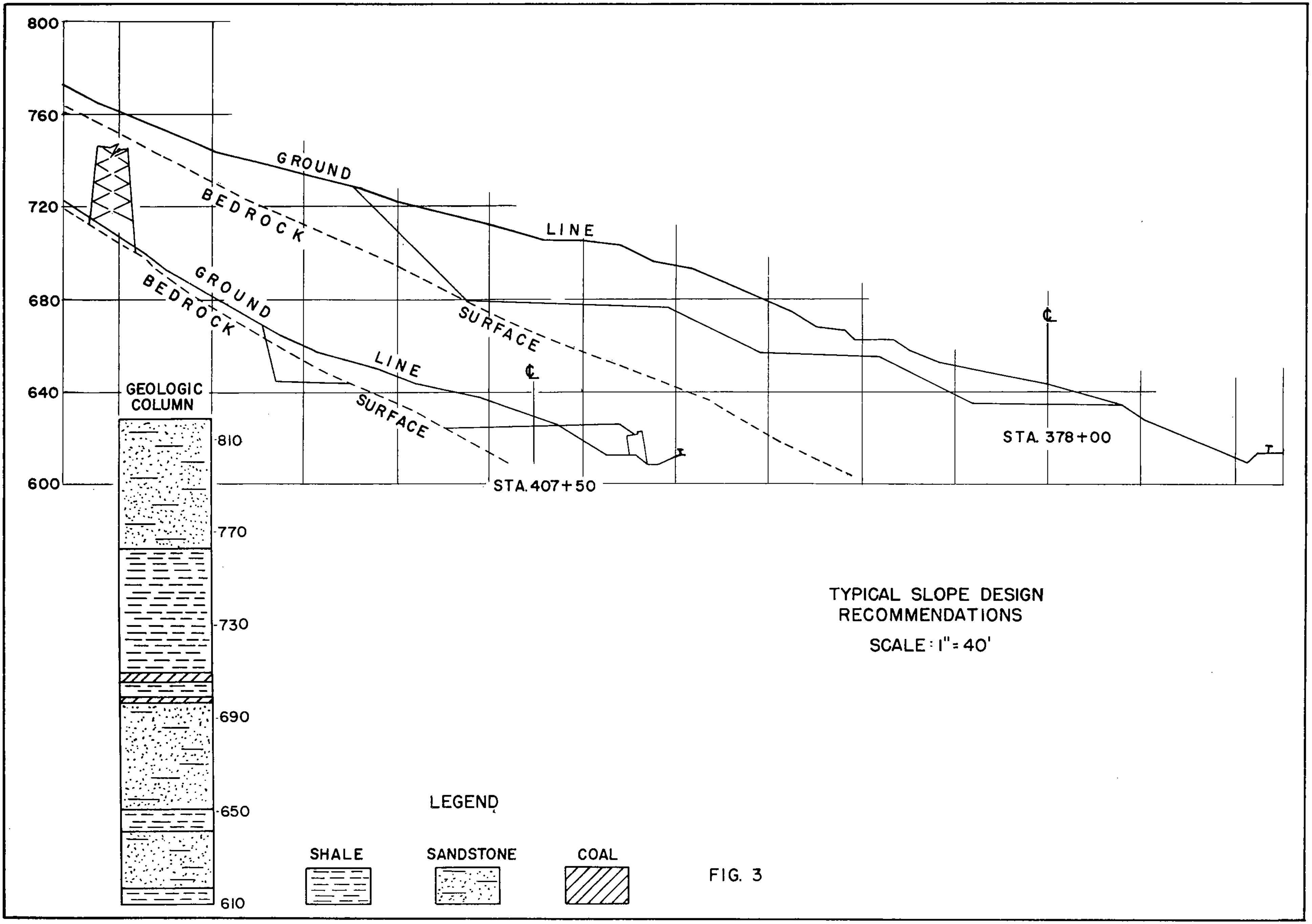


TYPICAL DESIGN RECOMMENDATIONS

FIG. 2



TYPICAL SLOPE DESIGN
RECOMMENDATIONS
SCALE: 1" = 40'

FIG. 3

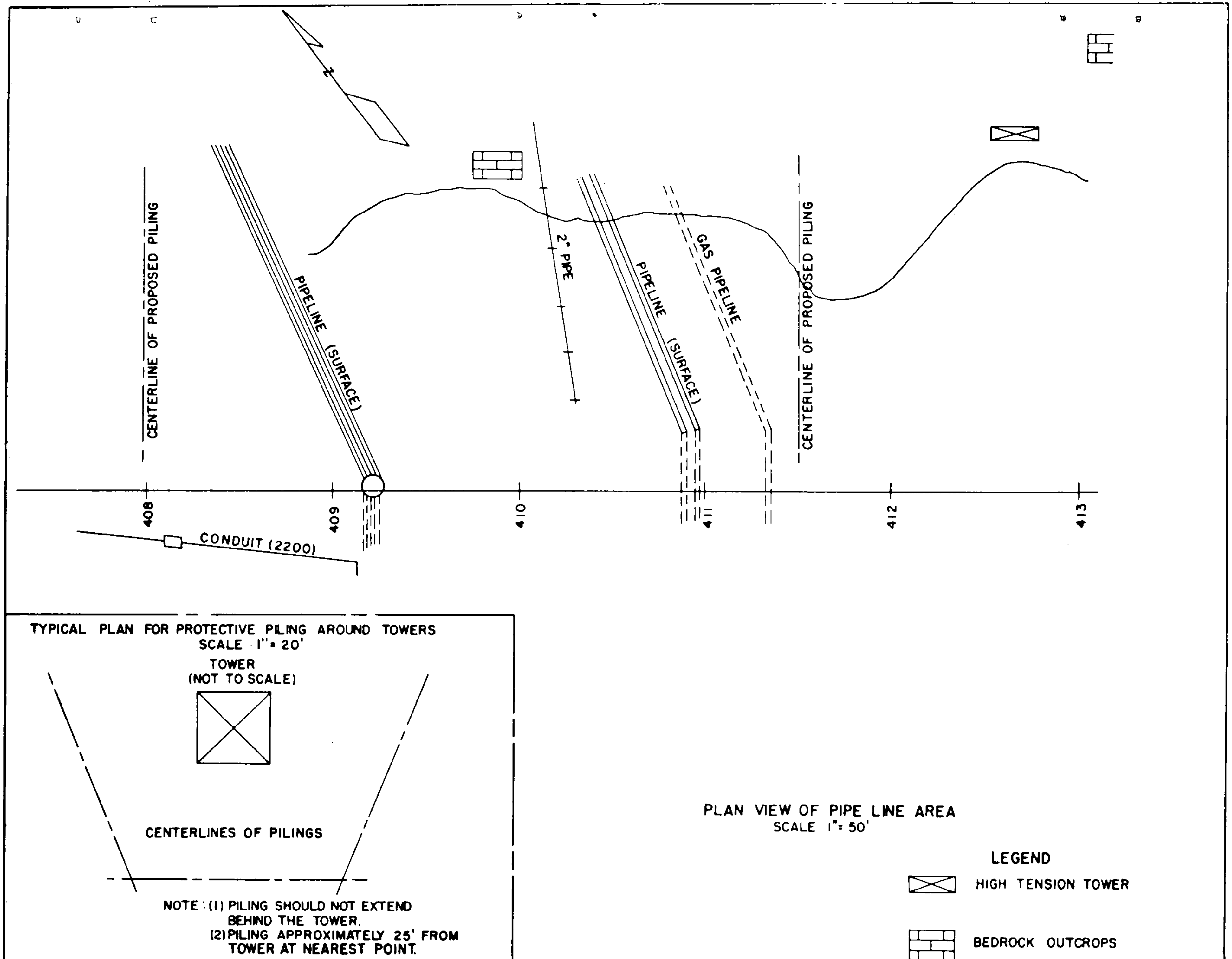
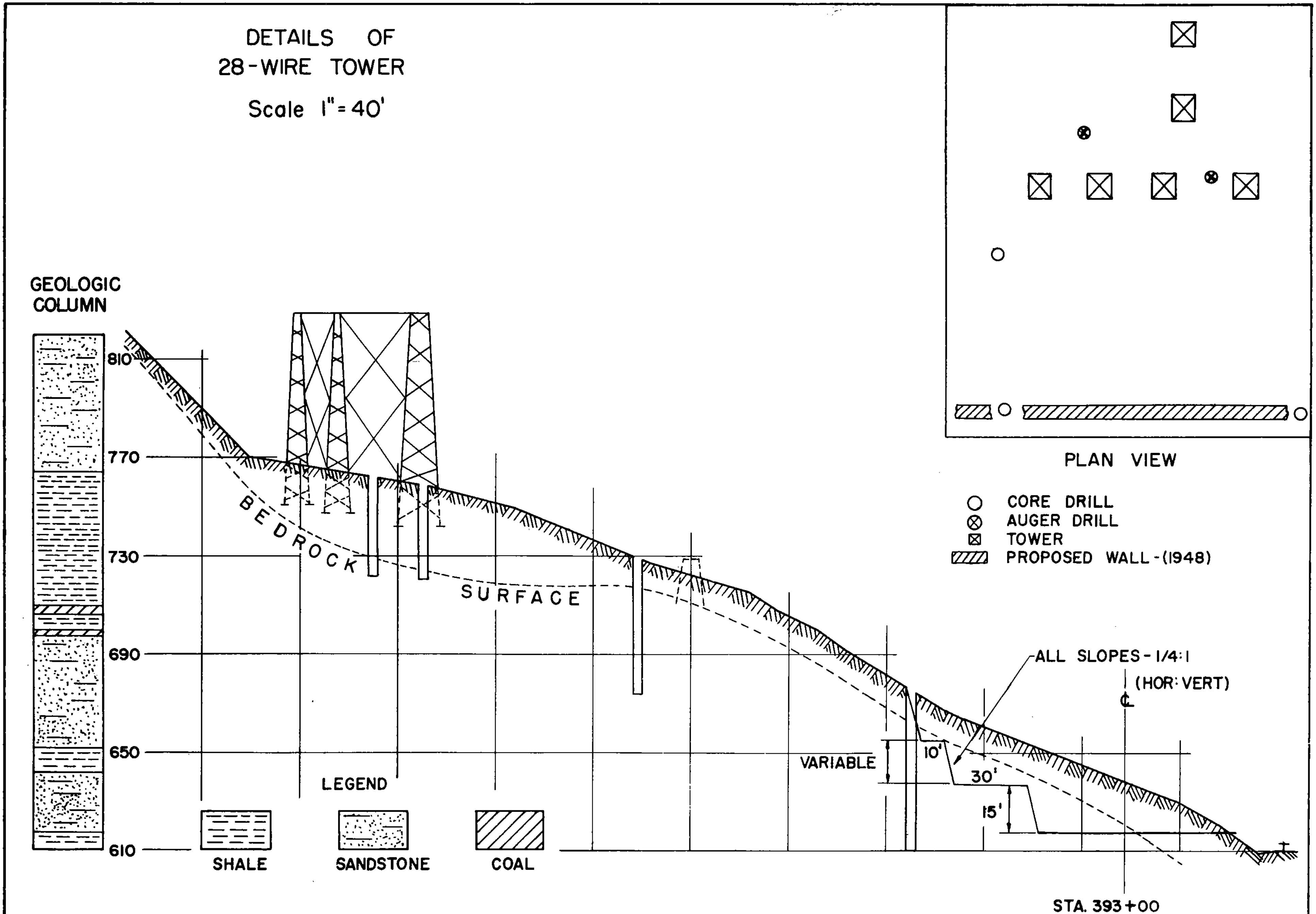

CENTERLINE OF PROPOSED PILING
PIPELINE (SURFACE)
2" PIPE
PIPELINE (SURFACE)
GAS PIPELINE
CENTERLINE OF PROPOSED PILING
408
409
410
411
412
413
CONDUIT (2200)
TYPICAL PLAN FOR PROTECTIVE PILING AROUND TOWERS
SCALE 1" = 20'
TOWER
(NOT TO SCALE)
CENTERLINES OF PILINGS
NOTE: (1) PILING SHOULD NOT EXTEND BEHIND THE TOWER.
(2) PILING APPROXIMATELY 25' FROM TOWER AT NEAREST POINT.
PLAN VIEW OF PIPE LINE AREA
SCALE 1" = 50'
LEGEND
HIGH TENSION TOWER
BEDROCK OUTCROPS

FIG. 4



DETAILS OF
28-WIRE TOWER
Scale 1" = 40'
GEOLOGIC COLUMN
810
770
730
690
650
610
BEDROCK SURFACE
LEGEND
SHALE
SANDSTONE
COAL
VARIABLE
10'
30'
15'
ALL SLOPES - 1/4:1
(HOR: VERT)
STA. 393+00
PLAN VIEW
CORE DRILL
AUGER DRILL
TOWER
PROPOSED WALL - (1948)

FIG. 5

APPLICATION OF PETROGRAPHIC PROCEDURES TO HIGHWAY ENGINEERING

by

Bryant Mather*

The fact that this is the seventh symposium on geology as applied to highway engineering should signify that there is no longer a need to advocate as new the use of geological knowledge and techniques in highway engineering. As Dr. Singewald[5]** remarked last year: "Geology has become a more and more indispensable adjunct to highway engineering. The highway engineer had been inclined to disregard geology and the geologist because the engineer's was a quantitative art, whereas geology was only a qualitative science in which the conclusions were only qualitative, and hence not of the exactness demanded by the engineer. Failures in highway construction little by little aroused the highway engineer to the fact that the results of his exact engineering calculations had no more validity than the assumptions on which they were based, and that those assumptions were inextricably involved in the geology of the terrain. At the same time, an outstanding advance in geology has been to become more and more quantitative. Thus as the needs of the highway engineer for geology were increasing, the geologist was making geology more and more useful to him."

The segment of the field of geology about which I am to speak - petrography - has for over a century been one of the most quantitative branches of geology and hence rather more readily adaptable to the service of engineering. The problems of making petrographic knowledge and procedures available to the service of engineering lie primarily in the field of communication. Katharine Mather[4] in a paper that has become available only within the last two weeks, has written: A petrographic examination...ordinarily begins and ends with a problem of communication between the person who requests the examination (usually an engineer) and the person who makes it (usually a petrographer). Unless the two succeed in producing a clear, mutually understood statement of the problem, they cannot expect a clear, useful answer economically obtained. The engineer who asks for an examination of a particular (sample) suspects that the (sample) is unusual; the more clearly he defines the peculiarity, the more he directs the petrographer toward the important aspects. The engineer is not familiar with petrographic techniques and approach; the petrographer does not realize the engineer's responsibility for decision and action, does not find out all the engineer could tell him about the sample, and may not realize which petrographic findings are useful and relevant. The

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** Raised numbers refer to similarly numbered items in the list of references.

petrographer should not expect petrographic results to be taken on faith unless the rationality of the techniques producing them are demonstrated. Both should remember that the essentials of petrographic examination...are practiced any time anyone looks intelligently at (rocks, concrete, or similar materials) either in a (large mass) or as a specimen and tries to relate what he can see to the past or future performance of the (material). On this basis it is clear that many of the most useful petrographic examinations are made by inspectors, engineers, chemists, physicists - anyone concerned with the production and use of (the material)."

The introduction to ASTM STP 169 begins with the statement: "The usefulness of a material in serving a given purpose is dependent upon its properties, or conversely the properties of a material determine the purposes it can usefully serve." The authors of the introduction comment that this is "axiomatic and requires only passing attention by those dealing with materials." This may well be true, but a careful consideration of some of its implications should clarify our understanding of the proper application of petrographic procedures to highway engineering.

No rock or mineral ever performed well or poorly in highway use simply because it belonged to a type or a species that had a particular scientific name such as "granite" or "kaolin." The good or poor performance results from the collection of properties possessed by the rock or minerals. Only a little reflection is required to remind us that in most cases the diagnostic properties that are employed to determine if a given rock shall be called by one name rather than another will not suffice as a basis of categories of quality for engineering use. This fact, long recognized by some petrographers in certain fields, has often been overlooked by engineers. A typical communications problem arises when the highway engineer hears a petrographer speaking favorably of an aggregate containing high percentages of shale and unfavorably of another containing high percentages of granite. The engineer feels that, if he understands anything about types of rock, it is that granite is better than shale! Of course, stated that way, the statement is not only correct but also valuable - as a statement of an average statistical situation. Since, however, the categories "shale" and "granite" are based on criteria that do not include properties of engineering importance, it follows that individual examples of shale may possess quite satisfactory properties for many engineering purposes for which individual examples of granite would be quite unsatisfactory. This principle has been summed up[2] as follows: "The description and classification of materials present in samples... will logically include identification of the rocks in the sample. Identification is usually a necessary step towards recognition of the properties which may be expected to influence the behavior of materials in their intended use. It is not an end in itself. Rock of any type may perform well or poorly...depending...." A similar comment has been included in the ASTM Standard Recommended Practice for Petrographic Examination of Aggregates for Concrete (Designation: C 295).

To apply petrographic procedures to highway engineering you must first catch a petrographer. I have suggested[1] that the place to look for petrographers to work on the materials with which highway engineering is concerned is among the products of those "colleges and universities with good geology departments, located in those parts of the United States underlain by crystalline rocks." This approach is, I believe, the one that has most often been used. Other approaches used less often and less successfully have involved attempts to convert an "engineering geologist-foundation geologist" to a full or part-time petrographer; to add petrography to the duties of a chemist or a materials engineer either with or without a short course.

Having caught a petrographer - the engineering organization that has him must then teach him to understand them and learn to understand him. Then we get to work. One phase of petrography as to technique is the use of the light microscope; one aspect of highway engineering to which petrography may be applied is portland-cement concrete. The zone of interaction of these two facets has been thoroughly reviewed by Katharine Mather in her prize-winning paper, "Applications of Light Microscopy in Concrete Research."[3] In this field we may differentiate between examinations of cements, aggregates, and concrete. As early as 1882, De Chatelier began petrographic studies of portland cement clinker and thus made the first contribution to our knowledge of the constitution of cement. In 1927 the X-ray diffraction techniques were first applied to cement. In 1934 the metallographic etching techniques were first employed by Tavasci. The first record of a petrographic description and classification of concrete aggregate material of which we know is that in 1915 by C. W. Tomlinson - although as far back as 1905 Lovegrove reported studies of the relation of Deval abrasion test results and properties of rocks. With Stanton's discovery in 1940 of concrete expansion produced by a chemical reaction between the alkalies - sodium and potassium hydroxide - and certain aggregate constituents, there was a large expansion of petrographic work in concrete since this seemed to be one case where minerals or rocks could be sorted petrographically - by name - and classified directly as good or bad. It is probably true that one can come closer to doing so here than in most other engineering problems, but those who attempt to do so indiscriminately find that there are unforeseen problems. While most of us have believed that deleterious alkali-aggregate reactions can only occur when rocks or minerals composed of or containing certain types of silica are present in the aggregate, we now know that the mere presence of such materals provides no guarantee that trouble will follow nor that their apparent absence will insure freedom from trouble.

As has been mentioned, petrographic procedures are employed whenever anyone looks intelligently at construction materials with the purpose of deriving information to interpret past history and to predict future service. In the narrow meaning of the term, petrographic procedures, especially as applied to concrete and concrete materials, refer to the use in the laboratory of equipment originally developed for the examination of rocks and minerals such as the polarizing microscope, microscope lamp, immersion media, and X-ray diffraction apparatus. The latter may use either a camera for film techniques or a Geiger counter pick-up and strip chart recorder for direct measuring techniques. In petrographic work in a concrete laboratory X-ray diffraction is used for identifying not only minerals but also such other substances as organic compounds in admixtures for concrete.

As was mentioned, petrography originally, and to a large extent still, involves the stury of rocks. One kind of rock of particular interest in concrete construction is chert. It may possess undesirable properties, either physical or chemical or both, or it may be free of such properties and be an entirely acceptable material as concrete aggregate. Petrographic procedures are well illustrated by a consideration of how they may be used to study chert. One typical mode of occurrence of chert is as beds or nodules in limestone quarry rock. Chert may have a layered structure as in "agate". Being a very resistant material, it survives erosion and transportation and forms gravel particles in streams draining areas underlain by limestone. These chert gravel particles may be either dense or porous. Many rough surfaced particles are porous, while many smooth particles are dense. Some chert is formed as a replacement of oolitic limestone, in which case the chert may faithfully preserve the oolitic structure of the rock. Examples of such chert are found in gravels near Memphis, Tennessee. It sometimes is desirable, or even necessary, to observe the manner of distribution of chert with regard to particle size in natural aggregates. Such data will prevent wrong interpretations based on comparisons of samples of different gradings.

A study was made of a concrete structure in which approximately 42 per cent of the gravel coarse aggregate was chert. Cores were taken, sawed in half, and the surfaces examined. The concrete contained cracks, many of which passed through chert pebbles. At various points in the concrete there were pockets of gel, the product of alkali-aggregate reaction. Sixty-eight chert particles were picked out as being associated with gel pockets. Forty-eight of these particles when examined in immersion media were found to contain chalcedony and for forty of these particles the index of refraction of chalcedony was determined. The criterion for distinguishing chalcedony from quartz is an index of refraction below 1.544. The average index of these specimens was 1.536, the minimum 1.524. On this basis, it was calculated that the opal content of the chalcedony ranged between about 6 and 18 per cent in the chalcedony and between 1 and 5 per cent in the aggregate.

Petrographic examination is, of course, made on rocks of all types for physical properties as well as to detect the presence of constituents of interest chemically. A study was made of granite from two quarries in North Carolina. One showed no microfractures, the other sample was from a quarry located only a few miles away where the rock is shattered with microfractures. This difference was found to be related to very great differences in performance.

Petrographic procedures have also been applied with great benefit to the other materials used in concrete such as portland cement clinker, cement paste in concrete to observe unhydrated cement particles, and materials that may be considered as replacements for cement, such as fly ash. Even the bubbles of air produced in a sand-water mixture by an air-entraining agent may be studied with a microscope. Finally, concrete itself may be studied, both when it is freshly hardened and when it is old. Examination under a microscope will show the telltale marks formed by ice crystals in concrete that had frozen before initial set, if they are present. Thin sections of concrete are examined to observe the structure and texture of the aggregate and the matrix. Cores drilled from concrete pavements may show voids beneath gravel particles due to segregation of bleed water. Concrete subjected to weathering - wetting and drying - frequently can be made to reveal much of its history, if the substances deposited in voids are identified. Such voids may be lined with crystals of calcium sulfoaluminate, crystals of the mineral aragonite, and other substances.

One often hears of engineers who claim to be unable to understand either the results of the procedures of petrographic examination. Actually, I think that there are certain advantages to presuming an understanding even if in fact it does not exist. On the other hand, neither the engineer nor the petrographer should expect that the application of petrographic techniques will solve all problems.

References

1. Mather, Bryant, "General discussion." Symposium on Light Microscopy, ASTM STP No. 143, p 126 (1953).

2. Mather, Katharine, and Mather, Bryant, "Method of petrographic examination of aggregates for concrete." ASTM Proceedings, vol. No. 50, p 1288 (1950).

3. Mather, Katharine, "Application of light microscopy in concrete research." Symposium on Light Microscopy, ASTM STP No. 143, pp 51-70 (1952).

4. ________, "Petrographic examination - hardened concrete." Significance of Tests and Properties of Concrete and Concrete Aggregates, ASTM STP No. 169, pp 68-80 (1956).

5. Singewald, Joseph T., Jr., "The role of the state geological surveys in geology as applied to highway engineering." Proceedings, Sixth Annual Symposium on Geology as Applied to Highway Engineer, Johns Hopkins University, pp 1-6 (1955).

INVESTIGATION OF BRIDGE FOUNDATIONS

By

Martin Deuterman

The bridge engineer's primary use of geology is to obtain sub-terranean data with which to design foundations for bridges, approach fills, and incidental highway structures. The data are used to determine the most desirable structure location and to determine the most suitable types of substructure. The foundation must be adequate to support the structure without excessive settlement. Included in the data must be the types of underlying material as well as the characteristics of the material from which may be predicted the probability of local settlement under loads, slides, scour, filling, and faults. In earthquake areas, data on the frequency and intensity of disturbances are also essential. In some regions, volcanic or glacial action are important factors.

The evaluation of the data for foundation design is the responsibility of the bridge engineer. In most cases, his education and experience are adequate to permit performance of this evaluation without assistance. On large structures and for questionable subsurface conditions, the recommendations of the geologist and the soil mechanics specialist are required.

The available information pertaining to the surface and subsurface material should be collected and studied in the planning stage. A paper presented at the Sixth Annual Synposium on Geology as applied to Highway Engineering on Sources of Information on Ground Conditions, by Alice S. Allen, discusses the many available sources of such information. A geologist may be consulted to assist in interpreting the collected material. A study of the collected data is then made to select a tentative type of structure suitable to the estimated conditions. The type of structure may have to be changed if a later determination of the foundation so indicates. A structure layout is then made to determine the location, type, and depth of foundation explorations.

After the necessary foundation explorations are made and plotted, analysis of the data is made to determine the type of structure and the method of foundation support. The assistance of a geologist will often be necessary to correlate the data of adjacent borings and to estimate the preloading of various strata. The soils mechanics specialist will also be required to estimate the safe load and settlement where questionable material is encountered.

The use of geology in bridge engineering will be illustrated by the description of preliminary work recently performed for the Jones Point Bridge over the Potomac River between Virginia and Maryland at Alexandria, Virginia. The structure is to be part of the outer circumferential route around Washington, D. C., and a link in the by-pass for north and south traffic on U. S.

Route 1 around Washington. The authorization by Congress provides $14,925,000 for a bridge with a six-lane roadway, a swing span with 150-foot horizontal clearance and, in the closed position a 70-foot vertical clearance. Construction plans will not be started pending a decision by Congress on a request to change the law to permit the use of a double leaf bascule span and a 40-foot vertical clearance.

The location of the beginning and end of the structure is fixed within narrow limits. On the Virginia end, the approach must be along South Street, with a new apartment development on the south and an old cemetary on the north. For economic reasons, the Maryland end is located on Rosalie Island, a small island near the Maryland shore.

The 7,500 feet of construction on the Washington circumferential highway will consist of 5,900 feet of bridge, 500 feet of fill over Rosalie Island, and 1,100 feet of fill over the channel between Rosalie Island and the Maryland shore. The bridge will cross 1,900 feet of river flats from the abutment to the Virginia river bank, 1,000 feet of channel, 2,000 feet of the shallow center portion of the river, and 1,000 feet of deeper water next to Rosalie Island. The bridge roadway section on the circumferential highway will consist of two 38-foot roadways, a 4-foot median and two 3-foot sidewalks. The access ramps to Alexandria will require about 1,300 feet of bridge, and 500 feet of fill. Bridge roadway on the ramps will be 20 feet with a 3-foot sidewalk and a 2-foot safety curb.

The bridge is located in the tidewater section of the river. The straight channel extends from north to south.

Physical data from the following sources were assembled prior to the preliminary study:

Configuration of the Bed Rock Surface of the District of Columbia and vicinity, Department of Interior, Geological Survey Professional Paper 217.
Chart of the Potomac River, Mattawomas Creek to Georgetown, 560, U. S. Department of Commerce, Coast and Geodetic Survey.
Photeostat of Section of Hydrographic Survey H-2688, date of Survey 1904, U. S. Department of Commerce, Coast and Geodetic Survey.
Airport Obstructional Plan and Profile, Washington National Airport, OP443, U. S. Department of Commerce, Coast and Geodetic Survey.
Topographic Map, Maryland-Virginia, Potomac River, Alexandria and Vicinity, T-5756, U. S. Department of Commerce, Coast and Geodetic Survey.
Topographic Map, Washington and Vicinity, U. S. Department of Interior, Geological Survey.
Borings, Potomac River, Alexandria, Virginia, to

Marbury Point, D. C., File No. B61-85, October 14, 1941, U. S. Engineer's Office, Washington, D. C.
The line of these borings crosses the river about 1 1/2 miles above the proposed bridge site.
Foundations of Hunting Tower Apartments, data obtained from consultation with the Building Department of Alexandria, Virginia.

The physical data were assembled and analyzed in order to estimate the underground conditions and materials that could be expected at the bridge site. A determination of the suitable types of substructure supports were determined. The method of carrying out the analysis is outlined below.

A preliminary profile was plotted using Geological Survey topographic maps for the land sections and Coast and Geodetic Survey charts for the hydraulic sections.

Washington National Airport is about 3 1/2 miles upstream on the west bank. The Airport Obstruction Plan and Profile indicated that the bridge would be in the line of the main flight strip but well below the plane of the minimum glide slope.

The 1904 hydrographic survey showed the Maryland mainland bank extending upstream and downstream along the line of the river bank of Rosalie Island. Investigation developed that a sand and gravel company had purchased the land and in dredging for sand and gravel had formed the bay in the mouth of which Rosalie Island is situated.

Inquiry was made to the Building Department of the City of Alexandria about the foundations of the new apartment development near the Virginia abutment. The information furnished by the Alexandria Building Department indicated that displacement piles not longer than 40 feet would support the desired load for the abutment and approach retaining walls and that the soil would support the approach fill.

The 1,900 feet between the abutment and a concrete sea wall at the Virginia bank is firm material across a World War 1 shipyard at an elevation of about plus 6 feet. The 1904 survey indicated that the major portion of this area was a shallow bay of the river only about one foot deep at low tide. It was assumed that this area was filled in when the shipyard was built. The Marbury Point borings, as well as experience with other structures along the Potomac shores, indicated that sand and gravel strata would be located near the surface. It was estimated that displacement piles about 40 feet long would be satisfactory, to support piers for short spans in this location.

The center line of the ship channel and 1,000-foot natural channel is on the Virginia side of the river about 500 feet from the bank. The average depth is about twenty-five feet. The 2,200-foot center portion of the river is about five feet deep. The 1,000 feet adjacent to Rosalie Island is about 15 feet deep. Reference to Geological Survey Professional Paper 217 showed that bedrock is elevation -500 on the Virginia end and -625 on the Maryland end. The bedrock is principally granite, granite gneiss, schist, and diorite. Sand and gravel has been dredged from various portions of the river bed to depths up to sixty feet by commercial companies. No reliable estimate could be made of the character of the material down to -60. The Marbury Point soundings indicated for the channel section river mud to about elevation -80, fine sand with some clay mixtures, varying from hard to soft, between -80 and -120, below -120 the material was hard clay with some lenses of sand and clay. This hard clay was identified from Geological Survey Paper 217 as part of a sedimentary formation of the lower cretaceous age belonging to the Potomac group. These sediments extend down to bedrock. It was decided that caissons or large size steel pipe piles could probably be stopped at elevation -135 for the substructures of the movable spans and the long river spans. From the channel to the Maryland bank the soundings indicated the bottom of the river mud sloped up from -80 to -20, the top of the hard clay sloped from -120 to -30 and between the mud and hard clay was sand with some intervening strata of unconsolidated mixtures of sand and clay, and sand and a little gravel. It was decided that concrete displacement piles, with some large size pipe piles near the channel, could probably be used for the foundations of the shorter spans over the shallow water from the channel to Rosalie Island.

The trees and old stone sea wall on Rosalie Island made it apparent that the island was an undisturbed portion of the old mainland. The Corps of Engineers borings indicated that sand and gravel could be expected down to -30, with hard clay from there on down to at least -110. This would be suitable for the Maryland abutment if supported on displacement piles, and on which to place a fill across the island.

The 1,100-foot dredge section between Rosalie Island and the Maryland mainland had probably been dredged for sand and gravel. The distance below the present bed to which dredging had been done was not known except that it was not in excess of 60 feet. Comparative estimates indicated that a hydraulic fill would probably be more economical than a bridge.

The available data and experience with the Potomac shore led to the assumption that the area of the Maryland mainland adjacent to the bank would be satisfactory to support the approach fill.

Let us summarize the foundation conditions as assumed from the available data. The Virginia abutments and a few adjacent spans would be founded on sand and gravel satisfactory for displacement piles. The spans in the flats from the first few spans to the river bank would be founded on unconsolidated sand and gravel, covered in places with fill material, satisfactory for displacement piles. The channel section would be mud over unconsolidated sand on hard clay with the clay being suitable to support caissons or large size steel pipe piles at -135. The shallow river section would be the same as the channel section with the hard clay closer to the surface and the unconsolidated sand strata being mixed with varying amounts of gravel. Displacement piles could be used for about half of this section with large size pipe piles probably needed adjacent to the channel. Rosalie Island would be undisturbed material with sand and gravel near the surface suitable for displacement piles for the abutment and for support of the fill. The material under the dredged channel between Rosalie Island and the Maryland mainland was questionable due to the fact that it was not known how deep sand and gravel had been removed. It was assumed a fill could be supported by this material. The Maryland mainland would be sand and gravel under a thin layer of swamp mud, suitable for supporting the approach fill.

The most economical span lengths were computed from the assumed data and a preliminary design, layout, and estimate made. A deck structure was selected with plate girders for long spans and I-beams for short spans. The span lengths were selected to decrease gradually toward each end from the 200-foot span of the bascule until the economical short span lengths were obtained. The minimum span length was set at 60 feet for esthetic reasons.

Location and depth of proposed borings were determined from the pier locations and from assumed foundation material. Undisturbed samples were called for in order that deformations and bearings values of plastic material could be estimated. A layout of the boring holes is included in the appendix.

Since the borings were to be done under contract, invitations for bids, specifications, and special provisions were prepared. Every effort was made to fully describe the scope of the work, including the information to be secured and the purpose of the borings. The extent of the work was made flexible by allowing for increase or decrease of 30 percent in depth and number of holes or number and location of undisturbed samples.

The equipment, and drilling and sampling devices to be furnished were specified in such a way as to allow a choice but to insure satisfactory results. Split-spoon samplers 2 1/2 inches and 3 inches outside diameter, 24 inches long, were specified. The procedure required was such as to insure accurate data for both undisturbed and disturbed samples. Sampling was required at a 5-foot intervals of depth and at all changes in strata. The number of blows of a 300-pound hammer falling 18 inches required to drive the sampler 12 inches was to be recorded. In addition, undisturbed samples with a 3-inch outside and 2 7/8-inch inside diameter, 24 inches long Shelby tube sampler or equal were to be obtained at specified elevations. The sampler was to be pushed into the soil by a continuous or rapid motion without impact or twisting.

A copy of the specifications used on this project is included in the appendix.

Bids were let, the contract awarded to the low bidder, and supervision and inspection furnished by Division 15 of the Bureau of Public Roads. Frequent field inspections were made by members of the Bridge Branch of the Bureau to get first hand information as to the conditions encountered.

The contractor moved in on the job in July with supplies and equipment to operate three drilling rigs at one time.

Two framed barges supported on oil drums were build for use on the water holes. A 36-drum barge was constructed for the shallow water holes and the holes between Rosalie Island and the Maryland shore, while a 56-drum barge was constructed for use in the channel. A hole about 2 feet square, through which to drill, was left in the center of each barge. A mast was mounted from which to display lights and signals in accordance with Coast Guard regulations. The extreme heat down in the river flats slowed up barge construction operations somewhat.

Each barge was equipped with a drilling rig, piston water pump, a large well-supplied tool box, and an adequate supply of 6-inch and 4-inch casing pipe, drill pipe, samplers, and core drills. When the crew was on board, the 36-drum barge was loaded to about capacity as to both space and weight.

The location of land holes was staked out by a survey crew. The barges were loaded over the water holes by range poles for line, and by transit sight from a point on the base line for distance. The transit point was 835 feet from the center of the bridge.

Land rigs were moved with steel cables from their drums fastened to various types of improvised anchors. The barges were moved with an outboard motor equipped metal row boat. An anchor fastened to each corner permitted accurate spotting and held the barges securely in place.

The manner of drilling and driving casing depended upon the character of the material being penetrated. Water was pumped under pressure into the drill pipe during drilling and, in most cases, into the casing pipe during driving.

Split spoon samples were taken at 5-foot intervals and at change of character of material. Undisturbed samples were taken with Shelby spoons at locations designated on the plans if these locations were in plastic material. It was found that the Shelby spoons could not be pressed into some of the hard clay, even with the drilling rig held down to the barge with chains. Several methods were tried to remedy this situation. The final solution was to allow the Shelby tubes to be driven when they could not be pressed.

Some difficulty was encountered with fine sand running out past the trap. At some holes, the fine sand would run back between the casing and the split sampler as fast as cleaned out. This sand would transfer the force of the hammer blows to the casing, forcing the casing down and resulting in inaccurate data. No record was made of the number of blows in these cases.

Large gravel and small cobbles made driving and sampling difficult. In many cases, it was necessary to drill in the material with a diamond drill before the casing could be driven.

A 6-inch casing pipe was driven to solid material outside the 4-inch casing on the deep water holes. This supported the 4-inch casing through the water and river mud and facilitated driving and removing the 4-inch casing. The hard clay was stable enough to stand without casing so that the 4-inch casing needed to be driven only about 15 feet into the clay. One difficulty in the hard clay was the breaking up of the hard cores that fell out of the drill. If these were not broken up in cleaning out the holes, faulty samples and number of hammer blows would have resulted.

The difficulties are recorded above; however, it is noted that for most of the drilling, the operations proceeded smoothly in the usual manner. The barges were removed from the river twice while hurricanes passed up the coast.

While making the boring, it was found that the abutments of the on and off ramps to Alexandria were in a swamp of river mud. As this material appeared unsuitable to support a high fill, additional borings were ordered along the lines of the ramps in order to provide data for design of the foundations for the fills or extensions of the bridge ramps.

The samples were stored and the testing performed at the laboratory of the Physical Research Branch of the Bureau of Public Roads. Members of the Physical Research Branch were consulted in the planning of the borings and sample tests and in the solution of field sampling problems.

The following tests were made on the undisturbed samples: mechanical analysis, liquid limit, plasticity index, soil classification, direct shear, initial wet density, initial dry density, initial moisture, triaxial compression, and consolidation. Consolidation tests were omitted for some of the samples at locations where settlement of the structure would not be affected.

Logs of the test borings were furnished by the contractor and were plotted by Division 15. Prints of these plotted logs are included in the appendix.

The test borings indicate that the subsurface materials conform in general with conditions assumed in the preliminary planning. The results of the tests have not been analyzed but an examination of the test data strongly indicates the foundation methods discussed below.

The lowest sampled material, which underlies the entire area, is hard, plastic, brown and gray clay. This clay will be used to support large size steel pipe piling under the channel piers and a few of the short spans in the river in the Maryland side of the channel. Steel pipe piles will be driven about 15 feet into the clay to support a design load of 80 or 100 tons. The sedimentary material over the clay in this portion of the structure is not considered satisfactory to support displacement piles.

The material over the hard clay is mostly unconsolidated fine sand, both brown and gray, laid down in Recent time. This sand is not in continuous strata and is usually mixed in various combinations with other sedimentary materials. These materials range from clay to large stones. The borings indicated good bearing values in this material except near the channel. Displacement piles in this sandy material will be used to support all of the structure except the section at the channel and, possibly, a few piers on the Virginia flats under which piles may be omitted.

Dark gray, silty clay, best described as river mud, covers the sandy material in all but a few locations. The river mud may be at the surface or covered with water or fill. The sampler usually sank under the weight of the hammer in the mud, indicating very little supporting value. Special attention was given to the sampling at the elevations of the bottoms of the seals to determine if the mud would support the concrete seals. The material has such low bearing values that it is proposed to excavate two or three feet below the elevations of the bottoms of the seals and backfill with gravel. It may then be necessary to pour subseals before the main seals can be supported.

Analysis of the test results will be made by the designing engineers in cooperation with the Physical Research Branch and the results will be checked by the Physical Research Branch.

During construction, load and settlement tests will be made on single piles in each channel pier and in enough of the other piers to evaluate safe pile loads from driving resistance.

The geological investigation and use of the results in bridge design may be summarized as follows: Collect all data available on geological conditions; analyze this data with such geological and soil mechanics assistance available and necessary; make a preliminary design; locate test holes and determine type and extent of explorations necessary; perform foundation exploration, making such changes in the field as are deemed necessary as the work progresses; perform necessary tests on soil samples; analyze test results with such geological and soil mechanics assistance as is necessary; check preliminary design for revisions in type and span required; complete final design.

Standard Form 36 Prescribed by General Service 80-M, No. 1949 Edition	CONTINUATION SHEET (Supply Contract)	Contract, Order, or Invitation No. (As applicable) Invitation No. D-15	Page No. 3

Item No.	Supplies or Services	Quantity (No. of Units)	Unit	Unit Price	Amount
1.	Foundation borings on land (including dry samples) from natural ground surface to a depth not exceeding 100 ft. below ground surface.	1,040	Lin. Foot	$ 3.45	$ 3,588.00
2.	Foundation borings on land(including dry samples) from a depth of 100 ft. below natural ground surface to a depth not exceeding 200 ft. below ground surface.	20	Lin. Foot	$ 5.00	$ 100.00
3.	Foundation borings through water (including dry samples) from surface of water to a depth not exceeding 100 ft. below surface of water.	1,940	Lin. Foot	$ 4.65	$ 9,021.00
4.	Foundation borings through water (including dry samples) from a depth of 100 ft. below surface of water to a depth not exceeding 200 ft. below surface of water.	790	Lin. Foot	$ 5.95	$ 4,700.50
5.	Securing undisturbed samples at a depth not exceeding 100 ft. below surface of ground or surface of water.	128	Each	$12.50	$ 1,600.00
6.	Securing undisturbed samples at a depth greater than 100 ft. below surface of ground or surface of water.	79	Each	$15.00	$ 1,185.00
	Items 1 through 6 will be awarded as a whole.............................Total				$20,194.50

Bidders desiring to inspect borings site should contact the office of Mr. E. L. Tarwater, Supervising Highway Engineer, Bureau of Public Roads, 1554 Columbia Pike, Arlington, Virginia. Telephone: Jackson 8-5600.

The work under this contract shall conform to the attached Specifications, Special Provisions, and Drawing G-8-55 (May 1955).

The word "engineer" as used in the Specifications refers to the Division Engineer of the Bureau of Public Roads.

Invitation D-15
May 24, 1955

SPECIFICATIONS

Scope of Work

It is the intent of the test borings to secure complete and accurate information of the topsoil and foundation conditions at the locations indicated on the plans or as directed by the Engineer. Reliable information shall be secured and fully recorded of ground water elevations, classification of soils penetrated, the elevation of changes in stratifications along with samples of the strata penetrated. Undisturbed samples of cohesive and clay-like materials shall be secured at designated locations. All samples shall be delivered to the Bureau of Public Roads Testing Laboratory at Gravelly Point, Arlington, Virginia.

It is the purpose of the borings to determine character, thickness, resistance to penetration, and firmness of the various materials and strata encountered through proper drilling operations and sampling of materials by the use of the type of tools and equipment specified.

It is expected that materials of many varying types may be encountered, such as topsoils, fills, refuse, debris, and other miscellaneous material. Drilling into rock or other material requiring a core drill will not be required.

Whenever any feature of the work is not fully covered in the special provisions, it shall be understood that the same will be interpreted in accordance with the best current practice in this kind of work.

The analysis of all samples taken will be made by the Bureau of Public Roads Soils Laboratory and is not a part of this contract.

Location

The locations of the borings and the undisturbed samples shall be as shown on drawing G-8-55. Location of the center line of the structure and reference stations will be furnished by the Engineer.

Permits and Licenses

The contractor shall secure all necessary permits for the occupancy of the Potomac River from the District Engineer, Washington District, Corps of Engineers, U. S. Army. He shall secure the necessary permits from such other public agencies as may have jurisdiction for the occupancy of streets and other public properties. He shall also obtain permission for work to be done on private property. All permits and licenses shall be secured at the contractor's expense. The contractor shall comply with all local ordinances covering the areas in which the work is performed.

Maintainance of Navigation

The contractor shall keep free and unobstructed the navigable channel or channels prescribed by the Army Department. He shall provide and maintain navigation lights, fog and other signals in accordance with the regulations prescribed by the District Engineer.

Work in the river shall be carried on in conformance with the terms of the permit of the U. S. Army District Engineer and the contractor shall keep the District Engineer's office fully informed with respect to location and intended changes of location of his equipment so that river navigation may be fully informed regarding location of floating equipment.

Should the contractor during the course of the work sink or lose overboard any material, plant, or machinery which may be dangerous to navigation, he shall forthwith recover or remove such obstruction. The contractor shall leave the site in condition satisfactory to the U. S. Army District Engineer.

Protection and Restoration of Property

The contractor shall take every precaution to avoid damages to utilities, including underground utilities, and private property, and shall repair any damage at his own expense and to the satisfaction of the Engineer.

Equipment and Schedule of Operations

Immediately upon award of the contract, the contractor shall submit to the Engineer for approval a schedule of operations and list of equipment he proposes to use. The contractor shall furnish equipment satisfactory to the Engineer and it shall be of sufficient capacity to accomplish the work contemplated in this contract well within the time limit specified. The specifications covering the detailed execution of the work contemplate the use of modern equipment and tools especially designed for efficiency in carrying out subsoil investigations with samples taken as described later in these specifications.

The minimum size of casing used shall be adequate for securing undisturbed samples of the size specified.

The boring operation shall start on the west (Alexandria) side. The dry borings at this point shall preferably be completed ahead of the other borings. The contractor shall notify the Engineer and other interested parties of his intention to start operations at least 24 hours in advance.

It is known that many types of noncohesive soils exist and can be samples only by the use of special trap devices. The contractor should, therefore, have a complete selection of the varieties of soil samplers which may be required to secure satisfactory specimens of any type of soil which may be encountered.

Inspection

The work is to be directed under the general supervision of the Engineer and shall be subject to inspection by his authorized representative. No drilling shall be done except in the presence of the Engineer or his inspector. The contractor shall provide access to the work for the inspector at all times during operations in the river, and shall have available a power boat for the transportation of inspectors between the drilling barge and shore. The presence of the Engineer or his authorized representatives shall not relieve the contractor of any responsibility for the proper execution of the work.

Dry Samples

Dry samples shall be taken with a split-spoon sampler, minimum inside diameter of 2 1/2 inches and 3 inches outside diameter of a minimum length of 24 inches, for the purpose of obtaining representative samples of soils (dry samples) for identification purposes, and to obtain a penetration record of the samples into undisturbed material.

The test hole shall be cleaned out to the sampling elevation by augering, washing, or other methods insuring that the material to be sampled is not disturbed by the cleanout operation. Sampling shall be done at an approximate average of 5-foot intervals of depth and at all changes in strata. The casing shall not be driven into the layer to be sampled in advance of the sampling operation.

With the split-spoon sampler resting on the bottom of the hole, the sampler shall be driven into the soil for a distance of approximately 18 inches by a series of blows from a 300-pound hammer falling freely for a drop of 18 inches. After the spoon has penetrated about 6 inches into the soil, the number of blows required to produce the next foot of penetration shall be recorded. Where the boring is below the water table at the time of sampling, the water level in the hole should be at or above the ground water or river water level.

In cohesion or nearly cohesionless sand located below the water table, a core catcher, or a scraper bucker, or other similar devices, shall be used in order to prevent the sample from falling out before it can be brought to the surface.

The sample shall be removed from the spoon and immediately placed in an air tight wide mouthed glass jar of sufficient size to hold a section of the sample intact.

The jars shall be marked to indicate job designation, boring number, sample number, date, elevation at which sample was taken, penetration record, and classification of soil, and shall be shipped to the Bureau of Public Roads laboratory for further analysis.

Undisturbed Samples

In addition to the sampling described above, undisturbed samples for laboratory testing shall be taken at locations and elevations shown on drawing G-8-55 or as may be directed by the Engineer in the field.

The test hole shall be cleaned out to sampling elevation by augering, washing, or other methods insuring that the material to be sampled is not disturbed by the cleanout operation.

With the sampling tube resting on the bottom of the hole and the water level in the hole approximately at ground water or river surface elevation, a tube sampler (Shelby tube sampler or equal) of a minimum outside diameter of 3 inches, inside diameter of 2-7/8 inches, and a minimum length of 24 inches, shall be pushed into the soil by a continuous or rapid motion without impact or twisting, preferably by the aid of a hydraulic jack, for a distance of about 6 inches less than the length of the tube. The drill rod shall then be rotated to shear the end of the sample and sample tube raised to the surface. The disturbed material in each end of the tube shall be completely removed, and metal disks inserted firmly at both ends, and the metal disks sealed over in the tube with paraffin.

Each tube shall be labeled with the job designation, boring number, sample number, and elevation at which sample was taken.

Drilling Records

The contractor shall keep an accurate chronological record of the progress of the work as well as a detailed log of each hole. He shall seal, store, pack, and deliver the samples as previously directed.

The records in all contain a description of the equipment used, the location and identifying number of test borings, and reference to survey data, date of start and completion of borings, ground surface elevation and ground water level at each boring, and datum used. The records shall further include depths at which changes in character of the soil take place and a record of the number of blows required to drive the spoon sampler 1 foot in undisturbed material.

For each undisturbed sample, the elevation of bottom of sampler at the start of taking each sample, the elevation to which samplers were forced into soil, length of the sample obtained, and the stratum represented by the sample shall be shown.

Samples shall be described from visual identification in the field as to general composition and predominance of the several grain sizes of materials as follows: gravel (coarse or fine), sand (coarse or fine), silt, clay, vegetable mulch, and peats, and such other terms as may be required. Mixtures shall be described using the predominating types of material as the first entry modified successively by terms indicating relative quantity and grain size. The relative moisture content may be indicated by the use of the terms "dry," "moist"," or "wet". The consistency shall be indicated by the terms "hard", "medium", or "soft", and the use of the terms "loose" or "compact" in coarse grain soils. The predominent color should also be designated. Copies of the contractor's records shall be furnished to the Engineer within three days of the completion of each hole.

Abandoned Holes

No payment will be made for any boring hole which has been abandoned by the contractor before reaching the required depths.

Cleaning Site

Upon completion of the work and before acceptance and final payment, all surplus material, temporary structures, and debris resulting from the work shall be removed and the premises left in neat orderly condition. All test holes on land shall be backfilled with earth and thoroughly compacted so no settlement will result. The cost of clean-up shall be included in the unit price bid on contract items.

Measurement and Payment

The various items shall be measured and paid for as follows:

Foundation Borings on Land (including dry samples)
Holes Nos. 1 to 10, inclusive, 28, 29, 33, and 34.

Payment for borings on land will be made on the basis of the prices bid per linear foot for "Foundation borings on land (including dry samples) from natural ground surface to a depth not exceeding 100 ft. below ground surface," or "Foundation borings on land (including dry samples) from a depth of 100 ft. below the natural ground surface to a depth not exceeding 200 ft. below ground surface," as the case may be.

Foundation Borings Through Water (including dry samples)
Holes Nos. 11 to 27, inclusive, 30, 31, and 32.

Payment for borings through water will be made on the basis of the prices bid per linear foot for "Foundation borings through water (including dry samples) from surface of water to a depth not exceeding 100 ft. below surface of water," or "Foundation borings through water (including dry samples) from a depth of 100 ft. below surface of water to a depth not exceeding 200 ft. below surface of water," as the case may be.

The above prices and payments for borings shall constitute full compensation for furnishing all apparatus, equipment including power boats, tow boats, barges, etc., for supplies and materials including casings, and for all tools, labor and incidentals necessary for making the borings and for the removal thereof upon completion of the contract, and for the securing, bottling, labeling, and delivering dry samples including the submission of records thereof. The surface of water shall be taken as Elevation 0.00, mean sea level, U. S. Coast and Geodetic Survey.

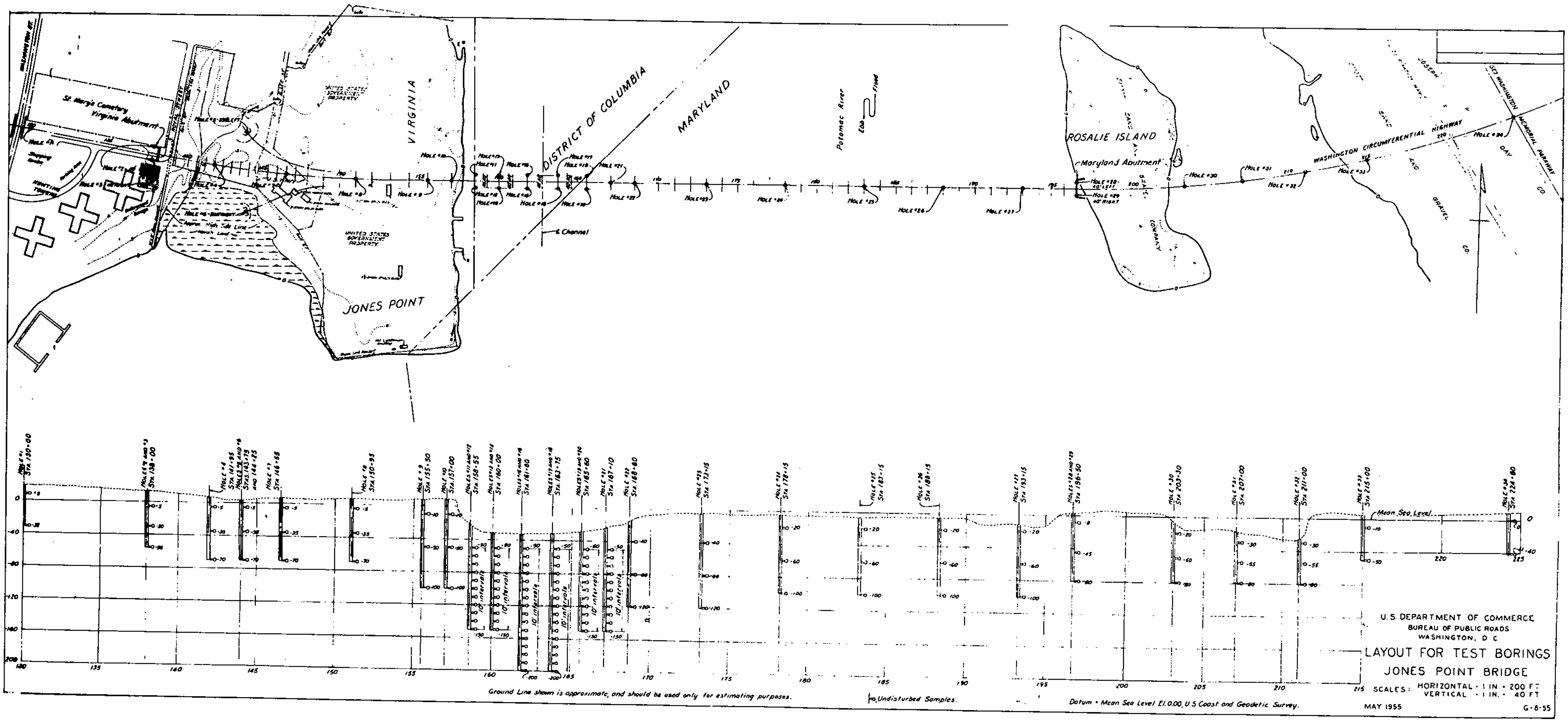
Securing Undisturbed Samples

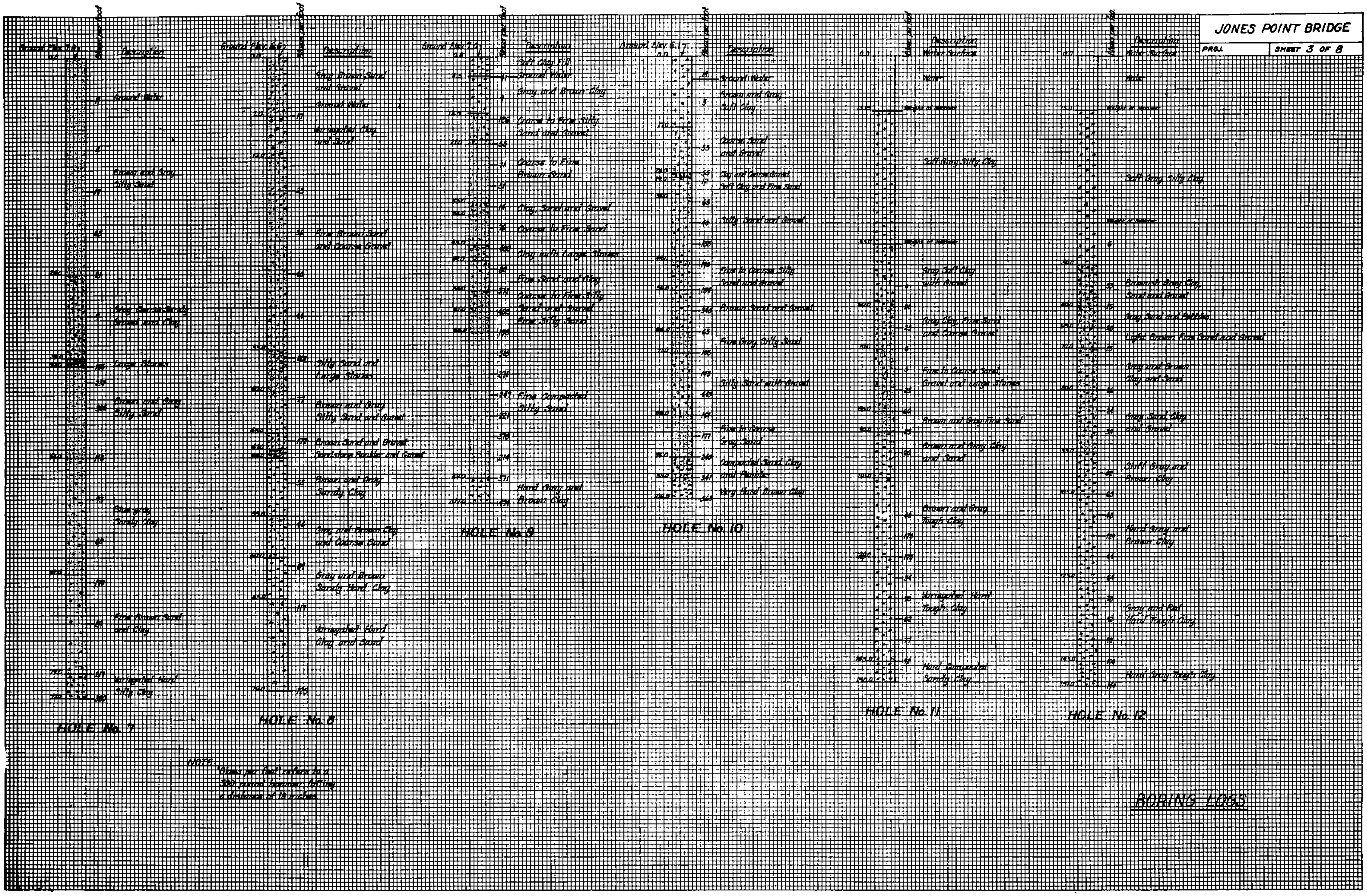
Payment for undisturbed samples will be made on the basis of the prices bid each for "Securing undisturbed samples at a depth not exceeding 100 ft. below surface of ground or surface of water," or "Securing undisturbed samples at a depth greater than 100 ft. below surface of ground or surface of water," as the case may be, which payments for undisturbed samples shall constitute full compensation for furnishing all equipment, tools, labor and incidentals necessary for securing the samples, for capping, packaging and delivering the samples, for the preparation and submission of required report, and for all other work necessary to complete the item.

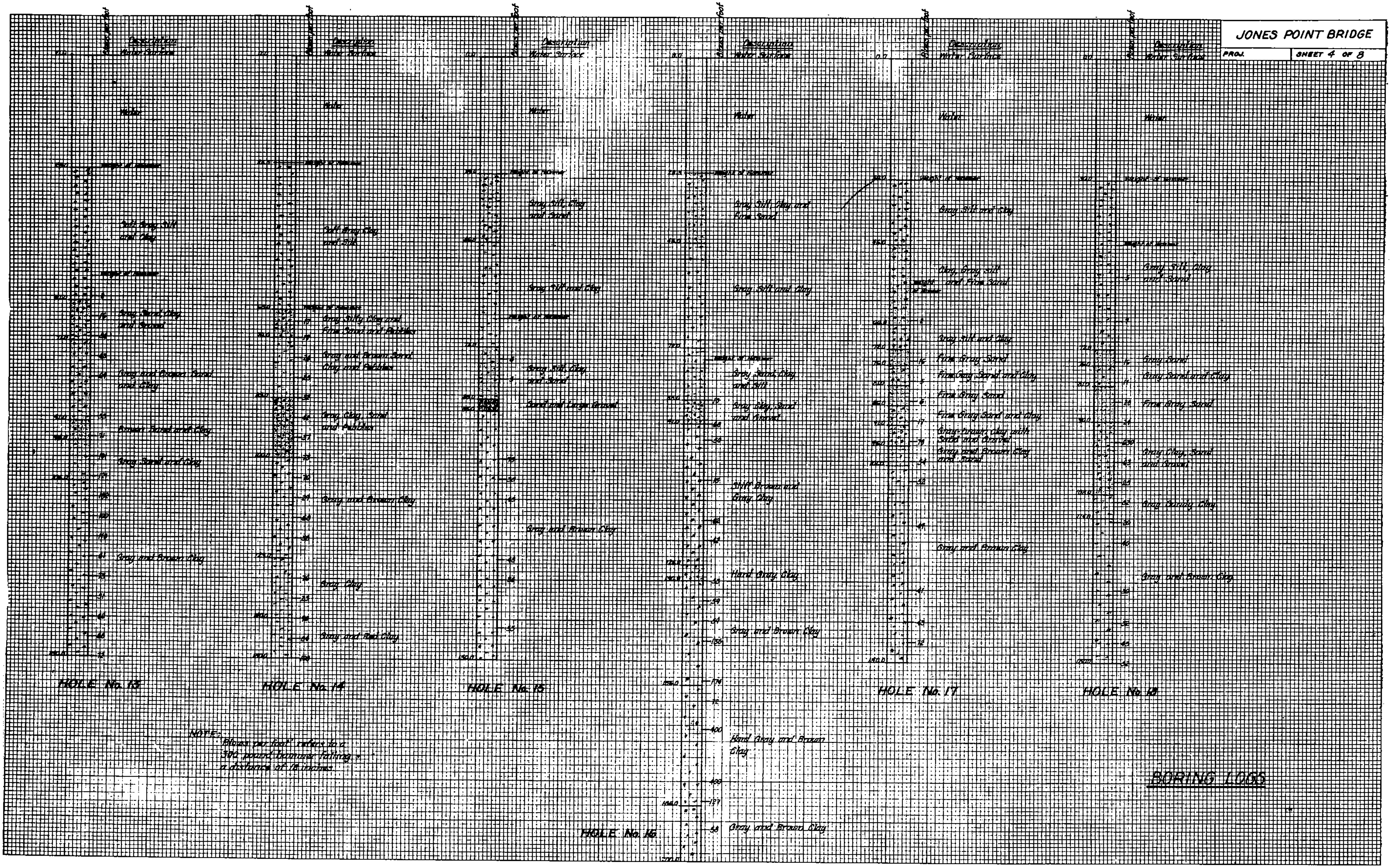
Although number, locations, and depths of borings, and number and location of the undisturbed samples are indicated on drawing G-8-55, the Engineer may make such changes as he may deem necessary as the operations proceed. Such changes shall be made without changes in the unit price bid. The Engineer reserves the right to increase or decrease the estimated quantities by 30 percent, either in depth and number of holes or number and location of undisturbed samples.

Final payment for all work shall be made on the basis of final quantities of work at the unit prices bid.

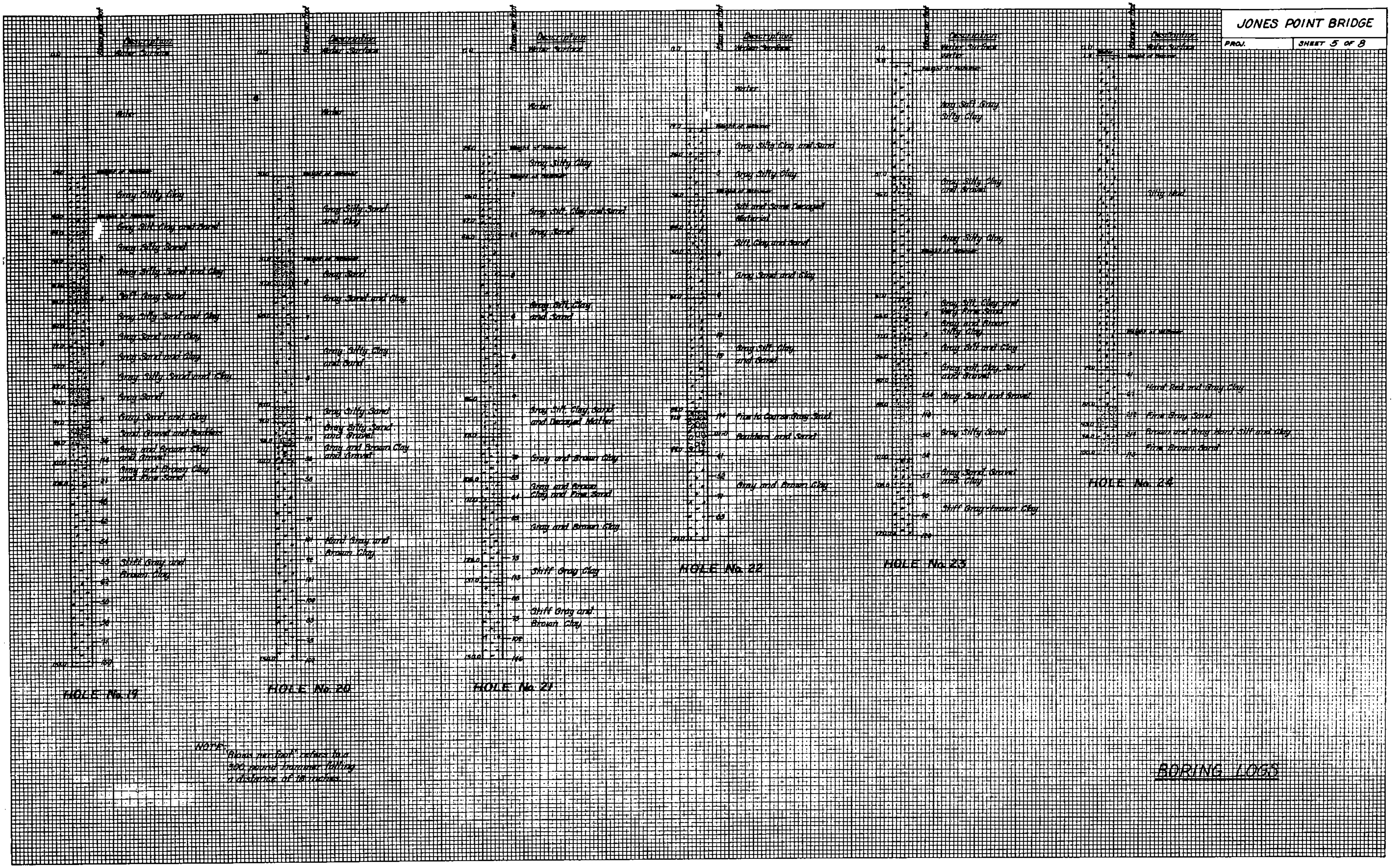
Quantities and depths shown on the drawings are estimates only and are to serve as the basis for comparison of bids and determination of awards.


St. Mary's Cemetery
Virginia Abutment
VIRGINIA
JONES POINT
UNITED STATES GOVERNMENT PROPERTY
DISTRICT OF COLUMBIA
MARYLAND
Potomac River
Ebb
Flood
Channel
ROSALIE ISLAND
Maryland Abutment
WASHINGTON CIRCUMFERENTIAL HIGHWAY
GEO WASHINGTON MEMORIAL PARKWAY
Mean Sea Level
Ground Line shown is approximate, and should be used only for estimating purposes.
Undisturbed Samples.
Datum = Mean Sea Level El. 0.00, U.S. Coast and Geodetic Survey.
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D.C.
LAYOUT FOR TEST BORINGS
JONES POINT BRIDGE
SCALES: HORIZONTAL - 1 IN. = 200 FT.
VERTICAL - 1 IN. = 40 FT.
MAY 1955
G-8-55


JONES POINT BRIDGE
PROJ.
SHEET 3 OF 8
HOLE No. 7
HOLE No. 8
HOLE No. 9
HOLE No. 10
HOLE No. 11
HOLE No. 12
NOTE: Blows per foot refers to a 300 pound hammer falling a distance of 18 inches.
BORING LOGS



JONES POINT BRIDGE
PROJ.
SHEET 4 OF 8
Blows per foot
Description
Water Surface
Water
Soft Gray Silt and Clay
Gray Sand, Clay and Gravel
Gray and Brown Sand and Clay
Brown Sand and Clay
Gray Sand and Clay
Gray and Brown Clay
HOLE No. 13
Soft Gray Clay and Silt
Gray Silty Clay and Fine Sand and Pebbles
Gray and Brown Sand Clay and Pebbles
Gray Clay, Sand and Pebbles
Gray and Brown Clay
Gray Clay
Gray and Red Clay
HOLE No. 14
Gray Silt, Clay and Sand
Gray Silt and Clay
Gray Silt, Clay and Sand
Sand and Large Gravel
Gray and Brown Clay
HOLE No. 15
Gray Silt, Clay and Fine Sand
Gray Silt and Clay
Gray Sand, Clay and Silt
Gray Clay, Sand and Gravel
Stiff Brown and Gray Clay
Hard Gray Clay
Gray and Brown Clay
Hard Gray and Brown Clay
Gray and Brown Clay
HOLE No. 16
Gray Silt and Clay
Gray Silt and Fine Sand
Gray Silt and Clay
Fine Gray Sand
Fine Gray Sand and Clay
Fine Gray Sand
Fine Gray Sand and Clay
Gray and Brown Clay and Sand
Gray and Brown Clay
HOLE No. 17
Gray Silt, Clay and Sand
Gray Sand
Gray Sand and Clay
Fine Gray Sand
Gray Clay, Sand and Gravel
Gray Sandy Clay
Gray and Brown Clay
HOLE No. 18
NOTE: "Blows per foot" refers to a 300 pound hammer falling a distance of 18 inches.
BORING LOGS


JONES POINT BRIDGE
PROJ.
SHEET 5 OF 8
HOLE No. 19
Water
Gray Silty Clay
Gray Silt, Clay and Sand
Gray Silty Sand
Gray Silty Sand and Clay
Soft Gray Sand
Gray Silty Sand and Clay
Gray Sand and Clay
Gray Sand and Clay
Gray Silty Sand and Clay
Gray Sand
Gray Sand and Clay
Sand, Gravel and Boulders
Gray and Brown Clay and Gravel
Gray and Brown Clay and Fine Sand
Stiff Gray and Brown Clay
HOLE No. 20
Water
Gray Silty Sand and Clay
Gray Sand
Gray Sand and Clay
Gray Silty Clay and Sand
Gray Silty Sand
Gray Silty Sand and Gravel
Gray and Brown Clay and Gravel
Hard Gray and Brown Clay
HOLE No. 21
Water
Gray Silty Clay
Gray Silt, Clay and Sand
Gray Sand
Gray Silt, Clay and Sand
Gray Silt, Clay, Sand and Decayed Matter
Gray and Brown Clay
Gray and Brown Clay and Fine Sand
Gray and Brown Clay
Stiff Gray Clay
Stiff Gray and Brown Clay
HOLE No. 22
Water
Gray Silty Clay and Sand
Gray Silty Clay
Silt and Some Decayed Material
Silt, Clay and Sand
Gray Sand and Clay
Gray Silt, Clay and Sand
Fine to Coarse Gray Sand
Boulders and Sand
Gray and Brown Clay
HOLE No. 23
Water
Gray Silt, Gray Silty Clay
Gray Silty Clay and Gravel
Gray Silty Clay
Gray Silt, Clay and Very Fine Sand
Gray and Brown Silty Clay
Gray Silt and Clay
Gray Silt, Clay, Sand and Gravel
Gray Sand and Gravel
Gray Silty Sand
Gray Sand, Gravel and Clay
Stiff Gray-Brown Clay
HOLE No. 24
Silty Mud
Hard Red and Gray Clay
Fine Gray Sand
Brown and Gray Hard Silt and Clay
Fine Brown Sand
NOTE: "Blows per foot" refers to a 300 pound hammer falling a distance of 18 inches.
BORING LOGS

RIVER BED SCOUR AT BRIDGE FOUNDATIONS

by

Emmett M. Laursen
Research Engineer
Iowa Institute of Hydraulic Research
State University of Iowa

INTRODUCTION

To the geologist a river is an important agent in the erosion and transportation of the soil mantle of uplifted highlands. The erosional half of the geological cycle has as a limiting condition the classical concept of the peneplain in which erosion and deposition combine until relief is almost obliterated. This limit is probably never achieved because of the continual movement of the earth's crust. Thus, in the geological sense a river is never in equilibrium, but is always active and changing its environment.

To the engineer a rate of change which is significant in geologic time may well be practically negligible. The engineering concept of a stream which is poised or in regime, therefore, is not equivalent to the geological concept of equilibrium. The condition of geological equilibrium is represented by the limiting peneplain whereon the slopes are the maximum consistent with the critical tractive force for the beginning of movement; engineering equilibrium is a condition wherein the active transportation of sediment does not result in a significant rate of erosion or deposition.

This active character of the stream can be safely ignored in the design of the piers and abutments only if the approaches and foundations of a bridge do not disturb the natural flow conditions of a stream, or if the substructure is founded on sound rock. Otherwise, the scour which will occur should be considered in the determination of foundations which will be both safe and economical. The lowering of the stream bed in the vicinity of piers and abutments can be ascribed to three causes: the local scour due to the distortion of the flow pattern by the pier or abutment itself, the general scour due to the disturbance of the flow pattern by the overall bridge-crossing geometry, and any degradation of the stream which may occur whether natural or caused by the works of man.

The scour at bridge piers and abutments has been studied since 1948 by the Iowa Institute of Hydraulic Research under the joint sponsorship of the Iowa State Highway Commission and the Bureau of Public Roads. From this experimental investigation a method of predicting local scour at bridge piers has been evolved (1). The study is continuing in an effort to obtain a comparable method of predicting general scour which is important in both abutment and pier scour. The last cause of stream-bed lowering degradation, is not a problem associated with the bridge crossing but with the general behavior of the stream itself.

LOCAL SCOUR

For turbulent flow such as is found in a river, the boundary geometry will determine the flow pattern around an obstruction such as a pier. Included in this comprehensive term, is the shape of the pier itself, its relation to the direction of flow, the depth of flow, and the proximity of other piers or banks of the river. Since the flow pattern will determine the capacity for sediment transport at every point, the final equilibrium bed configuration is also determined. The equilibrium form will be such that at every point the capacity for transport is equal to the rate at which sediment is supplied. Until this balance is achieved, active scour and deposition will occur.

The typical form of a scour hole around a pier is shown in Figure 1. The forepart of the scour hole can be approximated by a truncated, distorted cone with side slopes equal to the angle of repose; the afterpart of the scour hole is composed of two tails on each side of the wake behind the pier, and a built-up dune in the wake of the pier. A spiral roller within the scour hole is the active agent of erosion and transportation.

The effect of both depth of flow and proximity of piers can be seen in Figure 2, which summarizes a series of experiments on multiple round cylinders. Until the scour holes from adjacent piers meet, the contraction of the flow does not appear to have any influence on the depth of scour. For sufficiently large contractions the depth of scour is equal to that of a long contraction, which according to Straub (2) is

(1)

where d_s is the depth of scour, y is the depth of flow in the uncontracted approach, and (1 - B) is the ratio of the contracted to the uncontracted width.

Similar experiments established the effects if pier shape, angle of attack, and depth of flow. Equally important a finding was the lack of effect of velocity of flow and sediment size. The techniques, results, interpretations, and qualifications of the investigation and a method for predicting the local depth of scour at piers are fully reported in Reference 1 and, therefore, will not be repeated here. The basic relationship for the prediction of scour is shown schematically in Fig. 3. The depth of scour at a rectangular pier aligned with the flow is first determined as a function of the depth of flow and the width of the pier. If the pier is not aligned with the flow, the scour depth at zero angle of attack is multiplied by a factor greater than unity which depends on the angle of attack and the length-width ratio of the pier. Only if the pier is aligned with the flow can the effect if a rounded or streamlined pier shape be considered. A table of coefficients less than unity has been prepared for this condition.

GENERAL SCOUR

Basically all scour phenomena are similar in that for any flow pattern there will be an equipibrium bed configuration so that at every point the capacity for transport is exactly equal to the rate at which sediment is supplied. General scour at a bridge crossing differs from local scour only in the factors which affect the flow pattern, or, more specifically, the pattern of sediment-transport capacity. It is to be expected that the general scour will be a function of such geometric characteristics as the width-depth ratio of the main channel, the relative location of the abutment to the bank of the stream bank, the presence of thick growth along the bank or of a spur dike parallel to the stream at the end of the approach fill, and the angle between the approach fill and the flood flow. Since both the local scour at the abutment and the general scour at the crossing will be centered on the upstream corner of the abutment, they can not be measured separately and any investigation must also include a geometry of the abutment itself. It is anticipated that the effect of the overbank flow can be described by the ratio of the total discharge to the discharge in the main channel.

By an approach similar to that of Straub previously cited, the scour depth in a stream where only the overbank flow is contracted can be obtained (3). The solution is for the case if a contraction in which the flow is uniform and in which the overbank flow does not transport a bed load. As in the case of the multiple cylinders, the scour at a bridge crossing will be greater than that in a long contraction because of the non-uniformity of the flow. An estimate of the minimal scour to be expected, however, can be obtained from the relation

(2)

If the general scour extends to the piers, the effects of both general and local scour must be added to obtain the total lowering of the river bed at the pier. Independently from the effect of the general scour it should be noted that the flow around the abutment may determine the angle of attack between the flow and the pier and thus may also affect the local scour depth.

The continuing experimental investigation has been designed to assess the effect of these various factors with the goal of establishing means of predicting general scour, both as to depth and extent, similar to those evolved for the local scour around piers.

DEGRADATION

Any scour, general or local, caused by various elements of the bridge crossing must be measured from the normal bed elevation of the stream. Lane and Borland (4) have demonstrated conclusively that there is no appreciable overall lowering of the river bed during a flood. Rivers, however, are not uniform in section and scour will occur in the contracted reaches during floods. Equations (1) and (2) indicate the magnitude of this effect. Especially in bends, there may also be a natural rearrangement of the flow pattern at the higher stages of the flood - aside from and indepent of the effect of the bridge crossing. Any change in the flow pattern, of course, is liable to result in a new bed configuration. As an extreme of this behavior, some of the wide, shallow, western streams are very erratic in the shifting of deep channels.

In addition to this normal variability of the stream channel, a river may be degrading, usually rather slowly. In an alluvial valley the degradation will usually be the result of man's interference with the stream. Cutoffs or stream straightening will cause degradation upstream approximately equal to the slope of the stream multiplied by the reduction in the length of the stream. Because the bed load is trapped in a reservoir, degradation will extend far downstream from a dam unless the sills of the spillway section are essentially at bed level, as, for example, on the navigation dams on the Ohio and Mississippi. Any disturbance of the normal functioning of the stream which effectively increases the capacity for transport or decreases the sediment supply will result in a tendency for degradation. This lowering of the stream bed may be small in any one year but may become important within the useful life of a bridge.

CONCLUSIONS

Successful application of the proposed method of predicting local scour and of its anticipated counterpart for general scour depends on an understanding of the general nature of the scour phenomenon, an ability to visualize flow patterns, and a knowledge of the behavior of the individual stream. Any engineer with a background which includes the study of hydraulic engineering or fluid mechanics should be qualified to handle the methods after a reasonable amount of study of the particular problems involved.

Confidence in the methods of predicting scour can only be built up through successful experience. Field measurements and model prototype conformity tests on existing structures can provide the factual information in an expeditious manner. One such study has been made (1) which not only provided valuable data but served as a proving ground for the instrumentation and techniques of operation. Another type of study which could be made is a correlation of failures and non-failures of bridges in past floods with conditions of scour as predicted by these methods.

References

1. Laursen, E.M., and Toch, A., Scour around Bridge Piers and Abutments, Bulletin No. 4, Iowa Highway Research Board, in press.
2. Straub, L.G., "Approaches to the Study of the Mechanics of Bed Movement" Proceedings, Hydraulics Conference, State University of Iowa Studies in Engineering, Bulletin 20, 1940.
3. Laursen, E.M., and Toch, A., discussion of Reference 4.
4. Lane, E.W., and Borland, W.M., "River Bed Scour during Floods" Trans. ASCE, Vol. 119, 1954.

List of figures

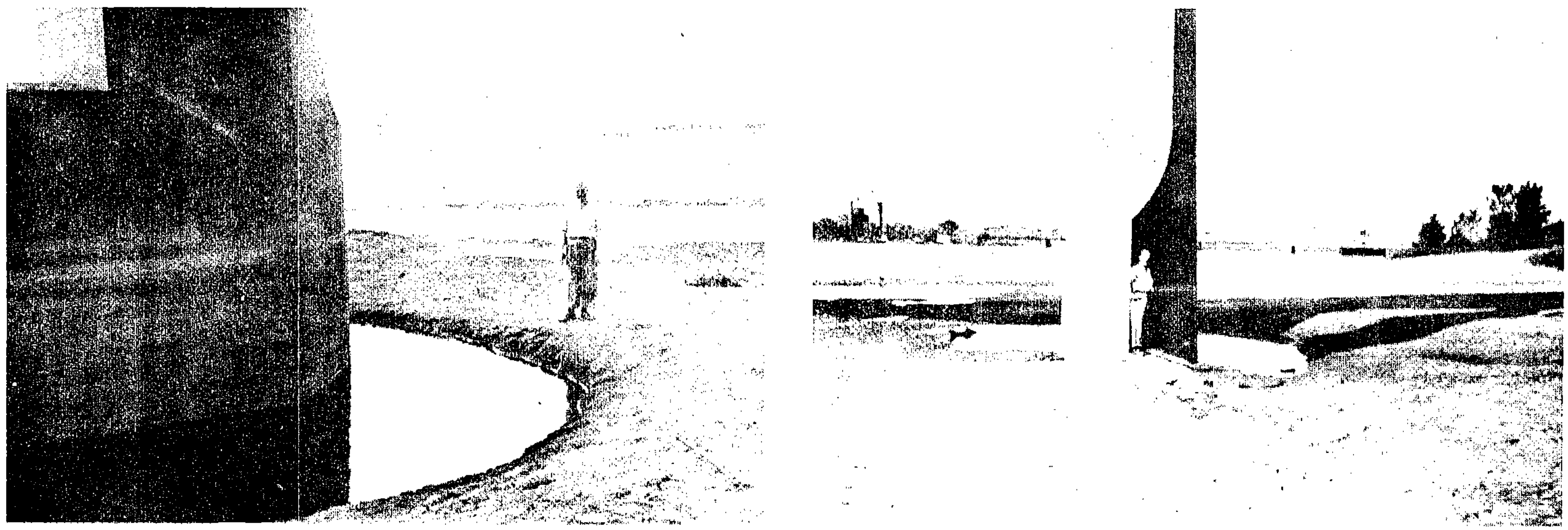


FIGURE 1

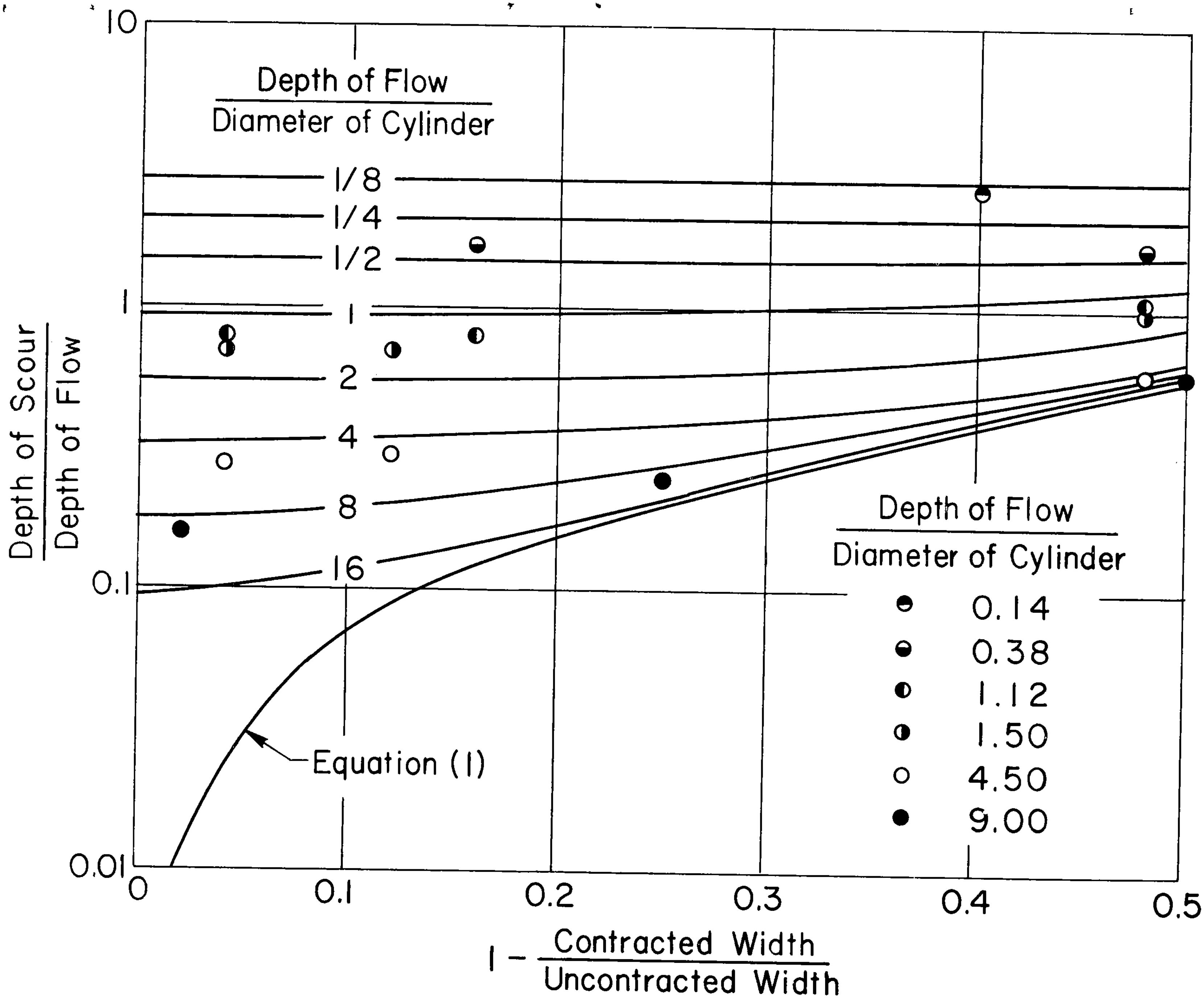

10
1
0.1
0.01
Depth of Scour / Depth of Flow
Depth of Flow / Diameter of Cylinder
1/8
1/4
1/2
1
2
4
8
16
Equation (1)
Depth of Flow / Diameter of Cylinder
0.14
0.38
1.12
1.50
4.50
9.00
0
0.1
0.2
0.3
0.4
0.5
1 − Contracted Width / Uncontracted Width

FIGURE 2

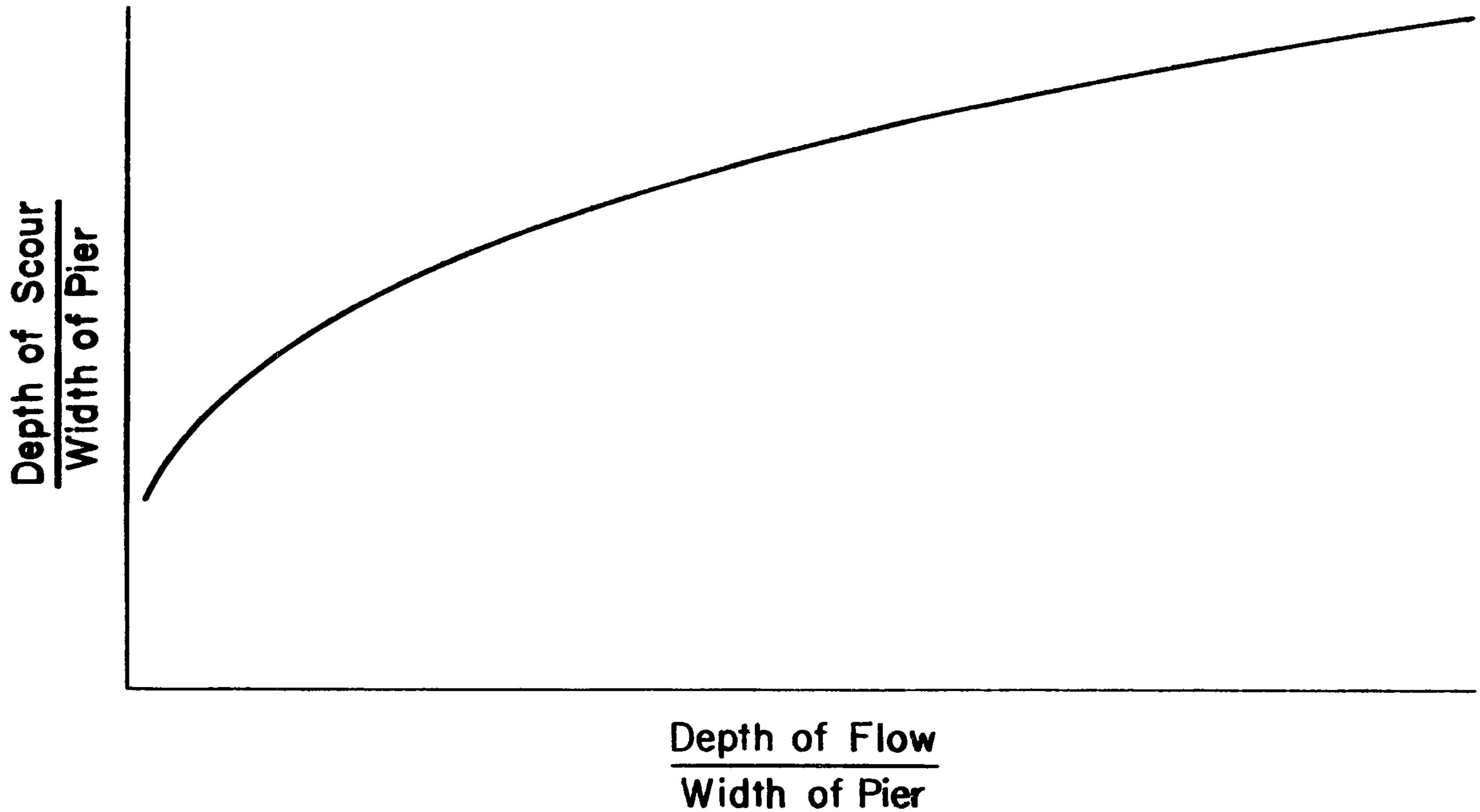


FIGURE 3

Geologic Features of the Southeastern States as Related to Highway Engineering

By Robert A. Laurence
U. S. Geological Survey
Knoxville, Tennessee

Geology as related to highway engineering can be divided into three main groups:

1. Geology as related to problems of location, design, and construction.

2. Geology as related to the location and selection of construction materials.

3. Geology as related to the occurrence and control of water in highway construction and maintenance.

This paper will discuss principally the first two of these. A general discussion of the third was presented to the Second Annual Symposium at Richmond in 1951 (DeBuchananne, 1951).

Geologic problems of highway engineering are more or less similar within a single geologic or physiographic province, but differ strongly from one province to another. Thus, the problems of building a bridge over the Potomac River at Jones Point, near Washington, in the Coastal Plain are quite different from those of a bridge over the same river at Harpers Ferry in the Appalachian Valley, while the bridge foundation at Harpers Ferry would have many geologic features in common with the foundation of a bridge over the Tennessee River at Chattanooga.

Following is a summary of the geologic features of the various physiographic provinces of the southeastern states (Fenneman, 1938 pp. 1-342, 411-448, pls. II, III):

Coastal Plain.--Chiefly unconsolidated or poorly consolidated sedimentary rocks--sands, clays, marls. Cuts and fills subject to rapid gullying; slides may occur where clay beds underlie thick permeable sands. Bridge foundations require extensive exploration, as crystalline bedrock lies too deep to be used for foundations except along the inner margin of the Coastal Plain. Materials for aggregates are chiefly gravel and sand. In Atlantic seaboard region, gravels are largely composed of quartz; in western Gulf and Mississippi embayment region chert and sandstone make up much of the gravels and although usually sound, they may be inferior to quartz and novaculite gravels or crushed stone.

Piedmont.--A region of relatively low relief, composed largely of metamorphic and igneous rocks (schists, gneisses, granites and slates) with small areas of sedimentary rocks; entire region is deeply weathered so that bedrock is mostly covered with residual clay or saprolite. Large quarries in granite, gneiss, quartzite, and limestone supply much of the area, but in many areas good quarry sites do not exist. Stream gravels and sand are used locally.

Blue Ridge.--A mountainous area underlain by rocks comparable to those of the Piedmont but generally less weathered, so that a much greater portion of cuts are in solid rock and so that bedrock is usually at slight depth at bridge sites. Strike and inclination of foliation and jointing has considerable effect on stability of cuts (Hadley 1952). Construction materials abundant and good quarry sites are available almost anywhere; granite common but limestone scarce.

Valley and Ridge province.--An area of parallel ridges and valleys, underlain by folded, generally steeply dipping, sedimentary rocks. Deep weathering and solution of carbonate rocks has produced a thick residual cover over much of the region; depth to bedrock is variable. Cavities in limestone affect foundations of bridges and other structures. Cuts which cross strike of dipping rocks at right angles are usually best; sliding is easily started by cuts parallel to strike (Parrott, 1952, Laurence, 1951). Limestones and dolomites abundant; quarry sites available at most localities.

Appalachian Plateau.--A mountainous region of horizontal sedimentary rocks, chiefly sandstone and shale; very little limestone. Slumping and rockfalls resulting from sapping by shale beds beneath thick sandstone is a widespread problem; benching of cuts provides best solution (Baker, 1952, Philbrick, 1953; Marshall, 1953). The general absence of soluble carbonate rocks is favorable to bridge and tunnel sites. Some sandstones are suitable for aggregate (Gregg & Havens, 1953); limestone is available at a few places in the region or from adjacent regions where it is abundant.

Interior Low Plateau.--A hilly area underlain by flat-lying sedimentary rocks; much less relief than Appalachian Plateau. Limestone is abundant, and much of it is cherty. Caves and areas of deep weathering are common.

REFERENCES CITED

Baker, R. F., 1952, The design of the slopes of highway rock excavations in West Virginia: 3rd Annual Symposium on Geology as applied to Highway Engineering, Proceedings, 1952.

DeBuchanne, G. D., 1951, Control of ground water in consolidated rocks: 2nd Annual Symposium on Geology as applied to Highway Engineering, Proceedings, 1951.

Fenneman, N. M., 1938, Physiography of Eastern United States, New York, McGraw-Hill Book Co., 714 p.

Gregg, L. E. and Havens, J.H., 1953, Applications of geology to highway engineering in Kentucky: 4th Annual Symposium, 1953, pp. 1-11.

Hadley, J. B., 1952, Geology of a highway slide at Gatlinburg, Tenn.: 3rd Annual Symposium, 1952.

Laurence, R. A., 1951, Stabilization of some rockslides in Grainger County, Tenn.: Economic Geology, vol. 46, pp. 329-336.

Marshall, H. E., 1953, Some experience of the Department of Highways with landslides in Ohio: 4th Annual Symposium, 1953, pp. 12-23.

Parrott, W. T., 1952, Geologic problems in the design and construction of highways in Virginia: 3rd Annual Symposium, 1952.

Philbrick, S. S., 1953, Design of deep rock cuts in the Connemaugh formation: 4th Annual Symposium, 1953, pp. 79-88.

EQUIPMENT USED IN GEOLOGICAL ENGINEERING

by

J. O. Bickel

When planning a highway and its related structures, one of the most important considerations is: What type of soil are we going to build on or in? In seeking the answer, geological engineering can employ a number of tools, some old, others new, and all having their special uses.

Soil exploration may be divided roughly into two phases:

The first furnishes general information at a low cost in the minimum time for preliminary planning and route studies. This information includes:

> Visual surface inspection from the ground, or aerial photography and observation.
>
> Probings and simple borings.
>
> Geophysical exploration.

The second group produces the more accurate data about soil properties required for the final foundation design of structures and the classification of materials which will be encountered during highway construction. Here belong:

> Borings furnishing samples of soils for analysis and laboratory tests, or rock cores. These may be combined in some cases with further geophysical investigation, particularly the seismic refraction method.

Surface Inspection

A good pair of observing human eyes, backed up by ample knowledge and experience and, today even as in the past, the support of a pair of sturdy legs, might be called the initial tool, and the basic one, for surface exploration. Today the range of vision has been vastly increased by the airplane and the camera - with its black and white as well as color pictures. Not only do these permit quick mapping and stereometric determination of contours but they also make it possible for a skilled observer to deduct an amazing amount of geological information. He can thus quite readily spot areas which would obviously be troublesome, such as swamps, and determine a great deal about the nature of the underlying soil from the type of vegetation and coloring of the ground surface.

Probings

As soon as some likely route have been selected by preliminary studies, based upon the maps and information produced by aerial surveys, exploring below the surface becomes essential. The simplest tool is a probing bar driven into the ground. This will give an approximate idea of the firmness of the soil and the location of any rock at shallow depth - provided the rock does not consist of a series of boulders! The results of these probings have, obviously, only a very limited value.

Auger Borings

More informative are auger borings. In this case, an auger-like drill, 3 to 8 inches in diameter, is screwed into the ground, and the material brought up by it is typed. A depth of as much as 20 feet may be reached. A series of such borings, spaced at approximately 500 foot intervals, usually give adequate information to enable us to classify the material for excavation and fill for regular highway construction.

Geophysical Exploration

For more complete and rapid subsurface surveys, geophysical explorations are rapidly gaining favor. These comprise four basic methods:

Measurement of electric resistivity of the ground.
Measurement of seismic refraction.
Measurement of seismic reflection.
Magnetometric surveys.

For highway work only the first two have found practical application, and of these two seismic refraction has become the more widely used.

Electric Resistivity

Resistance to the flow of electric current varies widely in different soils. Dry silicon and other minerals, which form the solid component of all non-organic soils, are practically non-conductors of electricity. Rocks and dry sands have, therefore, a high resistivity and act as conductors only because even they contain small amounts of water. It is the latter which principally determines the conductivity of any soil, so that the lowest resistivity is found in loose materials, such as sand and gravel below the water table, provided the water contains some dissolved salts. For this reason, the electric resistivity method has proven especially useful when exploring for water supplies. Here, a pair of electrodes is inserted about a foot into the ground and a curren is passed through the soil between them at a potential of about 100 volts. Intermediate electrodes are used to measure the current pattern. The further apart the two base electrodes are spaced the deeper the section of soil through which the current passes. By varying the distance resistivity depths can be calculated and conclusions drawn regarding the nature of the soil. An attempt was made to determine the solid rock profile in the portal areas of the T. J. Evans Tunnel by this method (refer to Mr. R. Woodward Moore, B.P.R.). The agreement of the results with those obtained by later borings was not very good, which is understandable in view of the mass of talus covering the slopes of the mountain.

Seismic Refraction

Different materials propagate sound or shock waves at greatly different speeds. This also applies to soils, as the following figures indicate:

Velocity of shock waves is:

1000 to 2000 ft./sec. in dry loam.
1500 to 3000 ft./sec. in dry sand and loose till.
3500 to 5500 ft./sec. in compact tills, gravel in water table, and some clays.
6000 to 8500 ft./sec. in old tills, hardpan, and across schistosity of foliated rock.
12000 to 15000 ft./sec. in solid rocks.

The seismic refraction method uses these differences to determine the thickness of soil layers by measuring the speed with which a shock wave from an explosion is propagated. It is most readily applied to find the depth of solid rock below a covering of loose material, say gravel. An explosive charge, usually 1/4 to 1/2 lb. of high velocity dynamite, is detonated between 3 to 8 feet below ground surface. From this point, seismometers are set on the ground in a straight line at intervals of 20 to 100 feet, depending upon the accuracy of detail desired. Electric oscillators measure the time interval between the firing of the charge and the arrival of the wave at each seismometers. Waves travel over different paths. One will reach the instruments through propagation in the gravel at a speed of approximately 3000 ft./sec. Another wave will go through the gravel down into the rock where it will be refracted and will propagate through the rock at the higher speed of approximately 12,000 ft./sec. This will cause waves to be refracted up through the gravel and these will be detected by the instruments. At a certain distance from the origin, the refracted wave will overtake the direct wave, due to the higher velocity imparted to it by the rock. This is called the "point of intersection", and the greater the thickness of the gravel the further away this point will be. If the arrival times of the wave at the various seismometers are plotted as ordinates over the distances as abscissae, the two wave paths are represented as straight lines with different slopes. Their intersection coincides with the "point of intersection", where the refracted wave arrives simultaneously with the direct wave. Their slopes permit the calculation of the two velocities. If the rock surface is parallel to the ground, the true velocity in the rock can be figured. If the surfaces are divergent, another reading in the reverse direction, that is with a second charge fired at the other end of the track, will be required so that the true rock velocity can be averaged out. Best results are obtained when the rock surface is relatively regular. Abrupt dips and rises of the rock are apt to give confusing readings. In order to reduce the chance of errors due to local irregularities it is usually necessary to use spreads of tracks, some running at right angles to each other. Only layers with large differences in refracting characteristics can be identified. The method requires great care and considerable experience to insure accurate results. In a recent demonstration over a stretch of the Connecticut Expressway, where borings had previously been made, the coincidence between the rock profiles established by the two methods was reasonably close. On the other hand, in a subaqueous tunnel in Europe now under preliminary study, seismic refractions indicated about 30 feet of alluvial material over the bedrock, but subsequent borings found this cover to be as great as 200 feet, which necessitated a change in the whole concept of the project.

Borings

Before starting the final design of a highway structure, such as a bridge or tunnel, it is necessary to have considerably more information about the soils than was secured for the preliminary studies. More comprehensive geophysical exploration may give results adequate to determine the areas along a highway where rock must be removed from cuts, but for structures, and sometimes for fills and embankments, only one tool will fill the bill: borings, which lift soil samples from the depths for inspection and analysis. They may often be combined to advantage with geophysical tests.

The simplest type is the auger boring, previously mentioned, for shallow depth. An auger, as all of you undoubtedly know, is a helical drill, with some kind of cutting edge, which is screwed into the ground. The helix fills with the soil which is brought to the surface when the drill is raised. The samples, which are naturally quite disturbed, are suitable for classification only.

When deeper layers are to be explored and samples which more nearly represent the soil in its natural condition are required, the cased boring is used.

A casing of extra strong steel pipe, usually 2-1/2" to 4" in diameter, is driven into the ground and the material washed out from its inside by water from a jetting pipe lowered into the casing. The casing is driven by a falling weight, as in the old type of pile driving, and a record is kept of the number of blows per foot. Every five feet, or when the color of the washwater, or the driving of the casing indicates a change in material, a dry sample is taken from the bottom of the hole. The casing is cleared to the required depth by fishtail or auger bits and washwater. A sampler of a diameter slightly less than the inside diameter of the casing is attached to drillrods and driven into the soil below the end of the casing. Basically, the sampler is a tube 24 inches long with a cutting edge at the lower end. The upper end is screwed into a head threaded to fit the drill rod and bored with a small hole acting as a vent. A ball check closes this vent when the sampler is withdrawn, thereby preventing the sample from dropping out. At the surface the sample is removed, immediately placed in a Mason jar and labelled with the identifying bore hole, the number and depth from where it came, and classification of the material.

Samplers are either solid or split lengthwise. The latter are very popular because they allow easy removal of the sample by opening up the spoon like a book.

Since the number of blows required to drive the sampler is a good indication of the density and bearing capacity of the soil, this information is carefully entered in the boring log.

Undisturbed Samples

For more complete determination of soil characteristics by Laboratory tests, undisturbed samples are recovered whenever possible. For this purpose the regular pipe sampler is replaced by a thin-wall tube, called a Shelby Tube, attached to a similar type of head equipped with ball check. The tube has a wall thickness of 1/16" and a sharp cutting edge. It comes in outside diameters of 2 to 4-1/2 inches and has a standard length of 24 inches. After the hole has been cleaned out the tube is pushed into the undisturbed soil below the casing with a steady pressure rather than by a falling weight. Upon withdrawal, the tube is removed from the head, sealed at both ends to preserve the moisture content of the sample, and shipped to the laboratory. Undisturbed samples can be obtained only of soils which have adequate cohesion, such as silty sand, silt, or clay. Pure sand will not remain undisturbed in the tube upon withdrawal.

There are a number of special samplers available to suit most types of soils encountered, such as sticky or loose materials.

If the ground becomes very firm the wash water may not be able to remove the material from the casing. In this case it has to be loosened up with special tools, such as chopping bits, similar to chisels, or cross chopping bits, which will cut through small boulders and coarse gravel. Fishtail bits with power rotation can cut through hardpan and soft shale. If the material becomes firm enough, a hole may be continued without casing.

Laboratory Tests

In the laboratory the undisturbed samples are cut up into suitable pieces to perform all or a part of the following tests:

Unconfined compression: This gives an accurate measure of the ability of a cohesive soil to stand on slopes or support foundation loads.

Triaxial Compression: A cylindrical specimen subjected to predetermined lateral pressure is loaded to failure under axial compression, to determine the angle of friction.

Consolidation: To determine the amount and time curve of consolidation under certain loads.

Compaction Tests: To determine how much the density can be increased by compaction.

Other general tests are sieve analysis, to determine the grain sizes; specific gravity, capillarity, etc.

Rock Core Borings

Where rock is encountered, such as in tunnel construction, or under heavily loaded foundations, core borings are made with diamond drill bits. These are short lengths of heavy pipe, one end of which is threaded to fit the core barrel. The forward end is rounded and set with a number of small diamonds cemented into recesses. These may number as many as a hundred or more in a 3-inch bit. The bits come in various sizes, giving cores of 7/8" to 2-1/8" diameter. They are mounted on core barrels, steel pipes 5 to 20 feet long, which are screwed to the drill rods. The latter are hollow to allow drill water to be pumped to the bit. Here the water will cool the bit and wash away the drilling dust. More details of the operation will be explained later when describing the borings made for the T. J. Evans Tunnel of the Pennsylvania Turnpike.

Borings for Hampton Roads Bridge-Tunnel

Two recent projects, both now under construction, may serve as illustrations of practical boring programs.

The first of these is the Hampton Roads Bridge-Tunnel Project, consisting of a 7500 ft. tunnel under the Hampton Roads Ship Channel, with about three miles of trestle bridges completing the water crossing, and a total of 20 miles of limited access approach highways.

SLIDE 1: GENERAL PLAN OF PROJECT

For the highways, cased borings were made at all bridge crossings. Over the rest of the alignment, where no structures were involved, auger borings every 500 feet sufficed.

SLIDE 2: TYPICAL PLOTTING OF THESE PROBINGS FOR HIGHWAY

The main crossing - especially the tunnel section - presented a major design and construction problem. The distance between shores is over four miles. The main ship channel has a width of 3700 feet between 6-fathom lines and a maximum depth of about 70 feet; the rest is shallow water from 20 to 4 feet deep.

SLIDE 3: PLAN OF THE CROSSING

SLIDE 4: TUNNEL PROFILE

The tunnel portals, open approaches and ventilation buildings are located on two artificial sand islands.

SLIDE 5: BORING PLAN

Borings were spaced 500 feet along the center line of the project. They were started with 4" casings; this was reduced to 2-1/2" after a penetration of about 60 feet. Since an exact soil analysis was important, undisturbed samples were taken at frequent intervals in the holes in the tunnel and island areas. Over the projected islands pairs of holes were used, and on the south side of the channel additional borings were located at offsets up to 1500 feet from the center line. The purpose of these was to explore the extent of the deep layer of silt encountered at this location.

SLIDE 6: TYPICAL LOG

The following conclusions were reached from the analysis of the soil samples made by the laboratories of the University of Virginia:

On the north side of the channel the tunnel island could be placed directly on the existing bottom as hydraulic fill.

At the south island a layer of silt up to 90 feet in depth was found to be compressible to such an extent that radical measures were needed to protect the structures against excessive settlement. After several solutions had been studied it was decided that removal of the silt and replacement with good sand was the most economical method. The samples indicated sufficient cohesion of the material to permit slopes no steeper than 2:1 for the excavation; this was borne out during actual construction.

The trench to be dredged for the tunnel sections would be located for most of its length in firm sand; only at the south side would it be partly in silt, most of which would be removed anyway.

SLIDE 7: SOIL PROFILE

Holding the trench slopes and loading the subsoil by tube and backfill did not, therefore, pose any problem, except for the last 300-foot tube on the south side, and the short length of cut-and-cover section. In order to avoid increasing the load on the subsoil, a lightweight type of backfill will be placed over these parts. The tubes themselves weigh less than the material they displace.

The hydraulic sand fill of the two islands was compacted by the vibroflotation method to obtain a density sufficient to support the structures without piles. Borings were taken after compaction to obtain samples for checking the density obtained, which was specified at a minimum of 75 per cent relative density. Hydraulically operated piston tube samplers were used for this sampling with satisfactory results.

Twelve of the 23 sunken tubes have now been placed and the sand islands are practically completed.

T. J. Evans Tunnel

The T. J. Evans Tunnel, which will carry the Northeast Extension of the Pennsylvania Turnpike through the Blue Mountains near Palmerton, Pa., presented a quite different soils problem, namely the exploring of rock formations prior to the design and construction of a rock tunnel.

The tunnel will have a length of about 4500 feet. At the midpoint it is some 800 feet below the top of the mountain. Geological studies made of the area over a period of years indicate that the formation consists of shales, quartzites, and metamorphic sandstone, rather faulted, with a dip between 30 and 60 degrees south and an east-west strike parallel to the ridge and more or less at right angles to the tunnel axis. Both slopes are well covered by talus, the south more so than the north.

It was decided that longitudinal borings offered the best chance to obtain a record of the formations in the minimum time and with the shortest total length of hole. Because of the dip of the rock, it would have been necessary to have vertical holes spaced very closely for complete coverage; these would have been quite deep.

SLIDE 8:

Since horizontal borings have to be started on a solid rock face, it was necessary to begin them above tunnel alignment, where rock could be reached without excessive excavation. Access roads were bulldozed into the flanks of the mountain to reach the locations where the core drilling machines were set up. The north boring, intended to be the longer one, was started about 190 feet above tunnel grade and projected horizontally, that is with the requirement that it stay within a 100 ft. diameter circle over its entire length. The south hole had to be started higher, about 300 feet above tunnel grade, and was projected at a slope of 7 degrees below the horizontal.

The borings were specified to be started NX size with a 2-29/32" O. D. core barrel, giving cores of 2-1/8" diameter. Successive reductions were permitted, if necessary, to BX and AX, giving 1-5/8" and 1-1/8" cores respectively.

First, a short casing or starting barrel was drilled into the rock and cemented in. Then the regular core barrel was set in. This is a steel tube, 10 ft. long, carrying at its forward end the diamond bit and at its rear end a threaded drill rod connection. The hollow drill rods are gripped by the drilling machine and the hole assembly rotated under pressure. Water under pressure enters through the rods into the core barrel and washes the drilling dust away from the bit and out through the hole to the starting barrel. Here it is carried off through a pipe connection. Drill rods are extended in five-foot lengths as the hole advances. Whenever this advance reaches the length of the core barrel, the drill is withdrawn and the core removed from the barrel by unscrewing the bit. As the boring becomes longer, this operation becomes steadily more time-consuming. The bit consists of a short piece of heavy tube, one end of which is threaded to fit the core barrel. The other end is rounded and set with about 100 small diamonds cemented into recesses.

The cores are immediately placed into boxes and a record made of the percent of recovery, the type of rock, and the dip of the seams.

Direction of the bore hole can be controlled to some extent by varying the drill pressure. The direction of the hole was checked about every 150 feet. In the past this had been accomplished for the horizontal direction by inserting a compass in a shell filled with gelatine, which upon congealing held the needle in a fixed position. The vertical direction was established by a line etched into the sides of a glass container partly filled with acid and inserted into the hole. The accuracy of these rather crude devices often was questionable and a rather ingenious new instrument was substituted. This consists of a compass and plumb bob combination mounted in gimbals which allow both to move freely. A clock mechanism can be set for a time delay up to an hour, after which both compass and plumb bob are locked in the position they have assumed at that moment. The device was inserted into a 2" diameter brass tube with watertight plugs and screwed onto the core barrel in place of the bit. The clock was set for the time it would take to advance the head to the proper depth of the hole where it was left until the time was up. Upon withdrawal, the compass declination and plumb bob inclination could be read directly on graduated scales.

The north hole was advanced successfully a total of 1980 feet, with practically no vertical deviation and a horizontal deviation of 46 feet. The south hole took a dip somewhat larger than the intended 7 degrees below the horizontal and deviated transversely about the same amount as the north hole. As the two overlapped by about 90 feet, they provided a complete record of all rock formations inside the mountain.

<u>SLIDES 9, 10, 11, 12</u>

In the portal areas, below the longitudinal borings, a series of vertical holes were used to complete the information. These were partly rock borings and partly cased holes through the loose overburden.

SLIDES 13, 14, 15

It is interesting to note that, for about 600 feet at the south end, these borings indicated difficult rock formations, mainly badly seamed shale. They also gave warning that between these shales and the quartzite there would be a short stretch, a seam of maybe 20 to 30 feet, of badly disintegrated rock, probably heavily water-bearing. Construction has not yet reached this spot but all precautions have been taken to cope with the problem.

The north heading has been advanced about 2000 feet with rock conforming to that indicated by the boring, although requiring more steel supports than had been anticipated.

In conclusion, I would like to state that in both of these projects the results have more than justified the expense of the rather extensive boring programs. I believe the importance of thorough soil investigations for any construction project, whether highway, bridge, or other structure, cannot be stressed too much. It will pay off in the quality of bids received and avoidance of arguments with contractors.

SOME ASPECTS OF AERIAL PHOTOGRAPHS

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The purpose of this paper is to acquaint the reader with the general topic of aerial photographs, some of their physical characteristics, the technique of identifying soil types from airphotos, and some of the recent development in the engineering application of these techniques.

It is interesting to note that aerial photographs, in themselves, are not new. As a matter of record, the first successful aerial photograph was recorded as early as 1858. This first airphoto was made from a balloon at a height of 262 feet and depicted a suburb of Paris, France. Four years later, in 1862, airphotos were taken from a balloon near Richmond, Virginia, and actual planimetric maps were prepared from these photos. It remained for the invention of the airplane and the military applications of World War I to provide the first real impetus to the development of improved aerial cameras, films, and processing techniques. Since that time literally millions of square miles of the earth's surface have been photographed. The United States Air Force alone has photographed over one-half of the earth's land area, 28,000,000 square miles, in black and white film. By the end of 1954, more than 95%, or better than 3,000,000 square miles of the United States had been photographed. The vast majority of this area was photographed for the Department of Agriculture under their crop measuring program which began in the late 1930's. According to the latest information available about 75% of the United States has been photographed more than once. At the present time, the largest area of the United States which has not been photographed consists of the western desert region. This desolate, non-populated area has, thus far, presented no economic justification for aerial photographing.

Coming closer to home, by December, 1954, the state of North Carolina had been completely photographed with the exception of a very few miles in the extreme western portion of the state along the Tennessee border. Specifically, this small, unphotographed area is located in the northern part of Swain County. More than 90% of the state has been re-photographed at least once because of changes in land culture or other changes on the ground.

The governmental agencies under whose auspices these photographs were taken include the Production and Marketing Administration and the Soil Conservation Service of the Department of Agriculture, the U. S. Geological Survey, Tennessee Valley Authority, U. S. Air

Force, U. S. Coast and Geodetic Survey, and the U. S. Army Corps of Engineers. Other agencies with important photographic holdings in the United States include the U. S. Forest Service, the U. S. Bureau of Reclamation, various state agencies, and some commercial organizations. The photographs held by these agencies are generally readily available upon request.

One might easily wonder, "how much area does each aerial photograph cover and how much do they cost?" In answer to the first part of this question, the area covered by each airphoto depends, of course, upon its scale. The scale of the airphoto, in turn, is dependent upon the altitude at which it was taken and the focal length of the aerial camera. The majority of the Department of Agriculture airphotos of North Carolina are 7" x 9" or 9" by 9" contact size and are at a scale of about 1:20,000 which is about 3" to 1 mile. This means that each 9" x 9" photograph covers approximately 9 square miles. The cost of aerial photographs varies from 50 cents to $1.00 per print depending upon the type of paper specified and on the number of prints purchased. As an example of the cost of airphoto coverage of a large area; for a county 20 miles wide and 20 miles long it would require about 300 airphotos each being 9 x 9 and having a scale of 3" equals 1 mile to cover the entire county. The cost of 300 airphotos would be approximately $150.00. Because of the amount of overlap between each photograph along the line of flight and between each flight line, complete coverage could be obtained by ordering only alternate prints. To permit the use of stereoscopes to envision the third dimension or relief of the area, however, it is necessary to purchase every photograph rather than alternates. The cost figure of $150.00 given above is based on the purchase of every photograph along the flight line. It has been estimated that stereoscopic coverage for an area the size of the State of Indiana would require the purchase of about 25,000 contact prints at a cost of about $12,000. This figure is small indeed when one realizes the many and varied uses of airphotos and the voluminous amount of detail engineering information obtainable from them.

With this brief introduction to the airphoto let us now examine its potential uses. It may be stated that the uses of aerial photographs have become so many and so varied that it is difficult to pick any one of their functions as being the most important. A few of the more promising fields, other than military, in which airphotos serve as a valuable tool include: city, county and regional planning; United States Department of Agriculture Surveys, Soil Conservation planning programs; timber surveys, wild life surveys; preliminary geological and soil reconnaissance; and highway and airport engineering. Under the one item of city, county and regional planning the uses of airphotos and maps prepared from them might be outlined as follows: (1) preparation of zoning maps, (2) location of property lines, (3) preparation of land use maps, (4) planning of emergency housing, (5) planning of long-range redevelopment projects, (6) simultaneous traffic counts and traffic flow studies, and (7) preparation of tax and assessment maps. In regards to the latter type of map prepared from aerial photographs, one county in Pennsylvania reported "A reasonable estimate, based on the experience inaugurated in the county, and on our experience in the handling of properties acquired at tax sales, and our past two months' experience with the air photographs, is that we will discover at least 2,000 properties that are not paying taxes at the present time (as compared with 25,000 which are assessed)."

The location of highways and airports is a problem of major importance and new uses for aerial photographs in this field are frequently being discovered. Mr. E. T. Gawkins, Deputy Chief Engineer, New York State Department of Public Works, has this to say: "We have only begun to utilize this modern tool in the design of our highways. It is reasonable to expect that its effectiveness will increase to a degree that we dare not now predict."

The state of Massachusetts reports "- - - large-scale up-to-date aerial photographs of proposed highway locations are of value in two ways: (1) to help insure selection of the optimum route, and (2) to reduce the cost of engineering."

The Massachusetts report also lists seven note-worthy advantages of aerial photographs in their highway planning. Briefly these are as follows:

(1) Location of granular material can be determined from aerial photos.
(2) Presents a complete inventory of surface features.
(3) Much easier to read than maps.
(4) Scale of air photos is three to six times larger than existing maps.
(5) Before and after pictures of highway construction are valuable exhibits in damage suits.
(6) Property acquisition is simplified and expedited when aerial photos are available.
(7) Cost of location surveys is reduced.

New Jersey has outlined the advantage of aerial photographs and surveys in highway work as follows:

(1) Can become an indispensable adjunct in the preparation of soils data.
(2) An aid in the study of alternative alignments as well as in selecting a preliminary alignment for new routes.
(3) Can be profitably employed in the design of such projects as underpasses, overpasses, interchanges and traffic separations.
(4) Can be utilized in laying out the alignment of new parkways.
(5) Valuable in locating and determining the nature and limits of landscaped borders on proposed parkways.
(6) Clearly defines property lines.
(7) Possesses a very real value for display purposes.
(8) Disclose where new routes serving to relieve traffic may also remove blighted areas.

TABLE I

HIGHWAY LANDSLIDE PROBLEM

Phase of Study	Use of Geologic Principles	Use of Engineering Principles	Primary Influence
1. Preliminary Investigation			Combined
a. Office study	X	X	
b. Preliminary reconnaissance	X	X	
c. Planning of field study	X	x	
2. Field Investigation			
a. Surface survey	x	X	Engineering
b. Sub-surface studies	X	x	Geologic
(1) Auger drill	X	x	
(2) Core drill	X	x	
(3) Geophysics	X	x	
3. Analysis			
a. Assimilation of field data	X	X	Combined
b. Laboratory testing	x	X	Engineering
c. Theory of landslide development	X	x	Geologic
(1) History of the area	X	x	
(2) Existing conditions	X	x	
(3) Contributing factors or causes	X	X	
(4) Probable stability of the area	X	X	
d. Method of treatment	x	X	Engineering
(1) Type of movement	X	x	
(2) Motivating forces	x	X	
(3) Resisting forces	x	X	
4. Solution			
a. Empirical	X	X	Combined
(1) Degree of certainty based on experience	X	X	
(2) Cost estimates	x	X	
b. Analytical	x	X	Engineering
(1) Stability analyses for relative stability	x	X	
(2) Cost estimates	x	X	
c. Degree of certainty vs, cost relationships	x	X	Engineering

X - Major x - Minor

In addition to these many uses of aerial photographs in highway engineering, the strip-mapping of highways at low altitude results in an accurate and permanent record of highway performance characteristics.

Closely allied to the problem of highway engineering are the problems confronting the airport engineer. Many of the uses of airphotos discussed in the light of highway engineering apply equally as well to airport location and design. Naturally, a realization from airphotos of the general engineering characteristics of the soils present, the existing drainage, the accessability, and topography of a proposed airport site are of immeasurable value to the airport engineer.

One of the primary advantages of using aerial photographs for highway engineering purposes lies in the fact that detailed soil maps and detailed drainage maps cannot be made by any other existing method at a comparable cost or made as efficiently from the standpoint of time.

When interpreting and predicting soil types from aerial photographs, the trained observer is interested in what has been called the "natural" or "soil" pattern reflected by surficial markings on the earth. In cultivated regions the first impression of the casual observer is that of a surface covered with a checkerboard field pattern and other man-made features. In a heavily forested area the first impression is of a region carpeted with vegetation and in an arid desert region one is apt to gain the impression of a complete absence of any recognizable features. However, regardless of climate, vegetation, location, or man's influence, to the experienced interpreter who subjects the airphotos to a careful study, there is another pattern distinct from the superficial vegetative pattern. This pattern is that created by the forces of nature. Through detailed study and proper evaluation of this natural pattern, soil and bedrock studies and mapping can be accomplished accurately.

Particular use is made of inference by the airphoto interpreter. No doubt, most of you have observed that the rooftops of certain houses following a heavy snowfall were completely snow-covered, while the snow on the roofs of adjacent houses had completely melted except that on the overhanging eaves. The obvious and correct inference or interpretation is that the houses whose roofs were completely covered with snow had been provided with modern insulation while the houses upon whose roofs the snow had melted were not insulated. A similar, although somewhat more complicated reasoning process when applied to surface features of the earth as depicted by airphotos will yield much information concerning the subsurface conditions which would otherwise be unobtainable from the airphoto.

The factors that the soils interpreter uses in airphoto interpretation are known as "elements" of the soil pattern. These natural elements of the soil pattern compose the visible evidence in aerial photos of certain natural processes acting on the surface of the earth from the origin of a soil deposit to the present time. This sequence of events and the proper evaluation of these events are based on the principles of geology, physiography, geomorphology, pedology, and climatology. Stated in different words, this means that the logical reasoning of how a particular soil deposit originated and the proper evaluation of the effects of natural forces, such as erosion, weathering, and others which act on that deposit, provides a foundation for successful soil predictions and engineering evaluations from airphotos.

The basis for the validity of soil and rock predictions from aerial photographs is that soil and rock patterns are repetitive in nature. This means that any two materials derived from the same soil or rock parent material or deposited in a similar manner, both having undergone the same weathering conditions, under the same climate, and both occupying the same relative topographic position, will have similar profiles, similar engineering properties, support the same native vegetation, and will exhibit the same airphoto soil pattern.

It would appear reasonable that the natural elements of the soil pattern are: land form, regional drainage system, gully systems and cross-sections, erosion features, soil color tones, vegetation, man-made features, and many others of a special nature which result from some inherent quality or characteristic of the material. The element of man-made features is included in a list of natural elements because the dictates of climate, topography, and soils generally govern man's land use practices.

Let us examine briefly a few of the more important elements of the airphoto soil pattern. Generally, the most important pattern element is the land form. This element includes topography, topographic position, general shape, and local relief. Often an area or deposit can be identified on the basis of land form alone. As an example: the crescent shape dune is easily distinguished on the airphoto on the basis of shape or land form. When this particular land form is noted we immediately realize that the soil is a uniformly graded, fine to medium size, clean sand. To the interpreter who understands a few of the basic principles of physical geology, the element land form classifies a deposit and limits the type of materials to be expected.

The element, drainage pattern, is perhaps the second most important element. Again, as in the case of land form, the soil type in a few instances can be identified solely on the basis of the type of drainage pattern in the area. To illustrate: in areas of flat-lying soluble limestone bedrock, the drainage is confined to "sink" holes through which surface drainage waters pass on their way to underground drainageways. Sink holes occur only in this type of material; thus, we are able to identify the material on the basis of the drainage system. In the general case, however, the drainage pattern is used as a means of eliminating certain materials rather than as a means of positive identification.

Soil textural properties and soil profile development can often be interpreted from the gully system and the individual gully cross-sections. Deep uniform soils are reflected on the airphoto by uniform gully gradients, cross-sections, and regularity of gully systems. As a general rule, the finer the soil texture the flatter and softer are the gully side slopes and shoulders.

The soil color tone is another of the more important soil pattern elements. Soil color tones should not be confused with vegetative color tones. In general, light-textured and well drained soils photograph light in color and conversely, heavy-textured, poorly drained soils photograph medium gray to black.

At present time we are equipped at North Carolina State College to predict and evaluate engineering soils from aerial photographs and to prepare maps depicting general engineering soils, locations of suitable borrow materials, alternative alignments of highways, and other features of interest to the highway and airport engineer. We are also equipped to produce detailed surface drainage maps.

It is our sincere wish to be of assistance to interested engineering and planning agencies in this state and it is also to be hoped that they will take full advantage of these services.

REGISTRATION FOR THE SEVENTH ANNUAL SYMPOSIUM

GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING

Feb. 24, 1956

North Carolina State College, Raleigh, N. C.

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8TH ANNUAL GEOLOGY SYMPOSIUM

Friday, February 15, 1957

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PROGRAM

NEW DEVELOPMENTS IN THE STUDY OF LANDSLIDES

Edwin B. Eckel, Chief
Engineering Geology Branch
United States Geological Survey

GEOLOGICAL INFORMATION IN PENNSYLVANIA FOR THE HIGHWAY ENGINEER

Dr. Carlyle Gray
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EXPERIENCE IN DESIGNING ROCK SLOPES IN NEW YORK STATE

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PHOTOGRAMMETRY IN PRACTICE

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NEW DEVELOPMENTS IN THE STUDY OF LANDSLIDES

by Edwin B. Eckel*
Chief, Engineering Geology Branch
U. S. Geological Survey
Denver, Colorado

The engineer and the gologist who works with him are interested in landslides because their job is to build and maintain safe, economical, and useful structures on the earth's surface. A landslide, unforeseen or improperly provided for, may destroy their structure or impair its usefulness. Such a landslide may mean death to people who have trusted the structure; repair of the structure will most certainly cost money. Even an insignificant little slide, sloughing off into a roadside ditch, may wreck an automobile; or if it is continuous, it may in time run up enormous maintenance costs.

Like any other branch of the pursuit of knowledge that we call science, the study of landslides is a continuing operation. What we know about them today is the sum of facts that have been gathered in the past; what we are investigating about them now is based on, and an extension of, what we already know.

To present the state of knowledge of landslides adequately, or even to discuss all of the current investigations on them, is a far bigger job than can be accomplished in a brief talk. Instead, I intend to summarize the facts on these subjects as briefly as I can, and to do it largely by abstracting a single book. Later I will mention a few specific examples of noteworthy landslide investigations.

Aware of the great importance of landslides to the highway engineer, the Highway Research Board in 1950 set up a Committee on Landslide Investigations, giving it a very broad charter. The committee membership was intentionally drawn from a wide geographic range, it included soils engineers and geologists in about equal proportions, and it included representatives of state highway, governmental, and educational institutions. Several of its members are here today.

The first work of this committee was to sponsor compilation and publication of a bibliography on landslides, published in 1951 by the Highway Research Board. Then, little realizing what it was in for, the committee decided to compile a reference volume that would attempt to bring together all that the common or garden highway engineer should know about landslides. After many trials and tribulations, and after a questionnaire that troubled many of you, I'm sure - and to the great suprise of many of us - the manuscript of that book is now complete. Called "Landslides and Engineering Practice," it is now ready for transmittal to the Highway Research Board and should be published in the near future.

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

Perhaps it is unfair to ask you to take your time in listenting to the prepublican advertisement of a book. But, this book covers virtually the same subject as that of my talk. It attempts to summarize for the first time in English, the present state of knowledge on landslides, with particular reference to the engineer's problems in controlling or correcting them.

The first part of Landslides and Engineering Practice, called Definition of the Problem, starts off with an introduction that tells how and why the book was put together. This is followed by a chapter that brings together what is known of the costs of landslides and of the laws and court decisions that bear on landslides and the damage caused by them.

The average yearly cost of landslides in the continental United States runs to hundreds of millions of dollars. This money, paid out every year by taxpayers and private companies, includes not only the direct costs of corrections and repairs of damage caused by landslides, but also very large sums for such items as delays of traffic, interruptions of service and claims for damages.

Highways, railroads, and public utilities as a group receive the greatest direct damages, but even here, full costs can be assessed only in individual cases. Heavy losses are also sustained by local governments and homeowners in some cities. In parts of the country very severe losses are involved in destruction of farmlands, resorts, and homes, particularly along rivers and lakes where undercutting and slipping occurs. The damages in these categories are extremely difficult to evaluate, but they are obviously large.

Among the highway departments, one state reported annual costs of more than $1,000,000, three states between $500,000 and $1,000,000, one state $250,000 to $500,000, five states $100,000 to $250,000, six states $25,000 to $100,000 and eleven states less than $25,000. These figures apply very largely to maintenance costs, because costs of reconstruction and damage claims are not usually accessible. In fact, the figures reported are probably low even for maintenance costs, because many highway department accounting methods are not such as to fully disclose maintenance costs that are directly related to landslide problems.

Few legal precedents have been established to guide the courts in determining responsibility for landslides or in assessing the damages caused by them. This dearth of specific laws and legal decisions is perhaps due to two main factors - many, if not most, cases that involve private companies are settled out of court; most cases against State or Federal Agencies are settled out of court or the public agency exercises its sovereign right of refusal to consent to be sued.

The scattered and somewhat conflicting facts as to actual court decisions as well as the practices of various state highway departments are summarized in the chapter. In addition, it contains brief discussions as to the legal responsibilities of the engineer who works on reports on a slide, on the legal meaning of warning signs, and on zoning laws of some cities that are specially plagued by slides.

The next chapter of the book is called Landslide Types and Processes. It is essential to an understanding of the whole subject, for unless we understand the origin of a slide we hardly know where to begin in planning methods of dealing with it. Perhaps the most significant contribution in this chapter is a new classification of landslides (in chart form) that is, we believe, a considerable improvement over older classifications. This chart, several copies of which are on display, classifies slides primarily as to type of movement - falls, slides, and flows - and then as to size and type of material that is involved - whether bedrock or soils, and whether coarse or fine grained. Differences in water content in the slide material are also taken into account.

The classification is as logical as it is possible to make it; with the descriptions and sketches that it contains we believe that identification of any natural slide with one of the types on the chart should be relatively easy.

This chapter also outlines the principal factors that cause landslides. These fall into two main categories - those that contribute to high shear stress and those that contribute to low shear strength. These two factors appear again and again as the principal and strongest threads involved in weaving together the story of any landslide problem.

The next chapter sums up the methods and techniques that are used in the recognition and identification of landslides. It makes sharp distinctions between the evidence for actual slides that have already taken place and potential slides - those that may or will take place in the future, depending on future construction and other factors. The criteria used for identifying landslide types, and for fitting specific slides into the classification scheme, are developed rather fully.

Primarily because it represents one of the most modern and promising developments in the study of slides, the subject of airphoto interpretations is reserved for a separate chapter. Aided by a series of stereopairs of airphotos, its author describes the evidence for actual or potential slides in various types of terrain in terms that can be applied by almost anyone who is blessed with stereovision. We do not pretend that all landslide problems can be solved by study of airphotos, but we do believe that we have here a new tool, which in conjunction with other methods, can aid greatly in understanding slides and in designing plans to combat them.

Next comes a chapter on field and laboratory investigations that are needed to collect the data that must be used in reaching decisions as to the kind of engineering treatment required. It describes the topographic, geologic and soils maps that may be needed, the subsurface investigations, and the soil, mineralogic, and weathering tests that may be made in the laboratory. Above all, it makes distinctions between the scope of investigations that are needed to solve a small slide problem and those for a major slide, one that may cost many thousand dollars for correction and hence deserves a thorough investigation, even though that is very expensive.

The second half of the book, which we call "Solution of the Problem," consists of three main chapters plus a brief look into the future. Two of these contain the meat of the entire subject - prevention and correction of landslides. The third is on stability analyses and their use.

All the methods known to the committee to be useful in prevention or correction are described. So far as possible they are evaluated carefully and are related to the landslide types for which they are most applicable.

Of the many methods for prevention or correction of slides, the most certain one is avoidance - to go around or over the slide area. Economic or other reasons may make this method infeasible. All other available methods must treat one or both of the two principal causes of slides. That is, they must seek to decrease the shear stress in the materials or to increase their shear resistance. Some methods, like subdrainage, accomplish both objectives at the same time. Others such as partial or complete removal of material, tackle the problem by decreasing the shear stress. Still others, such as restraining structures of all kinds, do it by increasing the shear resistance of the materials.

Nearly all slides can be corrected or prevented by one means or another. The most effective means is the one that takes full account of the nature, history and geologic causes of the slide. Beyond this, the choice of method and its design are almost entirely questions of economics.

The chapter on stability analyses and the design of control structures presents the mathematical and graphic methods that can be used in analyzing the stability of a landslide mass, in designing specific corrective measures, and in comparing the safety factors that should be produced by different corrective methods. Perhaps its most unique feature is that the reader is led step by step through the entire set of computations instead of having to be content with a set of formulas and the summaries of their solutions.

Naturally I'm prejudiced, but I believe that the book just described is one of the more significant new developments in the study of landslides - at least so far as the highway engineer is concerned. Another recent development that has at least as much promise is the formation of a Subcommittee on Canadian Landslides under the auspices of the National Research Council of Canada. Unlike its American counterpart - and doubtless because it is smarter - this committee is not writing a book. Instead, it fosters research on specific landslide problems, by searching out the problems most in need of study, by delimiting such problems and by stimulating actual studies of them by the most qualified individuals available.

I have neither time nor knowledge to mention all other current research on landslides here. As samples of what is going on, however, I would like to describe a few projects with which I am acquainted.

The Swedish and Norwegian geologists and soils men are continuing their fundamental research in the study of "quick" marine clays - those thixotropic materials that suddenly turn from solid to liquid, often with disastrous results. Faced with many serious slides of the same type in the St. Lawrence Valley, the Canadians are also doing much work on quick clays. There are still differences of opinion, but the causes of slides in quick clays seem to be fairly well worked out. Almost wholly untouched, however, are answers to the problems of prediction and prevention of slides in the marine quick clays.

Several Japanese soils technologists are doing excellent work on landslides in their country. It is to be hoped that their promising results on prediction, and on correlation of laboratory shear and other tests with field behavior, will become more easily available to us who can read only English.

The U. S. Geological Survey has no comprehensive program of investigating landslides, but much of its work deals directly with them or produces basic scientific data that can be used in understanding them. Many of the Survey's geologic maps delineate landslide areas in greater or less degree, for instance. Some of them differentiate between active and inactive slide areas and some Geological Survey maps and reports contain detailed descriptions of the slides and their causes. Much fundamental work is being done, also, on identification of the clay minerals, on the base exchange properties of clays, and on the physical, petrologic and chemical characteristics of geologic materials that are subject to sliding in many parts of the country.

In addition to these general kinds of information that are gathered by the Geological Survey, we have a number of projects aimed at studies of specific slides or of specific slide problems. Thus, we have recently completed a long study of the landslides along the shores of Lake Roosevelt, behind Grand Coulee Dam. The sands, silts and clays along the Columbia River Valley have apparently been sliding ever since they were deposited in lakes or by glacial streams. The backing up of water by Grand Coulee Dam, however, and the rapid changes in lake level caused by drawdowns for power purposes, have caused hundreds of slides, some of them enormous, to develop during the past dozen years. Because the slides were so numerous, and were all rather similar in geology and environment, it was possible to study them by the methods of statistical analysis. This is the first time, we believe, that statistical analysis has been applied to landslide investigations. The results are encouraging. Whether these methods can be applied to other landslides in other environments remains to be seen.

Another Geological Survey project now in progress has to do with the effect of a rising water table on slides in a relatively watertight shale. The lake behind the Fort Randall Dam on the Missouri River is gradually covering, and saturating, a series of shales that are noted for their landslide propensities. By means of detailed geologic mapping and by a series of special piezometers set in drill holes at varying distances from the lake shore we hope to correlate the incidence of future slides with rise or fluctuations in ground water levels. This is necessarily a long-continued study; if its results are conclusive we will have a new tool for predicting slides in similar environments elsewhere.

Still another Geological Survey project that bears mention is that along the Pacific Palisades, in western Los Angeles. Because this is a highly developed residential area, even a small slide there can prove expensive - the fact is that many of the slides there are enormous, as is the expense in loss of roads, utilities, and palatial homes. To make the future seem even darker, Los Angeles has had few really bad rainy seasons since the post-war building boom reached the Pacific shores. When the rains do come, landslide troubles will be multiplied many fold. There are no readily apparent or economical means of preventing slides there, and few of correcting them. Our efforts are chiefly aimed, therefore, at delineating actual and potential slides so accurately that the results can be used in zoning the land, and in other engineering decisions of similar nature. For self protection, as well as to provide important basic data on the cause and character of the slides, the geologist in charge is basing his maps of actual slide areas on well-documented evidence with exact dates of movement and with the outlines of each successive slide plotted accurately on large scale maps.

Now, a few words on what we still need to know about landslides. We have a workable classification of landslides; the processes that cause them are well understood; we know dozens of methods for preventing or correcting slides; we know, at least in general terms, where slides are most likely to occur, in what parts of the country, and in what kinds of rocks and soils.

What do we not know? It seems to me to boil down to this - we need to know much more about when and where the next slide is going to occur and we need to know more exactly what treatment to apply to a given slide, potential or actual.

To answer these needs is not a simple job, but it can be done if enough engineers and geologists realize the economic importance of the landslide problem, make opportunities to study it, and record their results.

We need many, many more studies of actual slides - not simply descriptions of their size and shape and the damage they did. To get much farther with our understanding of slides these descriptions must include detailed records of the physical properties of the soils or rocks that were involved, detailed histories of the movements that took place, and enlightened inquiries into the causes of movement. With this kind of facts we can go a long way in comparing slides and in extrapolating the knowledge thus gained to the solution of new slide problems.

Finally, we need to record many more of our failures as engineers or geologists. The literature is full of descriptions of preventive or corrective methods that were successful. On the other hand, records of installations that failed to do the job expected of them are notably lacking. Nobody enjoys publicizing his mistakes, of course, but just the same there is little hope of improving our knowledge of how to handle landslides until we are able to study and compare the case histories of methods that have failed with those that have been successful.

GEOLOGICAL INFORMATION IN PENNSYLVANIA FOR THE HIGHWAY ENGINEERS

By Carlyle Gray

INTRODUCTION

When first asked to give a paper at this symposium, the author's reaction was that he had so little experience with the application of geology to highway engineering that he had nothing to contribute. On second though, however, it appeared that a useful purpose might be served by a discussion of the sources of geological information. Persons not working regularly in the field of geology are generally not familiar with the various types of geologic reports and the various ways in which they get published. This paper will review some of the categories of geologic reports and attempt to indicate both their availability and possible usefulness to the highway engineer. Particular attention is given to Pennsylvania, but much of what is said would hold true in other states as well.

GEOLOGICAL REPORTS OF USE TO THE ENGINEER

General Statement

Geological reports come in many forms, shapes, and sizes. There are four principal categories which will be of particular interest to you as engineers. They are:

1) Areal geologic reports and maps, usually quadrangle or county reports.
2) Stratigraphic reports, which deal with rock formations or groups of formations over some specified area, usually presented with only general geological maps as illustrations.
3) Economic, or mineral resource reports, devoted to detailed study of a mineral deposit, or regional study of the occurrence of a particular resource.
4) Groundwater reports, geology and hydrology or groundwater occurrence.
5) Miscellaneous subsurface information. Well logs, cutting descriptions, subsurface cross sections.

Areal Geologic Reports

Areal reports are general geologic reports of a given area, intended to serve the needs of as many different users as possible. Geographic units covered are commonly counties or 15-minute quadrangles. Since 1900 most or all geologic reports have contained maps prepared on a topographic base. The 15-minute quadrangle, (15' latitude by 15' longitude) topographic map series of the United States Geological Survey has therefore frequently served as the unit of area mapped. These maps are therefore usually on the 1:62, 500 scale; that is, about one inch equals one mile. In recent years, topographic maps on 1:24, 000 scale (one inch equals 2,000 feet) have become available, and the near future will bring publications of areal geologic maps on this scale.

A geologic map shows the distribution of rock formations. The usefulness of the map to the engineer will depend on his understanding of what the formations are. A formation is defined as a mappable unit; that is, a rock mass that has characteristics which can be recognized in the field as distinguishing it from adjacent units. The formations mapped in Pennsylvania are usually bedrock units, but a few maps show surficial geology in glaciated areas. It is possible to have both surficial and bedrock geological maps for a single area. From the engineering point of view, a formation may comprise quite a variety of rock types, or on the other hand, a series of several formations may be quite homogeneous as far as the needs of a particular project are concerned. For example, the Allegheny formation consists of a variety of rocks including beds of coal, shale, clay, limestone, and sandstone, while the Tomstown, Elbrook, and Conococheauge formations of the Chambersburg area are all limestones and dolomites, practically indistinguishable to any but the most practiced eye. A good areal report will therefore have a text accompanying the map which describes the formation units in detail. Through study of the text, the engineer can learn which of the formations will require further breakdown, and which can be lumped together.

In other words, a geologic map is an abstraction of nature that must be interpreted by each user to suit his own needs. As an illustration and guide to the engineering information which can be obtained from an areal geologic report, the United States Geological Survey has prepared a folio entitled "Interpreting Geological Maps for Engineering Purposes." Fortunately, from our point of view, an area in Pennsylvania was chosen to serve as the example; the Hollidaysburg Quadrangle, from the Huntingdon-Hollidaysburg Folio (Butts, 1946). With only limited field check, the following were prepared from the published map and text:

1) Map showing foundation and excavation conditions throughout the area.
2) Map showing distribution of construction materials.
3) Map of groundwater supply conditions.

In addition, a sheet was prepared giving examples of engineering problems and how the maps were used in the solution of the problems. The report is well worth a careful study by anyone active in engineering geology.

In Pennsylvania there are only two important sources of areal geologic maps: the publications of the State Geological Survey and those of the United States Geological Survey. The State Geological Survey issues a list of publications and an index map showing the location of all areas (counties and quadrangles) covered by geologic maps and reports. All "in print" publications are shown, as well as most out-of-print reports published after 1900. This list is a good starting point or running down geological information on given area.

The Pennsylvania State Geological Survey divides its publications into several types, depending on content. The geologic atlas, or A-series bulletins are quadrangle geologic reports with maps on a topographic base. To date all maps are at the one-inch-equals-approximately-one mile scale, though some reports now in preparation will contain larger scale maps. These bulletins also contain complete written description of the geology and mineral resources of the area.

The C-series bulletins are County reports. These usually contain a 1-inch-to-1-mile geologic map of the entire county and a full written text. In some cases, the geology is a compilation and revision of previous quadrangle mapping, and in others a county was mapped as a unit from scratch.

The PR, or Progress Report-series, contains some preliminary geologic maps, printed in just one or two colors, accompanied by brief expalatory text. In this way preliminary data can be made available several years before completion of the detailed report.

The United States Geological Survey publishes quadrangle geologic reports as Bulletins, Professional Papers, Folios, and in recent years as Geologic Quadrangles. The Bulletin and Professional Papers reports are very similar to the A-series reports of the State Geological Survey. The Folio series, now discontinued, consisted of a large, folio size, publication, with several maps and several large pages of explanatory text. Many of these old folios are fine publications with various special purpose maps in addition to general geologic maps. The folios have been replaced by the Geologic Quadrangle series which consist of a single, colored geologic map faced by a sheet of text, similar in content to the text accompanying the Pennsylvania Survey preliminary maps.

Modern Quadrangel and County reports have been published for about one-third of the area of the State. They are the most important source of geologic maps. Other sources of geologic maps include:

1) Reports of the Second Geological Survey of Pennsylvania. The Second Survey prepared county geologic maps for every county in Pennsylvania. Most of the maps are also supported by a text. None of the maps are on topographic base maps and all are on scale smaller than 1:62,500. The mapping was all done between 1874-1895, a remarkably short time in which to cover the entire State. The formation units are therefore usually rather thick and as a result the maps are more generalized than are modern maps. In much of the State, these maps are still the best available, and they are quite useful. They contain remarkably few errors that are not the necessary result of lack of time to study details.

2) Economic reports and groundwater reports sometimes include areal geologic maps, or structural maps from which geologic maps can be derived. These are discussed in more detail below.

3) Unpublished maps particularly student theses, and maps on open file with the state or Federal Geological Survey. The

Geology Department of Pennsylvania State University particularly, and other colleges such as University of Pittsburgh, and Lehigh University, have on file numerous unpublished theses. Some of these contain manuscript maps, usually of small areas. These maps are usually of limited usefulness. Nearly all government surveys have a form of limited publication known as placing on open file. This means that s report which is not to be printed, is made available for inspection at a Survey office. These reports are usually reports of very limited interest and application, or reports which because of lack of printing funds will not be printed in the foreseeable future. When a report is placed on open file, an attempt is made to notify the public and potentially interested persons. Information on the availability of older reports can be obtained by writing to the Geologic Survey in question.

Out-of-print reports can sometimes be bought from second-hand book dealers. More often they must be obtained from libraries. Reports of the Pennsylvania Geologic Survey are distributed to about 150 public and college libraries throughout the State, and to nearly 200 out-of-state libraries. The United States Geological Survey also has a number of libraries in the State which are repositories for all of their publications.

Stratigraphic Reports

A stratigraphic report is a specialized report giving particular attention to the composition, nature, and origin of a rock unit or group of rock units. It usually is regional in scope. It may cover a system, such as the "Devonian of Pennsylvania," or any other convenient category of formations. The reports seldom contain detailed maps, but usually contain regional maps that can be very useful. Stratigraphic reports are of particular use to the engineer as supplements to the information given in areal reports. If the text of the areal report is full and detailed, it will contain essentially the same information as a stratigraphic report. If the text is not detailed, a stratigraphic report will be very useful in interpreting the map. For example, it is sometimes possible to increase considerably the detail of information gained by interpreting an old Second Geological Survey map through the use of a modern stratigraphic report.

Of particular use to the engineer, when available, are the reports containing detailed descriptions of measured sections. These are bed by bed descriptions of lithology (and fossil content) which could be useful in making preliminary studies for tunnels or rock cuts.

The sources of stratigraphic reports are similar to areal reports, with the addition of the geological journals as an important source.

The Pennsylvania Geological Survey publishes most of its stratigraphic reports as G-series bulletins, an example is Bulletin G-19, "Devonian of Pennsylvania." Many of the economic reports dealing with petroleum are essentially stratigraphic reports, although the emphasis is on sub-surface stratigraphy and the usefulness to the highway engineer may be limited. Preliminary results of stratigraphic studies are to be found in the PR-series. Stratigraphic publications of the U. S. Geological Survey appear as Bulletins and Professional Papers.

The geological journals are important sources of all kinds of geologic reports except areal maps. (For areas outside of Pennsylvania, particularly New England, areal reports have been published in journals) The journals are published by the various scientific societies and universities. Those most apt to contain articles on Pennsylvania include: Bulletins of the Geological Society of America, Bulletins of the American Association of Petroleum Geologists, Journal of Geology, American Journal of Science, Proceedings of the Pennsylvania Academy of Science, and Economic Geology.

Finding out whether or not an article has been published on a particular area of subject can be an arduous task. Fortunately, a number of aids are available in the form of published bibliographies. Foremost is the Bibliography of North American Geology, published by the United States Geological Survey. This is published as a number of volumes, each covering a year, or span of years. A number of State Surveys have published bibliographies dealing with the geology of their state. New Jersey and Virginia both have published useful bibliographies. In Pennsylvania, we are now in the process of compiling such a volume. If a bibliography is not available, do not forget that your State Survey will be glad to help you find the information you need. We have in our library a card file which serves as a very complete source of references.

Once a reference citation is found, the problem is to locate a library with the journal on file. Libraries of all colleges and universities that have geology departments should have most or all on file. The Pennsylvania Geological Survey library, and the State Library in Harrisburg have them. However, only large public libraries are apt to have them.

Economic or Mineral Resource Reports

Reports dealing with mineral deposits can be either detailed local descriptions of a mineral occurrence, or a regional description. They are of direct value to the engineer when dealing with such problems as land evaluation, or supply of construction materials. They also are of indirect value as partial descriptions of the geology of the area of mineral occurrence. For example, oil and gas reports, while concentrating on subsurface information, usually contain a structure contour map of a near surface horizon, or in some cases, even a surface geologic map.

Sources of mineral resource reports are again the State and Federal Surveys, and Geological Journals. In addition, reports of the United States Bureau of Mines, and the College of Mineral Industries bulletins are valuable sources of economic geology reports.

The Pennsylvania Geological Survey publishes mineral resource reports as M-series bulletins, and as Progress Reports. Examples are Bulletin M 20, "Limestones of Pennsylvania," and PR 149, "The Occurrence of Rock Salt in Pennsylvania."

In addition to Bulletins and Professional Papers, the U. S. Geological Survey issues mineral information in Circulars, Oil and Gas Investigation Charts, and Coal Investigation Charts.

The U. S. Bureau of Mines publishes mineral resource information as Reports of Investigations. The Bureau investigates, samples, and frequently drills deposits of strategic and critical minerals in connection with the Defense Minerals Program. The results are published in brief form, with a minimum of interpretation. The data, however, may occasionally be of value in problems of land evaluation.

The publications of the Mineral Experiment Station of the College of Mineral Industries here at Pennsylvania State University are quite varied in their subject matter. Many of them, however, are mineral resource reports and stratigraphic reports and are of the same type of usefulnes as the reports of the State and Federal Surveys.

Ground Water Reports

Ground water reports describe the occurrence, availability, and geology of ground water. They are of course of particular value to the engineer when trying to locate a water supply. They should be useful also when information concerning the water-bearing characteristics of rocks and rock units is needed for design and construction of tunnels and rock cuts, or in connection with the solution of landslide problems. As mentioned before, Ground Water Reports also frequently contain geologic maps not available in any other way.

The Pennsylvania Geological Survey conducts its groundwater studies cooperatively with the United States Geological Survey. The results are published (with minor exceptions) entirely by the State. Water reports are designated W-series bulletins. Six of these bulletins are a unified series of reports, each covering one-sixth of the State. The information given is an excellent general survey of the State's groundwater resources. The data is largely of a qualitative nature, and the approach is necessarily that of a reconnaissance. Continuing investigations are being made of the quantitative aspects of ground water.

Subsurface Information

Miscellaneous subsurface information is available which may occasionally be of use. The Pennsylvania Geological Survey publishes various types of well logs in its regular oil and gas studies, and also well sample descriptions from deep wells as separate pamphlets. These can be considered as supplementary stratigraphic information. On open file at the Geologic Survey offices are fairly extensive files of drillers' logs of oil and gas wells. These usually have rather sketchy or no geological information on the near-surface horizon, although some indicate coal beds and thicknesses.

Summary

Geological information of many kinds useful to the highway engineer is available if he knows where to look. The publications of the State and Federal Surveys are the most readily obtainable. Additional information can be found in articles published in geological and scientific journals. Unpublished material can be found in university libraries in the form of student theses, and on open file at the government surveys. Do not forget that your State and Federal Survey personnel will always be glad to help you find the information you need.

BRIDGE FOUNDATION EXPERIENCES

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The science of geology has been of assistance in the design of bridge and structure foundations, particularly in describing the characteristics of certain rocks and soils which are encountered, and in explaining why they act as they do under load. We understand, by the way, that in the science of geology "soil" is also classified under the general term "rock"; and if we should continue to employ the term "soil" in describing certain materials, it will be because of the fact that the relatively new science of "soil mechanics" aplies particularly to sand, silt and clay materials which compress or consolidate noticeably under bridge loads. Since many of our large bridges have recently been founded in such un-rocklike materials, we are inclined to use the word "soil" in its general sense where it applies.

We expect to describe in this paper a series of foundations which progress from actual rock, through decomposed rock to silts and clays which have been formed by varying geological histories. Since we are engineers rather than geologists, we have necessarily had to consult with others to obtain these geological histories, and we will usually note in this paper the source of such geologic information.

Rock Bedding at Lehigh Gap

In the Lehigh Gap area of Pennsylvania, in the general area of Palmerton, we are currently designing a four-lane roadway to replace present two-lane Pennsylvania Traffic Route 29. Where Blue Mountain was cut through by the Lehigh River to form the Gap, both a railroad and the new highway must run beside each other on the east bank of the river in an area obviously restricted by the steeply sloping rock wall of the gap. It is necessary, therefore, to cut back into the rock of Blue Mountain to make room for the wider road, and also to provide for a retaining wall against the rock face. Moreover, in one stretch, where the roadway passes between piers of an existing railroad bridge, the available space is so narrow that a cantilevered upper roadway is designed into this retaining wall and is anchored back into the rock, as shown on the cross-section view.

It is now instructive to look into the geology of the region, to obtain which we had the valuable assistance of Dr. Carlyle Gray of the Pennsylvania Bureau of Geologic Survey. The origin of picturesque water gaps like that of the Lehigh River has been the cause of much conjecture in the past; but the best opinion today is that the river once flowed over a land surface about 1200 feet higher, corresponding with the elevation of the top of the mountain, and gradually cut through the mountain over millions of years' time until it carved out the Gap. Thus the Gap is not the result of a massive fault or cataclysmic earth movement, and any doubt as to whether the cantilever roadway will be broken by a continuing rock movement is minimized.

Borings taken for the new work reveal that the rock in most of this area of the proposed cantilever wall is a hard quartzitic conglomerate, composed of quartz grains and pebbles cemented by quartz, which has been identified as Shawangunk Conglomerate of the Silurian Period. The downstream portion of the wall is founded on hard shale of the Martinsburg formation. These rocks are obviously suitable both for founding and for anchoring, provided their stratification does not slope toward the river so that masses of rock would slide diagonally downward carrying the retaining wall and cantilevered roadway with it. That the rock will not do so is demonstrated by its geological history; for Blue Mountain itself is the is the south limb of an eroded syncline with its axis normal to the river and with a substantially level ridge crest. Therefore, although its strata in the Gap can be seen in cross-section to follow the curve of the syncline, the strike of the bedding planes is normal to the proposed retaining wall, alleviating any fear of disastrous slips along bedding planes in the direction of the river.

Decomposed Rock at Tacony-Palmyra Bridge

What we have just discussed shows the assistance of geology in determining the absence of major faults in rock and the determination of direction of rock strata.

We will now leave rock in its customary sense to discuss another Pennsylvania phenomenon - "decomposed" rock as it is found in the Philadelphia area.

In 1926, when we were making test borings for the Tacony-Palmyra Bridge across the Delaware River at Philadelphia, using the then conventional wash borings, the river bed was found to consist of river mud immediately below the water, then hard gravel and finally gneissic rock. When rock was encountered in such wash borings, the churn bit would ring with the impact as it struck the rock, and the cuttings brought to the top of casing by the wash water would show small rock particles and mica flakes. Toward the New Jersey shore, the penetration of this rock by the drill bit was not quite so difficult, and the characteristic ring would be absent during the drilling, but the materials brought up by the wash water were the same mica flakes and quartz particles. The driller described this material as "rotten rock." The piers were designed to be carried just into the rock surface. However, when excavation inside cofferdams was completed for two of the piers of this New Jersey side, and water had reached this "rotten rock" material, it softened noticeably, and it was found necessary to drive pipe piles through it to the sound rock beneath in order to make a sound foundation.

Decomposed Rock at Penrose Avenue Bridge

About twenty years later, when the Penrose Avenue Bridge across the Schuylkill River in South Philadelphia was being designed, the same phenomenon was encountered, but the investigation was carried out under naturally more modern conditions. Test borings were made in which a sharpened steel tube was driven into the material to bring up representative samples unaffected by washing water, and when sound rock was encountered, core samples were drilled with a diamond bit. Coarse sand and gravel was found in a thick stratum down to about El. -83, then decomposed rock to about El. -104, and finally sound gneiss rock was penetrated. The decomposed rock could be sampled by driving into it the sharpened steel sampler, with which specimens could be brought to the surface. These samples were tested and were found to consolidate under load like clay. Since we had found on the Tacony-Palmyra Bridge that it was not wise to expose this material to water by excavating to it, it was decided to found the two main piers well above it at El. -65 on coarse sand and gravel and to accept the minor pier settlements which would result due to consolidation of this 20-foot thick underlying stratum under bridge loads. Actually, the piers have settled less than an inch by being founded above this softer decomposed rock in its natural state.

This was done, however, only after learning from the Pennsylvania Bureau of Geologic Survey how this decomposed rock was formed. Their explanation for it was as follows. The hard gneiss rock was exposed at the surface and decomposed about 60,000,000 to 80,000,000 years ago, in Upper Cretaceous time when it was at ground level and water leached through it over thousands of years' time. The original gneiss rock contained quartz, mica and feldspar. The feldspar was hydrated during the leaching action and changed into kaolin, a white clay. Thus this decomposed gneiss rock actually contains clay and will consolidate under load, as the laboratory tests we had made indicated would happen.

Preconsolidated Clay at Walt Whitman Suspension Bridge

You will note that what we have just discussed is a rock which behaves somewhat like a clay. Our next bridge, the Walt Whitman Suspension Bridge over the Delaware River between South Philadelphia and Gloucester, is a case where decomposed rock is also in evidence at a great depth in the neighborhood of El. -200, with sound gneiss below it, but where the two tower piers in the river were founded at reasonable depths above it in a stiff red and white clay. Several of our recent large bridges have been founded in stiff clay, which consolidates somewhat under load but otherwise serves very satisfactorily as a founding material. However, it is alway desirable to investigate the geology of such clays, to determine whether there is any reason why they may have been preconsolidated, for such geological preconsolidation is very beneficial. In the case of the Walt Whitman Bridge, for instance, there was evidence from the consolidation test results that these clays had been heavily preconsolidated in the past, and therefore the settlements of about 7 - 1/2" which were calculated for the river piers by reading directly from the consolidation curves could be considerably discounted, and were. The piers have actually settled less than an inch.

Here we investigated the geology of the materials by consultation with the U. S. Geological Survey office then located in Philadelphia, with the following results. The red clay at the Philadelphia tower pier, which extends from about El. -100 to El. -145 is of the Raritan Formation, Upper Cretaceous Age of the Mesozoic Era, and has an age of approximately 80,000,000 years. Probably the decomposed rock beneath it here is of the same geological age. The sand and gravel and mica flakes underlying it are part of the same formation, which is very flat in this area; while the sand and gravel overlying El. -100 are of the Cape May formation, of the Pleistocene-Pliocene Age of the Cenozoic Era, and are much younger in geological age. Since the red clay is of Upper Cretaceous Age, it has been subjected to the weight of younger strata which have subsequently been partially eroded, and therefore had opportunity to be preconsolidated either by the weight of these other strata extending well above it, or by being at the surface at some time which would have presented the opportunity for drying out some of its water and causing it thus to preconsolidated by shrinkage.

Glacier Deposits at Newburgh-Beacon Bridge

At this point, before referring to our next project, which is in a glaciated area, it would be well to consider a geological time scale for the more recent eras, based on a table in "Engineering Geology" by Ries and Watson.

Geologic Time Divisions

ERA	PERIOD	EPOCH	REMARKS	
Cenozoic	Quaternary	Recent	10,000 years ago	Glacial
		Pleistocene	1,000,000 years ago	
	Tertiary	Pliocene		
		Miocene		
		Oligocene		
		Eocene		
		Paleocene	60,000,000 years ago	
Mesozoic	Cretaceous	Upper		
		Lower		
	Jurassic			
	Triassic			

It will be seen from this time scale that the geologic events we have been discussing are in the next era older than our present Cenozoic Era, since both the decomposition of the rock of the Philadelphia area and the formation of the hard clay of the Walt Whitman Bridge took place in the Upper Cretaceous Age of the Mesozoic Era. The next project we wish to discuss is the Newburgh-Beacon Bridge over the Hudson River, where glacial action took place in the Pleistocene and Recent Epochs of the Quaternary Period of the Cenozoic Era, probably between 10,000 and 1,000,000 years ago. After that, we will wish to discuss clays at New Orleans which are apparently of Pleistocene Age, probably not much more than 10,000 years old.

Now let us consider the Newburgh-Beacon Bridge, located across the buried valley of the Hudson River not far below Poughkeepsie. According to Ries and Watson in "Engineering Geology," the land of the Atlantic Coast stood much higher in recent geologic times, so that the Hudson River carved a deep gorge, whose continuation can be traced by a trench on the sea bottom for some distance beyond New York Bay. This rock gorge at the Newburgh-Beacon site is well over 300 feet deep near the west bank, and we did not reach the deepest point with borings. Subsequently, the land was depressed lower than it is now and the Hudson River gorge was filled by clay and sand brought down by the river, and in part by glacial drift, resulting in the very confused stratification shown, having sand, gravel and boulders in some areas, and silt and clay in others. A matter of principal concern to us is the mixed silt and clay pocket just overlying rock on the east side of the rock gorge. This is overlaid by coarse sand and gravel between elevations -80 and -140 which would be very difficult to penetrate with a bridge pier caisson; yet below it, between elevations -140 and -190, occurs this silt and clay deposit overlying rock which must sustain the distributed bridge loads at that depth.

A geologic examination, by the Gahagan Dredging Corporation of New York City, of the boring samples above this pocket of silt and clay has indicated that glacial till exists in the materials overlying it. This would indicate that glacial ice has over-ridden all of these deposits at some time in the past. Consequently, the silt and clay pocket itself must have been subjected to the effects of this action, increasing stability and compaction of the silt and clay. From this reasoning it may be assumed that some beneficial preconsolidation took place on these clays in glacial times, and if that is so, little pier settlement will result from bridge loads. Actually, bridge pier settlements beneath the heavy blanket of coarse sand and gravel should not be serious in any case, and it is proposed to found two piers in this coarse sand and gravel overlying silt and clay. If the silt and clay is actually preconsolidated, settlements will be negligible.

Pleistocene Foundations at the Greater New Orleans Bridge

The two main piers of the Greater New Orleans Bridge now under construction across the lower Mississippi River are founded on Pleistocene clay of an age of more than 10,000 years, although

The fine-grained alluvial deposits above this are of Recent age. The cross-section prepared by Dr. Harlan N. Fisk for the Corps of Engineers of the War Department shows the City of New Orleans on the left, with Algiers on the right, and indicates that the river in recent geologic times migrated from a position back of the New Orleans levee to its present position, filling in the older river position with fine sand. Surrounding the river on both sides are recent marsh and swamp deposits on the top, with marine and brackish water deposits immediately beneath. In consequence, the New Orleans anchor pier of our cantilever river crossing is founded at El. -65 in the sand deposited in the old river cross-section back of the New Orleans levee, while the remainder of each approach is founded on piles through the swamp and marine deposits to firm materials.

Small shells are found in the boring samples from the marine deposits. Immediately beneath this shell stratum on the Algiers side is a hard gray and tan sandy clay which is locally referred to as "the Pleistocene," and which as a matter of fact is the very top of the Pleistocene deposits, colored tan because it was once exposed as part of the coastal plain, and while thus exposed was oxidized to change its color from the characteristic gray of the Pleistocene deposits which lie beneath it. When these tan strata were at the surface in that fashion, they also dried to an extent, causing the clays to shrink and harden, which consolidates them as much as would a load long applied to them. We therefore have regarded the tan and gray upper crust of the Pleistocene as an excellent material for the support of piles driven on the Algiers side. Below this tan and gray stratum, the Pleistocene deposits again become softer until great depths are reached where they become harder in their natural state.

The two main river piers of the cantilever bridge have presented a special problem. The greatest river depth on bridge center line is actually at El. -120 and we have carried these two main piers to El. -180 and El. -165 in order to support them well below river bottom, upon a gray clay of satisfactory strength and stiffness. The gray Pleistocene clay does become stiffer at these depths and actually has quite adequate shearing strengths up to 4,000 pounds per square foot, ample to withstand the huge bridge pier loads. As a measure of its compactness at that depth, the moisture content of this clay is about 30 per cent. A pressure of about 2.5 tons per square foot in excess of previously existing pressures is being applied to this clay by the piers, the caisson size of the largest pier being 88 feet by 151 feet in plan, which places it among the largest caissons ever built. The total gross buoyant dead load at the base of this largest pier is 67,000 tons.

Now it is significant that this lower Pleistocene clay has not been preconsolidated, so far as we are aware, either by a massive additional weight upon it at some time in the past, which would tend to squeeze the water out of it; or by exposure to the air, for there is no evidence by change in color that it has been oxidized. Therefore, it is not expected that the consolidation of this clay which will occur under bridge loads will be lessened by the effect of preconsolidation.

Conclusion

It may be seen from the foregoing examples how geological history can enter into the design of bridge and highway foundations. Actually, we do not always go into the detailed stratographic history of an area which a bridge is to cross, but we are aware of the benefits of geology to us and hope, by such contacts as this symposium affords, to learn more of it as it applies to our projects.

PEDOLOGY HELPS THE HIGHWAY ENGINEER

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PEDOLOGY HELPS THE HIGHWAY ENGINEER

A January 17 news flash from the American Road Builders' Association yielded the following interesting statistics. "In 1956, the highway construction industry accomplished approximately $5 billion of new highway construction, an increase over the $4.8 billion of 1955. The forecast for 1957 is $5.6 billion. During the first nine months of 1956, there was an average of 5.6 bidders per Federal-aid contract. During this period the bids on $1,057 million of work placed under contract were $43.3 million below the engineers' estimates."

Such large volume of highly competetive bidding should provide the taxpayer with the most miles of the best highways per dollar spent. But, is this the case? In this same news flash the A.R.B.A. goes on to recommend administrative procedures which the contractors feel would improve the quantity and quality of the highways yet to be built. In order to increase the anticipated trouble-free service to be rendered by these highways the contractors suggest an updating of specifications - specifications written around end results rather than method - allowing the contractor a wider latitude of choice of methods and the newer types of equipment. This is not just the old cry of the contractor wanting to be let off the hook, it is the reasonable request of an entrepreneur venturing risk capital, who has real pride in his work. What chance does he have in exercising best judgement in the construction of a highway foundation? This depends on specification interpretation. Soil is the foundation material. How capable is the inspector of interpreting soil behavior? To pose the question another way - to what source of information can the inspector turn to get the information which will help him to make evaluation judgements regarding the soils in place of those being moved in? The information of which we speak is normally provided by the planners and designers, so that the inspector can decide questions regarding compaction, drainage, or the possibility of frost action.

Highway planners and designers anticipate such questions as these raised above. In an attempt to provide the answers, they make an effort to choose a route which will pass over the best soil types. Following this, they test the soils to be encountered, and prepare reports which classify and evaluate the soils as subgrade materials for highway construction purposes. It is with this phase of the work that this paper concerns itself. In order to have maximum carry-over of information from one job or area to the next, care must be exercised in the system selected for the identification and classification of soils.

Effective planning requires map information, not only of the topography, drainage, and culture, but also of the types of soils to be encountered. Soil type is to be interpreted here as meaning significantly different from adjacent soils with regard to construction characteristics or behavior in service as a highway subgrade material. This poses the very real question as to how best to identify the soils so that they can be mapped as significant map units.

Pennsylvania recognizes the need to embark on such a mapping program, and hopes to coordinate the ongoing work of the Soil Engineering Section of the Pennsylvania Department of Highways with a special development program. At the present time the department is operating with a Division Soils Engineer in each of its 11 Highway Districts. Each new project is reviewed in the field and the proposed center-line is profiled. Standards for field methods and presentation of the data are being established. There is no state-wide system of collecting and mapping soil information at this writing. In the face of the greatly expanding Federal-aid highway program, it becomes imperative that some coordinated program of soil mapping be started in order that the valuable data which is being collected can be organized and used in future planning without indulging in unnecessary repetition. A review of the literature reveals that there are several possible approaches.

One possible approach is simply to start to plot at large scale a record of the results of the standard Bureau of Public Roads classification tests as this data accumulates. At the present time this is the form of data that comes out of the profiling that is described above. It is immediately apparent that such a program would be a long time in revealing the existance of soil areas of engineering significance as map units useful to highway planners.

Another possible approach is to attempt to relate these test data to the existing county soil survey reports. This is partially what has been done by the state of Michigan's Highway Department. In their Field Manual of Soil Engineering they have listed some one hundred soil series names, plotted the soil profile, described the natural occurence of the soil and given construction information. A chart has also been prepared grouping the series by texture and origin within drainage classifications. A second chart tabulates soil engineering data and recommendations versus series name. Data for this latter chart comes from soil tests and performance survey records. Whereas this system has many merits its chief fault is the large number of map units that a highway planner has to deal with.

In the writer's opinion, an improvement on the Michigan system is that developed in Indiana. Again the soil series as mapped by the Bureau of Chemistry and Soils is the basic classification unit. Each series name is listed with its probable profile range, soil area, slope class, location, description, problems and corrections. In order to reduce the numerous series to a reasonable number of groupings of significance in an engineering sense, they are grouped using as parameters topography and parent material. Topography is divided into five slope classifications, and parent material into six areas each of which may have from one to four subdivisions. This results in about fifty-three map units to cover the entire state. A soil within a map unit may be identified as belonging to one of fourteen units by texture or one of four rock types. These textural groups are related to Bureau of Public Roads Class, Atterberg limits, and dry density. Moreover the engineering map units are related to air photo identifyable characteristics. This seems to give the maximum of useful data for highway planners and designers.

A thorough study of the Indiana system leads one to the recognition that the soil is first identified by its profile and given a series name. Pedology dictates that the profile must be studied over its full depth to appreciate soil relationships. This is true too for engineering relationships. Pavement behavior is related to the position relative to a soil horizon. Degree and development of horizons in a profile is a function of topographic position and drainage - within a climatic zone. A given series name represents a given degree of development of soil profile on one parent material. Different degrees of development adjacent to one another on a given parent material are largely in response to difference in drainage characteristics related to relative topographic position. Such adjacent series can be grouped in catenas. This is a term borrowed from the pedologists meaning a family of soils occupying different slopes within an area and derived from similar parent materials.

If soils are grouped by catenas, this leads to the recognition of identifiable units in an aerial photograph which is best described by the geomorphic unit called a land form. In many cases the land form is the map unit of engineering significance. This is not always true because of modifications related to drainage. However this leads to the next possible system of preparing engineering soil maps, namely land form and drainage maps.

The approach used by New York, New Jersey and others is to divide the state into physiographic provinces related to topography or elevation and to geology. Within a physiographic unit an area of significance from the point of view of a highway engineer is recognized by its deposition, parent material, soil and rock profile, land form and drainage characteristics; these items being usually identifiable on an air photo. Engineering soil maps made by this approach can then be related to field test results as the data is accumulated over a period of time.

What shall Pennsylvania adopt as a method of procedure? Perhaps the most eloquent presentation for the case of using soil series names as a basis for interpretive classifications for engineering purposes is that of Earl J. Felt in a paper presented as a part of a Symposium on The Identification and Classification of Soils prepared as Technical Report No. 113 by the American Society for Testing Materials. Mr. Felt makes the point that identification of a soil must preceed classification. There is a logic behind the identification of soils by series names which has stood the test of time and moreover provides a reasonable framework on which to tie additional information such as that provided by the Bureau of Public Roads Classification system. The Bureau of Public Roads system classifies, it does not identify. Since the most important first step is to identify, then let be adopted a system related to natural soil forming processes. This point can be well made in the case of a varved clay, and will be in a latter portion of this paper.

Another very practical reason for choosing a mapping system related to soil series names is the fact that much valuable work has been done in the past and is continuing to be produced. The soil mapping formerly carried out by the Department of Agriculture under the Bureau of Plant Industry, Soils, and Agricultural Engineering and the Soil Conservation Service has been centralized under the latter of these two bureaus. The Soil Conservation Service has entered into a cooperative program with the Bureau of Public Roads in order to present, in each one of its newly published county soil survey reports, a chapter on the engineering significance of the soils in the county. Potter and Lancaster counties, the next two reports to be published by the Soil Conservation Service will contain such chapters. The writer was privileged to be able to sit in on part of the conferences held by representatives of these two agencies as the soils of Potter County were discussed. This will be such an effective help to the highway engineer that the department of highways is planning to relieve the Bureau of Public Roads of the burden of the testing for classification purposes; and take over this work themselves. Effective as these new Soil Survey Reports will be, when it is realized that, at best, four county reports a year may be produced, Pennsylvania will be a long time in being adequately covered.

If a system of mapping related to soil series is to be chosen, some comment should be made regarding the adequacy of the maps already completed. First let it be recognized that the work is done by humans like ourselves and is therefore subject to human error. Then since the science of pedology has been progressing over the years, there is a marked difference in the degree of refinement of subdivision into soil series. Bulletin No. 22 of the Highway Research Board entitled "Engineering Use of Agricultural Soil Maps" contains a chapter on Status of County Agricultural Soil Mapping in the United States. Reference to Pennsylvania shows that only a few of the 67 counties are rated as most nearly adequate. Pennsylvania soil maps bear dates from 1904 to the present. A series of five Reconnaisance Maps were completed from 1908 to 1912 to cover the state by regions, but these are rated as "general but of some value." As examples of the information that is obtainable from maps of two different periods of time, soil problems from two different counties have been selected, namely Tioga published in 1929 and Union published in 1946.

One failure investigated is on the property of Mr. Powers of Wellsboro, Pennsylvania and is located in farming country ½ mile west of Mansfield, Tioga County, Pennsylvania, near the junction of Highways US 15 and US 6 about 15 miles from the New York State line.

The failure area is located along the bank of Ellen Run, a stream draining approximately ½ square mile of watershed area. A township road parallels the run at approximately 100 feet uphill. A house is on the uphill side of the road and a barn is approximately 300 feet further up the slope.

The failure area is approximately the shape of a segment of a circle and is outlined by the overlay attached to the enlarged aerial photograph, Figure 1. The area extends approximately 1000 ft. along and up to 270 ft. back from Ellen Run. The house is within the area of the slide. The electric power line connected between the house and barn has been extended 10 ft. since the start of the slide failure about ten years ago. Approaching the site from either side, the hard surface of the macadam road terminates at the failure area. Within the failure area, the macadam surface has progressively broken up, and is no longer in evidence. The alignment of the road is noticeably offset as it crosses each edge of the moving area. A telephone line runs along the road within the failure area, and service has been interrupted at times due to sinking of the poles causing breakes in the line. Although the house has been moving since the start of the slide, destruction to it has shown up only in the last four years. Previously there was no differential movement of the foundation in either a horizontal or vertical direction. In the last four years, frequent breaks paralleling contours have occurred, giving the surface a rippled appearance. These breaks, running under the house, have caused destruction of the foundation.

The start of the slide failure occurred during an unusually heavy rainstorm in the spring of 1945. Movement at the boundaries was evident in both the horizontal and vertical directions. A large volume of material moved into the stream channel, including a row of willows along the stream bank. The loosened material is continuously eroded by flood waters and carried downstream to the Tioga River. This movement of bank material into the stream channel has reoccurred during the wet period in each successive spring season.

A line of levels was run normal to both the stream and the breaks on the surface to establish a typical side-hill section and is reproduced in Figure 2. This section shows the unstable area to have a slope of only 12%. This is a flat grade, when thinking in terms of a soil slide.

A detailed examination at the site disclosed that the the soil is being pushed up at the bank adjacent to the stream. The pushed-up material does not accumulate in quantity, as it is being continually eroded away. The rise in the ground surface adjacent to the stream is shown by the tilt of the ridge line of the garage roof, as shown in Figure 3.

Further examination of the channel bank showed a pronounced and closely spaced strata formation, having a dip of approximately 45° due south on exposed sections at right angles to the stream. These facts indicate a typical arc failure extending to a considerable depth, similar to the base failure of an open cut cross section. Failures of this type are known to occur in weakened clay soils. The probable failure arc is outlined on the overlay attached to Figure 2. The accumulation of rising material at the lower end of a failure arc would normally produce equilibrium. However, at this site the rising material has continually been removed by flood waters, preventing equilibrium; thus the slide has continued through the years.

The inability of the soil to support a side-hill slope of only 12% indicates an unusual soil or soil structure. There is no evidence of excessive ground water pressure producing surface outflow at any of the many cracks in the moving area. The cracks normally intercept surface water from the upper hillside.

The previously mentioned stratification consists of two types of material in alternate layers of varying thickness and extends to considerable depth. On some vertical sections 24 layers can be counted in one foot. The bank surfaces showed a water-worn effect, indicating that alternate strata had less stability when in contact with the moving water. The weak strata had a prevailing gray color, and the stronger strata had a pronounced reddish tint. Hereafter, the strata will be designated as gray or red. A wedge cut was carefully made in the bank face, Figure 4, and the resulting fresh-cut face is shown in Figure 4. The alternate strata can easily be traced from each exposed face across the newly cut surface. The final field inspection date of March 12, 1955 proved profitab e, in that the exposed surface showed the water voids left by the melted ice lenticles, which occurred in quantity in the gray strata. This shows the gray strata to have porosity, permitting the movement of free water. On this date, soil samples were carefully taken from each type of strata for further testing.

The initial field inspection was made December 10, 1954, at which time random soil samples were taken representing the total soil mass. The laboratory testing consisted of the determination of plastic limit, liquid limit, and particle size distribution. These tests were made in order to classify the soil. The sieve analysis showed 100% of the soil passing the 200-mesh sieve (0.074 mm); therefore the hydrometer analysis had to be used exclusively in the determination of particle size distribution. The average results of several tests are shown by Curve A of Figure 5.

Particle size distribution Curve A shows about 91% of the soil is finer than 0.005 mm, the division between silt and clay in some classification systems. Classification by other systems is as follows: U. S. Bureau of Soils, by textural classification, clay; Revised Public Roads classification, Type A-5 (on basis of laboratory tests--liquid limit: 43.9%, plastic limit: 48.2%, plasticity index--(PL - LL): 4.3%, 100% of material passing 200-mesh sieve); Airfield classification, Type ML. Reference to the Tioga County soil survey map of 1915 discloses--"the soil between the stream and road was mapped as Alluvial Soils (undifferentiated) predominantly silty clay loams. The soil uphill in the failure area was mapped as Volusia, silty clay loam, containing seepy strips and springy spots, which is characteristic of the lower slopes." These wet strips are related to the stratified soil and are now showing up adjacent to the stream. Since this map was printed the original material adjacent to the stream was displaced by the failure and carried away; being replaced by stratified material.

The soil has been classified by various systems using the information obtained from laboratory tests. However, laboratory classification does not give sufficient information to fully characterize this unusual soil structure, further field inspection was necessary to obtain supplemental data, and undisturbed samples. In addition to the classification obtained from laboratory results, the fact that the soil is stratified enables us to classify it as a varved clay.

To further prove that the gray strata are distinctly different from the red strata in particle size, hydrometer tests were run on the samples taken from each type of strata. The gray strata are predominately silt and the red strata are predominately clay, and are respectively identified as Curves B and C in Figure 5. The silt layer is pervious, and allows water to flow through it. The clay layer is practically impervious, thus holding the water in the silt layer, which in turn becomes a path for ground water flow. There is a slight penetration of water into the clay layer, and when the water reaches a certain concentration in this layer, an unstable condition exists. It was previously pointed out that the failure started as a result of an extremely heavy rain, and that the continuing failure is more pronounced in the spring of the year when the water content is increased by the spring rains and thawing of the ground.

An additional factor, which was not fully developed is a prediction of other sites within this area (or like areas) where varved clays might exist. This information would be of value to the highway engineer, as well as structural and hydraulic engineers. Such predictions would rest on a knowledge of sedimentary and historical geology. Varved clays, like the anular rings of a tree are seasonal phenomena. The coarse particles of silt are laid down under one set of environmental conditions; whereas the clay particles would require still other conditions.

Consider the following hypothesis. In late pleistocene glaciation an ice lobe bulldozed an earth dam behind which is impounded a lake. Tributary streams feeding the lake would create sand and gravel deltas along its shores. Further away from shore during periods of free melting, silt would be deposited on the lake bottom. Only during winter freeze-up when distributive currents in the lake are inoperative would the clay particles settle out. After many years, spillage over the dam would erode an excape channel which would enlarge and carry away much of the dam. Downstream deposits should include unassorted gravels, outwash plains (area permitting) or terraces. Whereas behind the dam would remain the varved clays, deltas and other shore deposits: these modified by geologic periods of time. Reference to the Tioga county Soil Survey Report gives little help in this direction. The soils are grouped in series by internal drainage, the only recognition given to land form being the terms upland, terrace, and bottom soils. Nowhere are varved clays mentioned.. In this case then, it will be necessary to seek to supplement the soil series mapped with information from aerial photographs.

By way of contrast a review of a reconnaissance survey for a Federal Aid Secondary Road between Forest Hill and Mifflinburg in Union County may be of interest. The more recent soil survey report of 1946 subdivides the soils to a much greater degree of refinement. The selected road runs north and south. A study of the soil map shows many soil series as map units which are not significantly different from the highway engineering point of view: nor are the differences readily recognized on an aerial photograph. Hence it is advisable to search for a grouping of soil series which will make the engineering significance of their differences more apparent.

First a study of the geology of the area is in order. This area, as included in the Valley and Ridge Province of the Appalachian Highlands, a highly foliated region of sedimentary rocks, shows up as a succession of narrow ridges and valleys. Most of it was covered by a huge ice sheet from the north, that has left a marked effect on the area. The geological formations as listed in order from the youngest to the oldest are the Wills Creek shale, Bloomsburg red shale, Clinton shales and limestones. Also a small amount of Jerseyan Drift is present. The beds are highly foliated and, as can be seen by stereoscopic examination of the aerial photos, the resistant formations form the ridges and a trellis drainage pattern is exhibited in places.

The youngest formation, Wills Creek shale, is between 400-750 ft. in thickness and is chiefly thin fissile calcareous gray shale with thin layers of limestone and may contain red shale and gypsum. The next oldest, the Bloomsburg red shale, is largely non-fissile, lumpy, red shale with smaller grey, green or yellow shale beds, red sandstone and locally impure limestone. The oldest formation, Clinton shales and limestones, is largely a grey-greenish shale with a small proportion of limestone and greenish-red sandstone. It contains beds of fossiliferous or oolitic iron ore. (See plate 1)

The soil survey report describes the soils of this undulating to gently rolling area as quite complex in nature, in that they have been developed from (1) deep glacial till, (2) shallow glacial till and residual material, (3) colluvial slopes, (4) terraces and (5) the bottom lands. In general these soils are deep, productive, and high in organic matter and with good structure throughout, the exception being soils of residual material and very shallow till. The soils of groups (1) and (2) are well drained for the most part. Drainage has affected the soil profile to a marked degree.

In general the contribution to local materials from geologic formations beneath the soil mantle are in excess of those from foreign sources, and the chemical and physical characteristics of these local materials, as a result of weathering processes, have been passed on to the soil in modified form. These soils are derived from calcareous shales and limestone, for the most part. As for the color, it ranges from gray through pale brown, yellowish brown, brown and reddish brown. Textures range from loamy sands to silt loams and clays. Soils of the terraces and bottom lands lack uniformity.

Getting more specific as to the particular soils encountered along this road, under deep glacial till is found only the well drained acidic Norton reddish brown clay loam, derived from sandstone and shale and deposited as Jerseyan drift. Klinesville and Kutztown, soils from residual material and very shallow till, occur in considerable magnitude. The former has developed over Bloomsburg red shale at a rather shallow depth, while the latter yellow shaly clay loam has developed from Salina shale and occupies the high ridges. Also found to a large extent is the well drained Norton, as a soil from shallow glacial till and residual material; that is, a thin Jerseyan drift over sandstone and shale. Occuring in lesser quantities, but all developed from shallow till and residual material, are the Mifflinburg associated with the Kutztown; Hartleton; Hagerstown, developed from Helderberg limestone; Weikert, formed from gray Clinton shales. Soils of the colluvial slopes consist of Norton, Araby and Wiltshire, all of whose parent material is limestone and shale. Soils of the bottom lands result from stream alluvium of the limestone areas.

In an attempt to bring out significant relationships an effort was made at grouping the soil by parent materials. This involved shading adjacent soil series combining them into larger map units. See Plate 2. This grouping revealed land forms and drainage patterns which were readily recognized on the air photos. Stereoscopic study of the photos make the soil map come to life in terms of the factors that are of interest to the highway engineer.

Soils derived from deep glacial till were of little consequence from the point of view of lineal footage encountered.. Shallow till and glacial material were dominated by the geology of the underlying rock, hence for this job differentiation between shales and limestones was more significant. The terrace soils were well drained and would have been satisfactory for special subgrade had it been required. The troublesome soils were the bottom land soils, not so much as a factor of soil texture but because of the poor drainage.

Thus is presented two cases where the soil maps as they exist are inadequate for different reasons. The Bureau of Public Roads classification by itself is equally as inadequate. However a combination of identification of the soils by pedological methods, air photo study, and geologic investigation makes it possible to come up with significant map units. Admittedly areas of varved clays will be small as map units, but they represent trouble spots and must be forcefully brought to the attention of planners and designers.

Perhaps now a method of engineering soil mapping can be suggested:

1. Divide the state of Pennsylvania into physiographic provinces.
2. Select a unit area (perhaps one county which has been recently pedologically mapped) within one province and list its soils.

3. Study the soil series names within the selected area with respect to:
   a. land form
   b. drainage patterns
   c. erosion characteristics
   d. vegetative cover
   e. relative topographic position
   f. extent of profile development
   g. position of ground water table
   h. characteristics which relate to air photo identification.
4. Attempt to group the soils (perhaps by catenas) so as to reveal recognizable units on an air photo.
5. Study the aerial geology.
6. Visit the selected area in the field to:
   a. verify the aerial geology
   b. examine the soil profiles and identify by series names
   c. make field tests for density, and sample horizons
   d. record evidence of highway construction or maintenance difficulties
7. Make necessary laboratory tests to complete classification by the Highway Research Board modification of the Bureau of Public Roads system, as well as compaction tests.
8. Correlate:
   a. soil series names versus BPR classification
   b. soil series names versus air photo identifiable characteristics verified in the field
   c. air photo identifiable characteristics versus BPR classification
   d. soil series names versus construction difficulties
   e. soil series names versus maintenance difficulties
9. Prepare engineering soils maps on the county soil or highway series as a base map.

10. Select a test area previously unvisited, predict - and verify.

Commenting on the above outline it might be pointed out that initial studies as in item three would call for concurrent study of soil map and air photos. An attempt should be made to coordinate the anticipated field studies with the ongoing work of the Pennsylvania Department of Highways so as to benefit from the test data which would become available. Advantage should be taken of the IBM data processing equipment at the university to make the correlation studies.

In conclusion it should be emphasized that the end product, the engineering soil map, is of necessity reconnaissance in nature. It will serve well for location criteria, background information on which to base generalized engineering decisions, and to indicate sites where more particular soil sampling will be advisable. Accompanying tables and charts will be able to provide much physical data to serve as a guide to inspectors as well as designers and planners.

Well, -- shall we begin?

BIBLIOGRAPHY

1. The Origin, Distribution, and Airphoto Identification of United States Soils, by D. S. Jenkins, S. J. Belcher, L. E. Gregg, and K. B. Woods. Technical Development Report No. 52, Civil Aeronautics Administration, 1946.

2. The Formation, Distribution, and Engineering Characteristics of Soils, by D. J. Belcher, L. E. Gregg, and K. B. Woods. Research Series No. 87 of the Joint Highway Research Project Engineering Experiement Station, Purdue University, 1943.

3. Field Manual of Soil Engineering (Revised) Michigan State Highway Department, Lansing, Michigan, 1946.

4. An Engineering Grouping of New York State Soils, by Earl F. Bennett and George W. McAlpin, Bulletin No. 13 of the Highway Research Board, Division of Engineering and Industrial Research National Research Council, 1948.

5. Soil Series Names as a Basis for Interpretive Soil Classifications for Engineering Purposes, by Earl J. Felt. Special Technical Publication No. 113 American Society of Testing Materials, 1950.

6. Soil Survey of Tioga County, Pennsylvania, by B. H. Hendrickson, R. T. A. Burke, K. V. Goodman and K. L. Smith, Bureau of Chemistry and Soils, U. S. Department of Agriculture, 1929.

7. Status of County Agricultural Soil Mapping in the United States, Bulletin No. 22 of The Highway Research Board, Division of Engineering and Industrial Research, National Research Council, 1949.

8. Soil Survey of Union County, Pennsylvania, by S. R. Bacon, David Taylor, Alfred Boileau, and Gerald Yoder. Agricultural Research Administration, Bureau of Plant Industry, Soils, and Agricultural Engineering, U. S. Department of Agriculture, 1946.

EXPERIENCES IN DESIGNING ROCK SLOPES IN NEW YORK STATE

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EXPERIENCE IN DESIGNING ROCK SLOPES IN NEW YORK STATE

INTRODUCTION

Previous to World War II, it was common practice in New York State to design all rock slopes 4-vertical to 1-horizontal, or, to the geologist, corresponding to a dip of 76 degrees. This standard slope was carried over into the post-war period on a 23-mile stretch of one of our principal highways. Considerable overbreak occurred on this job, and the geologist was requested to make a study for the purpose of determining the cause. The rock is chiefly Cambrian slate or phyllite with the bedding and strongly developed slaty cleavage about parallel to each other and both dipping from about 50 degrees to vertical. The alignment of the highway is, through most of this stretch, approximately parallel to the strike of the rock structures. Breakage was almost exclusively controlled by the cleavage. In cuts where the cleavage was flatter than the design slope, there was, therefore, considerable unavoidable overbreak on the side of the road toward which the cleavage dipped. In such cases, the design slope was obviously too steep. A large amount of overbreak was also due to careless and crude blasting practices.

On a succeeding stretch of the same road, a detailed study of all cuts was made and slopes recommended on the basis of rock structure. Also, blasting was performed in accordance with acceptable modern methods. The resulting slopes were stable, conformed closely with the specified design, and overbreak was reduced to insignificance.

Then came the Thruway with more rock excavation between Albany and New York City than on all previous highways in the State. With but few exceptions, every proposed rock cut more than ten feet deep in this stretch was studied in as much detail as outcrops and drill cores would permit, and designs, based on rock type and structure, were incorporated in the detailed contract drawings. The north end of this part of the Thruway is in Silurian and Devonian limestone, shales and sandstones, all of which were severely deformed by the Appalachian Mountain building revolution. The central portion passes through Cambrian and Ordovician shales, slates, limestones and sandstones, that were involved in both the Appalachian and Taconic revolutions, with a downfolded and severely faulted block of Silurian and Devonian sediments. The south end traverses the Gneisses and granites of the Highlands of the Hudson, the shales and sandstones of the Triassic Lowland and the Gneisses, granites and schists of the New York City Series.

Since the Thruway, most of our efforts in slope design have been concerned with the reconstruction of New York State Highway Route 17 to modern standards through the rough terrain of the southern Catskill Mountains. Geologic conditions here are entirely different from those previously encountered. The bedrock consists of graywacke sandstone with a highly variable proportion of beds and lenses of soft red shale and sandstone, all a part of the great upper Devonian Catskill delta. For practical purposes, the bedding is horizontal. One cut on this job will be 230 feet deep and will involve, in round numbers, a million cubic yards of rock excavation.

Based on the experience gained in the above projects and others, it is proposed in what follows to outline the slope design criteria and standards that we are following at the present time and to briefly discuss the more important geologic prinicples involved in these areas.

DESIGN CRITERIA AND STANDARDS

The Hazard of Falling or Fallen Rock.

In view of humanitarian considerations, it is imperative that the possibility of rock falling on modern highway pavements be absolutely eliminated. To accomplish this, it is necessary to carefully evaluate the possible effects of the weathering of rock and rock structure in relation to slope height, inclination of slope and set-back of bottom of slope from pavement edge, or width of bench if benching is practicable.

Clean-Up and Snow Removal.

It is economically unsound and, in many cases, almost physically impossible, to design a slope from which some rock will not fall in time. In order to reduce the hazard of clean-up operations to both traffic and maintenance crews, sufficient set-back must be provided, irrespective of the height of slope, for the operation of equipment off the pavement, and to allow ample space for snow removal from pavement and shoulder.

Cost.

In our earlier efforts, cost was considered far more critical than at present. This trend away from heavy emphasis on cost has been influenced chiefly by the following considerations:

(1) Spot checks on removing fallen rock by maintenance forces indicate a cost from five to ten times as great as doing the same work under construction contracts.

(2) Rock falls occur much more frequently during the Spring break-up than at any other time, and, if they interfere with traffic, must be removed promptly, thus tying up men and equipment needed elsewhere during this critical period.

(3) Claims arising from injury to vehicles and people due to rock falls on a pavement can be sufficient to overbalance the additional expense involved in a design that will eliminate such occurrences.

Dimensional Limits.

Irrespective of rock conditions, we have arrived at fairly definite conclusions with respect to some elements in rock slope design. In experience to date, it has not been found necessary to design any slope steeper than three vertical to one horizontal, or flatter than one to one, although there are known rock structural conditions in limited areas of our State where conceivably it would be necessary to exceed these limits.

We have come to place more and more emphasis on the distance from edge of pavement to toe of slope, or set-back. Based on observations and reports of actual rock falls and in consideration of modern traffic conditions, we consider a 25-foot set-back the absolute minimum.

Benching.

During the past few years, there has been increasing evidence of a trend toward benching of more or less standardized dimensions, with no consideration whatever for rock conditions. This practice led to a rather serious problem on one of our projects in folded rock in which, in spite of every reasonable precaution in blasting procedure, benches could not be constructed as designed and the highway had to be realigned while construction was in progress in order to eliminate the hazard of rock falls reaching the pavement.

Since then, our Bureau of Soil Mechanics has made a very thorough study of the entire problem, which has resulted in certain conclusions that are now considered basic to sound and economic benching design. Above all other considerations, rock structure must be such that an effective bench can be built by average construction methods and practices. Except in unusually deep cuts, we do not consider the advantages of benching over a straight slope sufficient to warrant unusual costs per unit of excavation. For this reason, we have practically ruled out benching in folded or structurally complex rock conditions and are recommending it only in horizontally stratified sedimentary rock.

With regard to width of benches, as a result of average rock structural conditions, there is always a probability that irregular overbreak along the front edge will, under normal construction practices, make it difficult, if not impossible, to hold to design width. Therefore, if the design width is too narrow, the end result may be no bench at all at places. In view of this and the fact that a bench must be wide enough to intercept rock falls and permit the operation of clean-up equipment, we consider 25 feet the minimum design width.

It is advantageous in reducing overbreak to a minimum and obtaining a smooth surface to construct benches parallel to the bedding. For the same reason, in alternating beds of soft and hard rock, benches should be at the top of a hard bed.

APPLICATION OF GEOLOGIC PRINCIPLES

Weathering of Slopes after Construction.

Relatively soft rocks, such as shales and many sandy shales, argillaceous limestones and weakly cemented sandstone, slough or breakdown to small fragments in a relatively short time due chiefly to simple wetting and drying, or freezing and thawing. Such rocks often appear sound and massive in a freshly opened excavation or in a drilled core. Judgment based on such data can, therefore, be misleading. If a previous road cut or other openings of know history is not available for direct observation and evaluation of the weathering characteristics, a wetting and drying or freezing and thawing test of a few cycles will often prove indicative. In rock that weathers in this manner, a cut slope should be designed to approximate a talus slope, for, regardless of original design, it will weather to a talus slope in time. Our recommendations in such rock is for a slope between $1\frac{1}{2}$ vertical to 1 horizontal, and 1 to 1, depending upon the estimated rate of weathering. In a slope as flat as this, benching is not considered advantageous or necessary.

In horizontally stratified sedimentary rock consisting of alternate beds of resistant and soft, rapidly weathering rock, the slope problem becomes more complicated. The soft rock tends to weather back under the resistant rock. When this progresses beyond the center of gravity of a joint block or other structural unit, a rock fall results, and such rock falls are often of serious consequence. The relative proportion of the two types of rock is considered important. No sharp line can be drawn on this basis, but, in our experience to date, if the soft rock predominates and occurs in a number of beds more or less evenly distributed throughout the depth of the cut, an unbroken slope of from 2 to $1\frac{1}{2}$ vertical to 1 horizontal is usually recommended. On the other hand, if the resistant rock predominates, benching has been recommended in a number of cases, especially in cuts more than about 50 feet deep.

In rock that is uniformly resistant to structural weakening by weathering within a reasonable period of time, our recommendation is for an unbroken slope of 3 vertical to 1 horizontal in cuts up to approximately 75 feet deep. In cuts of greater depth, we consider it advisable to break the slope with a bench.

Structure.

The structural condition of a rock mass is considered the most difficult element to evaluate in arriving at a satisfactory slope design. To begin with, it is difficult, and sometimes impossible, to determine from available outcrops how many significant structures are present, much less to evaluate their effects in a finished slope, and drilled cores are in many cases of limited value, even if the problem is of sufficient magnitude to warrant the expense involved.

However, as a result of our experience, we have arrived at what, for want of a better term, might be thought of as a systematic, though somewhat flexible, approach to these problems, which is somewhat as follows:

The number of structures cutting a rock mass is of primary importance; in general the more there are, the greater the difficulty in evaluating their effects. Their angular relation to one another and to the finished slope, as well as continuity and spacing, must be considered. The depth to which they extend and their improtance at depth are critical. The degree to which they control breakage in blasting must be estimated and, as a corollary, it must be decided whether or not it is practical or possible to blast a trim line at an angle to a dipping structure or parallel to it. There are still other structural elements that have required analyses in our experience, but these are the ones that appear most frequently.

We have made numerous attempts to arrive at generalizations that could be applied to rock slope design in complex structural conditions, but every attempt so far has led to the inevitable conclusion that each and every cut presents a unique problem, thus precluding any worth-while generalizations.

And now for the knock-out blow. The best rock slope design that conceivably can be drawn up is not worth the paper it is presented on, unless blasting methods can be controlled within reasonable limits. This has been fully realized and its importance appreciated in all of our work. We now have made a definite move in this direction by the inclusion of the following in New York State Public Works Specifications of January 2, 1957:

"Where rock encountered in cuts requires drilling and blasting, all necessary precautions shall be exercised to preserve the rock in the finished slope in a natural undamaged condition, with the surface remaining reasonably straight and clean.

"Blast holes shall be drilled at the inclination of slope along the line of proposed finished slope and in the adjacent areas. An approved system of relief or delayed blasting shall be employed.

"The spacing of the blast holes and the method of delayed blasting required will be depenedent upon the quality and the structure of the rock encountered and the method of blasting used in approaching the slope. The Contractor shall adjust his operations to obtain the required slope conditions, as called for on the plans."

PHOTOGRAMMETRY IN PRACTICE

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PHOTOGRAMMETRY IN PRACTICE

The words photogrammetry, aerial photography, photo interpretation, and photogrammetric maps are fast becoming terms in general usage by the highway engineer and geologist.

A brief resume of the history of photogrammetry and photo interpretation in this country, which would interest the highway engineer, would start in the 1930's when the government mapping agencies including the United States Geological Survey vastly enlarged their use of aerial photography in the production of quadrangle sheets. These quadrangle sheets for many years have been the only map coverage of many areas and have, therefore, been used as a basis for the preliminary studies and location of new major highways. At about this same time many highway departments started using photographs for studying drainage areas and property involved in new construction.

During the war years of 1941 to 1945, a major problem confronted the government in the production of adequate maps of United States and foreign countries in and around the war zones. A new department called "Army Map Service" was organized and assigned the tremendous task of supplying the maps necessary in the defense of our country. Photogrammetry proved to be the only science which could be employed to produce the required topographic maps in the short available time. Photo reconnassiance and photo mapping units of the Air Force were organized, and the never ending job of obtaining up-to-date photography of the important defense areas was begun. From this time on, aerial photographs were used every minute of the war and certainly played a major role in its final outcome.

It was also during these war years that photo interpretation proved so valuable in picking enemy manufacturing and defense installations. Photographs taken by units of the Air Force were used to prepare target charts of enemy installations and on the bombing missions of our allies for navigation and pin-pointing the desired targets.

After the cessation of hostilities in 1945 the mapping program continued in this country by United States Geological Survey, the Aeronautical Chart and Information Center, and Army Map Service. It was realized that United States was far behind many European countries in having available adequate maps for planning peace time improvement projects. Projects which would benefit the major population of the United States during peace time including the construction for dams, electric power, irrigation, highways, recreation, and agricultural usage could not be undertaken without these more recent and more accurate maps. Using these maps and aerial photography taken by the government, the Department of Agriculture has greatly aided the farmer over the past few years with their program of soil and natural resource conservation.

Photo interpretation was used by soil experts, geologists, and foresters in classifying types of soil conditions for agricultural purposes, in identifying areas of undiscovered minerals, and in classifying and programming the vast forest areas of the United States.

During the past four or five years, many of the State Highway Departments and several private consulting engineering companies have experimented with and adopted the use of aerial photography and photogrammetry in making highway studies. This is, in brief, the history of the photogrammetric profession leading up to the year 1956 when the Federal Aid Highway Act was passed which states: "In carrying out the provisions of this title the Secretary of Commerce shall, to the fullest extent practicable, authorize the use of photogrammetric methods in mapping, and the utilization of commercial enterprise for such services." The Public Works Committee report commented that this language was inserted "because of the almost spectacular results in savings of time, manpower, and costs through use of the techniques developed from the interpretation of aerial photographs and their application to mapping..." The Committee said it desired to encourage the Bureau of Public Roads and the States to continue and extend "Their already well-advanced usage of this method" to the enlarged program.

In order to take part in the greatly expanded highway program, several highway departments have realized that it would be necessary to use photogrammetric maps to a larger extent than in previous practice, and in general, have adopted the following method in the preparation of engineering plans.

The preliminary center line location and sometimes several alternate routes are laid out on the best available source maps which are usually quadrangle sheets published by the U. S. Geological Survey. These routes are then flown to obtain aerial photography at a suitable scale to plot maps for preliminary location and cost estimates. This photography would be taken from approximately 6000 feet above the average ground elevation providing coverage of one mile in width on a single strip and suitable for plotting a topographic map with five-foot contours at a scale of 1" = 200'. This same photography would be used to make an aerial mosaic of the area which would be used along with the topographic maps. Ground control necessary for compiling these five-foot contour maps would then be established in the field and permanently monumented so that it could be picked up and extended at a later date. It is necessary to establish two horizontal and three vertical control points within the area covered by each picture for the compilation of these maps. Before the field control is started an investigation is made to determine all existing government control which is available through United States Geological Survey, United States Coast and Geodetic Survey, and the Corps of Engineers. The primary base line, which is established in the field, is then tied in to the government control and in most areas it is, therefore, not necessary to loop any lines to insure the required accuracy. The field survey necessary in this primary stage usually amounts to one mile of traverse and one and one-half miles of levels for each mile of the highway under study. If the government control is established at close enough intervals, the field work can be based on third order limits.

A more detailed study is then made with the information available at this point to definitely pick out the center line of the proposed highway and make cost estimates on construction and property damage. With this center line established, low altitude photography is obtained from an elevation of twenty-four hundred feet above the ground providing a strip of approximately one-half mile in width and suitable for plotting accurately two-foot contour maps. The scale of these maps is usually between 1" = 50' and 1" = 100', and in some cases, cross sections at regular intervals are plotted in conjunction with the contour maps. In this phase the photography taken at a lower altitude requires more photographs to cover the same area. Additional control is now established by extension of the preliminary line, and again permanent monuments are set at regular intervals. Using this additional control the topographic maps are prepared, which are to be the basis for the design and quantities shown on the final engineering plans.

The Photogrammetric compilation of highway maps ties into the design stage in which all the standard methods of design are incorporated with the exception that all existing planimetric and topographic features and quantities are taken from the photogrammetric maps. When the engineering plans have been completed the advertisement and contract for construction would incorporate an article stating that quantities would be based on photogrammetric maps. The field control, which had been previously established, is used as a base line for staking the highway for construction. After the construction has been completed new photography is then taken so that as-built contour maps and cross sections can be prepared for final pay quantities. In most areas the incorporation of photogrammetric methods in highway design as outlined above speeds up the start of construction by several months and decreases field costs by a very large percentage.

The final accuracy depends on the type of area involved, the vegetation, and the methods employed in the preparation of the final map. In accordance with Standard Map Accuracy ninety percent of all contours should be accurate within one-half the contour interval and ninety percent of the well defined planimetric features shall be accurately located horizontally within one-thirtieth of an inch at final map scale. Several test areas have been run which indicate that quantities based on photogrammetric maps compare with ground surveys within five percent. When speaking of the accuracy of photogrammetric maps it is common for most engineers to think of maps and cross sections prepared entirely from field survey to be one-hundred percent correct. However, anyone who has worked in the field and given some thought to this realizes that ground surveys are certainly never one-hundred percent correct and in some instances could be more inaccurate than the standards set up for photogrammetric methods. One of the main problems encountered in providing National Standard of Map Accuracy is the establishment of picture point control by conventional survey methods.

Before getting off the subject of highway design I want to mention that there are many State agencies and private companies doing research and improving stereoscopic plotting equipment for specific highway problems and in making use of electronic computers to speed up complicated structural analysis and to decrease the enormous amount of time involved in present day methods of computing quantities.

Photo interpretation, another specialty which can be used to a limited degree in connection with the Interstate Highway System, is usually accomplished by comparing soil conditions, vegetation, etc. with areas where known conditions exist. An example of this was the preparation of the preliminary report for an overseas highway from Miami Beach, Florida down to the Florida Keys. The photography taken along the proposed route which passed from island to island and tied in with the existing highway to Key West was very valuable in showing the conditions just underneath the water surface and in certain swamp areas encountered. Aerial photography is also used to classify soil types, Geologists are more interested and more familiar in the classification of large areas for the possible determination of minerals and the best agricultural use of the land. An example of soil classification of this type by photo interpretation was performed by Michael Baker, Jr., Inc. in connection with the report for the design of a power and irrigation system along the Jordan River bordering the countries of Isreal and Jordan. Soil experts worked for several months making detailed tests and classification for various types of conditions encountered in the Jordan Valley. These tests indicated the best agricultural use, the type of fertilization required, and the amount of irrigation water necessary to insure the development of the area into a productive state which would support the investment of the cost of the project and result in the people being self sufficient. After these detailed tests were made on types of soil encountered, photo interpretation was used to classify all soils along the Jordan Valley to the predetermined types. Based on this information a report was prepared summarizing the total amount of farm products which could be grown, the amount of irrigation water needed, and the size of dam and storage reservoir which would be required for the project. This program when completed will convert one hundred fifty thousand acres into productive farm land and cost approximately one hundred fifty million dollars. All this work was performed under the International Cooperative Administration of the United States State Department for the purpose of providing productive land to make the Arab refugees displaced from Palestine in 1948 self supporting.

Aerial photography and photogrammetry have come into common use during the past few years as an aid in the design of practically all civil engineering projects. Design of any project for construction must necessarily be based on a map depicting both the topographic and planimetric features existing in the area. I am not suggesting that it will completely replace field surveys for construction projects, but it certainly is a great aid in the preparation of plans for all projects from the design of a single building to a large dam and irrigation project of the type being carried on in the western part of our country. I will try to explain a few instances where projects have already been completed based to a large extent on maps prepared from aerial photography, not only decreasing the cost of design but affording a much larger store of information from which to obtain a more complete analysis of the problem.

City planning, which until recently, had been based on existing maps and a limited amount of information which could be obtained in the field, now used aerial photography and in many instances photogrammetric maps to more accurately depicted the present day conditions of the area being studied. Normally in the locations where city planning is required very limited source material is available and the cost by field survey to provide the information would be prohibitive.

Property surveys and subdivisions of properties amounts to a large volume of work for the civil engineer and surveyor each year. Photogrammetry will not perform the operation of placing property corners in the field or replace the items which must be done by the field survey party. However, it can be a great aid in the study of property boundaries over a large area. An example of this can be illustrated in conjunction with the tax mapping program presently in process in the State of Pennsylvania as required by the Act of the State Legislature of the 1951 Session of the General Assembly. This Act requires that all fifth to eighth class counties complete a series of maps showing all property lines and dimensions or acreage of each parcel and that these maps be used as a basis for the reappraisal of the county. The purpose of this Act is to provide an equalization of assessment and an up-to-date valuation to permit, in particular, school districts to finance the requirements of education as needed in the areas of new development. Aerial photography provides a complete inventory of all properties and is a necessary tool in performing this type of contract. Our company is presently developing tax maps for three of the larger counties in Pennsylvania totaling an area of twenty-three hundred square miles and involving considerably over one-hundred thousand parcels of land. In the performance of this work enlarged photographs are taken into the field for identification of ownership and visible boundaries. With the aid of these enlarged photographs, stereoscopic plotting equipment is used to compile a complete map of the county showing all visible ground features and tying in with existing government control. With the aid of microfilm copies of deed records, tax lists, and other information available from the County Assessor's Office, these maps are checked to insure the correct location of property lines and ownership for each parcel. A set of linen tracings is then prepared on which is placed an index number for each parcel. This index number corresponds with the standard system set up for the whole county and refers to property ownership cards and assessment cards indicating the value of structures and other improvements. This program has already been completed in several counties throughout the State, and the indication is that other States with an antiquated tax system may follow the same program.

I mentioned earlier that photogrammetry has aided in the development of all types of recreational areas from the large national parks down to small areas set aside for city and county use. Normally, development of these areas must be studied on a large scale map to permit the proper location of artificial dams, lakes, and camping facilities. Normally, a development of this type requires engineering investigation for water supply, sanitary facilities, drainage area, and soil conditions. Here again photo interpretation in conjunction with geological maps can be valuable in determining drainage areas, sources of water supply, and general foundation conditions for large structures. Although large scale maps as mentioned above are not available in most areas, Army Map Service has recently published accurate small scale topographic maps of a large part of the United States. These maps compiled from photography taken at an altitude above thrity-thousand feet can be used for preliminary studies of recreational areas and new highway construction.

Power and utility companies are realizing more and more the advantages of aerial photography to their particular industry. A large power company in the western part of Pennsylvania has been using aerial photography and photogrammetric maps to determine location of cross country power lines, to plot a profile of these lines for the actual location and height of their towers, and to determine the quantity of steel required in their erection. The maps are prepared with all the visible boundaries indicating the property required for purchase or easement. Deeds are also checked and compared to the maps to insure the correct ownership of property. Enlargements of the aerial photographs are a great aid in discussion with the property owners to identify and acquire the necessary easements. It is required that a plan of property to be condemned must be submitted to the court for the condemnation proceedings. This plan showing contours is very readily prepared from the original photography.

An interesting adaptation of photogrammetry to large industries is performing the task of furnishing an inventory periodically of stock piles of raw materials. One company for which we have made several surveys of this type previously had cross sectioned their stock piles by conventional ground survey methods. Since the survey was done by their own crews, a period of approximately two months was required to obtain the field notes necessary in computing quantities. During this period the stock piles were continuously changing as material was used by the plant and new material brought in. It was, therefore, very difficult to obtain accurate quantities at any given time. Aerial photographs give an absolute inventory at the time of exposure even though the computation of quantities is not performed until a later date. On a periodic contract of this type the ground control established for the first survey can be used over and over again for all later inventories. Photographs taken for each survey are permanently on file for a record of the material in stock at any given time.

Photogrammetry is a fast growing type of mapping readily adapted to the work of civil engineers and geologists. Its many phases requires a large investment in equipment and training of personnel. It is not practical for each engineer and geologist to become an expert in performing the operations required to obtain aerial photography and compile maps from this photography. However, each person in responsible charge of design for highway or other construction which requires information relating to the surface of the earth should have a thorough understanding of the principles of photogrammetry and its specific uses in connection with his work. This applies particularly to the highway engineer at this time because of the greatly expanded highway program and the many advancements taking place to aid him in producing a more economical and superior design.

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Highway Geology and the Contractor

by Archer B. Gay
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Highway geology is a subject on which few contractors would care to speak for the simple reason that they have little knowledge of the science itself nor any comprehensive understanding of its ability to make a real contribution to their own cost analysis of a project. Thus highway geology is suffering, along with other technical groups, from a lack of understanding by the highway industry in general. One of its needs is for better public relations. We realize that many scientists have an adversion to this term and to the practice which we call public relations. Many feel that there is no need for it and others feel that it is a form of advertising and is, therefore, not ethical. The fact remains that we have public relations whether we want them or not, it's just a question of whether they are good or bad.

Perhaps we might spend a few moments looking at this subject of public relations in order to engender a better understanding of its functions. It is not, as so many of our professional people think, a form of advertising. Rather, it is a means by which those who are without the door of our own particular area of work are given a better understanding of the part which we play and the contribution which we make to the whole. Perhaps we might coin a better phrase for it and call it public understanding or public acceptance.

In the pursuit of the objectives of a proper public relations program each of us must realize the truth of the statement made by Bernard Baruch, who once said, "every man has a right to his own opinion, but no man has the right to be wrong in his facts." The entire objective of our public relations program must lie in assuring ourselves that every man is right in his facts concerning our function as an indispensable part of the team which makes up the highway industry.

What contribution can the highway geologist make to the highway contractor's knowledge of a particular project and how will it benefit him. There seem to be many of these, though few of them are understood by the contractor and recognized as springing from the science of geology.

Perhaps the first field in which the contractor might expect aid is in that covered of foundation data necessary for bridges and other structures. The accuracy of information and the completeness of the coverage of the sub-surface conditions can have a profound effect upon the contractor's costs and, in turn, upon his bid prices. Will he encounter hard rock at the indicated elevation or will it be necessary to remove a foot or more of soft sap stone before the bearing is satisfactory. The geological report is his best authority in such cases. However, the geologist must realize the necessity for engendering confidence in the accuracy of his reports - so we emphasize again the term "accuracy" as a part of our public relations program.

In areas where the existence of rock is unlikely, piling is the engineer's answer to the problem of a proper foundation bearing for the bridge, the type of material which lies beneath the surface will supply two needed answers in his cost problem. First, how difficult is it going to be to get the piling driven. Second, how long will the piling be in order to secure a proper bearing value to meet the requirements for the structure. Our present method of determining the length of piles lies in driving test piles and relating all other piling to them. A more scientific anaylsis of sub-surface conditions might well serve to eliminate some of the burden of the cost of cutoff losses in piling which is purchased and might also, in time, eliminate the necessity for the test piles. Competition will always serve to channel any saving back to the taxpayer or the owner.

The next field of importance in which the highway geologist can make a fine contribution to the contractor's knowledge is that of general excavation. The type of earth, the character of rock or other similar materials to be moved is part of the essential knowledge which must be had by the contractor if we are to maintain a reasonable price level for this item of our construction. The information under some conditions which prevailed in the past has been largely a matter of judgement of, in the terms of our contractors - "guesstimating".

While it is our practice here in Virginia to bid all excavation as "unclassified" making no difference in the price paid for the movement of a cubic yard of rock or of earth. However, for the purpose of bidding, the contractor is forced to classify the entire project if he is to survive the competitive conditions existing in the industry. Thus, each project is analyzed as to rock and earth and each element is calculated at its cost and averaged into a single unit price per cubic yard for "unclassified excavation."

The method used by the average contractor for making this determination in the process of classifying his excavation is to again rely upon his inate judgement and experience which spring from his experience in the field. Often he looks at adjacent banks, roadway cuts, etc. in his attempt to make his anaylsis. Certainly there is no science involved in the process.

Perhaps this would be a

proper place to discuss the question of the methods employed by the highway geologist to determine the percentage of rock in a given project. Borings, we realize, are about the only fool-proof method of accomplishing this and these may, to some extent, prove to be incorrect if the interval between holes is too great. We also realize that the number of trained men available for this type of sub-surface exploration, when compared to the volume of work to be done, will make this an impractical method of evaluation. However, the geologist must realize that the contractor is gambling his own money and is often a rugged individual when it comes to accepting other methods of evaluation unless he has had some experience with them.

The rapid method and perhaps the one most employed is to use those electrical instruments which perform essentially the same function as the auger and core drill by measuring the resistivity of the material beneath the instrument. A convincing comparison of the two methods generally circulated among the contractors who bid on highway work would undoubtedly do much to improve the general acceptance of the geologists appraisal of the sub-surface materials. This is simply another instance of those public relations of which we have spoken.

The contractor, in determining his costs, needs more than a quantitative analysis of the rock beneath the surface of his project. He must know or assume something of its character. Is it sufficiently hard to require drilling and blasting shovel loading and movement by independent vehicles to other parts of the project, or is it sufficiently soft to permit the use of a ripper and moving it by pans to the point of ultimate deposit. The difference between the cost of the two methods is considerable and the man who makes an incorrect analysis will either lose the work or a lot of money.

In earthwork, especially in mountainous terrain, breakback is a rather important factor. If the sides of cuts break back beyond the line of slopestakes, the contractor may move a lot of material for which he will not be compensated. If the slope is designed for rock and the material proves to be full of planes and will not stand at the steep slope for which it was designed, this also can be a costly factor. It may well be that the contractor and the engineer do not discover this until the cut has been carried down for a considerable distance below the original ground line. In order to build a revised slope, the material must be moved from the higher elevations with considerable difficulty to say nothing of the excessive cost. Geology plays an important part in an accurate appraisal of this type of material before the work is started.

Rock stratification is often the cause of many collateral claims from property owners adjacent to the project. If the character of the stone is such that the shock of explosives will cause it to crack for a considerable distance on each side of the hole, a knowledge of this fact would be of great value to the contractor. He might then protect himself by a different method of loading the holes as well as by insurance coverage against the claims which might range from cracked plastering and broken window glass to the destruction of the building itself.

In many damage cases which actually go to trial in the courts, the highway geologist can be of inestimable value in giving testimony which will enable the court to establish clearly the responsibility for any damage incurred. A knowledge that such expert testimony will be forthcoming will undoubtedly reduce to a minimum the number of claims made which have no basis in fact.

Expert testimony in this field is usually welcomed by those who desire to be fair and just in such matters. Such testimony usually makes a proper impression on both the judge and jury without casting any weight in favor of either the litigants but in giving a clear and factual statement of any unusual conditions and other influencing factors.

Thus far our discussion has covered rather hurriedly the natural elements present in the appraisal of the data available on foundation and earthwork by the highway contractor. There is another way and another area in which the highway geologist can make a substantial contribution to the construction of our modern highways. This is in the area of local materials which are suitable for any part of the construction contemplated and which lie either on the project or in a reasonable circle of its location so as to make their use feasible.

The design standards for the Interstate System of Highways require a large quantity of special material beneath the surface of the pavement and on the shoulders of the road. In many instances all of this material must be produced in commercial plants. In some cases there may be waste material in the form of rock which can be utilized by processing through a portable roadside crushing plant. This can then be combined with the local earth and the finished product will have a sufficiently high California Bearing Ratio to make it usable for a portion of the required special sub-surface reinforcement. This is not a field which should be treated too lightly. The use of such materials can greatly reduce the over-all cost of a project and, at the same time, produce excellent results in service.

There is also the field of local, pit run materials which can often be utilized in the same manner as that outlined for material produced from the waste areas of a project. Tidewater and Piedmont Virginia still have some fine deposits of materials which might be used to advantage and which could produce a worthwhile saving for the citizens of the Commonwealth who pay for our highways. The Geological Section of the Division of Tests of our Highway Department has made some fine contributions to our economic well-being in the discovery of local materials and the evaluation of their use as a part of the available materials for our highway needs. We express the hope that time will permit their continued effort in this field which has been so productive.

The present specifications for the higher types of aggregate do not make the production of these products in roadside plants entirely feasible. The controls which are necessary to produce the desired uniformity of product can hardly be put on a roadside crushing plant without taking from it the element of mobility. Thus the discovery of new sources of material in the area of a project which might be used to produce the required material of the high types needed is not as important as it once was. Nevertheless new aggregate sources are an invaluable asset to every area, for there may be commercial producers who wish to open such deposits and produce a quantity of crushed material in answer to the foreseeable demand, if it be sufficiently large. The geological surveys made of several of our construction districts have proven their value and this is a work which is well appreciated.

One of the chief needs is that of making our contractor understand the contribution which geology can make to his operations in removing from them some of those elements of chance which are so prevalent in his business. Geology will enable him to know the quality, availability and possible uses which might be found for waste materials as well as in the field of other local deposits and their possible combinations for use on the project.

There is still one other need. It is that of asking the geologist to obtain a knowledge of the things which will help the contractor to understand the contribution which can be made by the geologist to his well-being. The end result in either case is the same - better and cheaper roads for our people.

Geology and Transportation Routes

by Robert F. Legget
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Your symposia on the application of geological studies in the practice of highway engineering are now so well and favourably known that it was for me a happy privilege to have been invited to contribute to the proceedings of this, your ninth annual meeting. That this year's symposium is being presented under the joint auspices of, and within the walls of the University of Virginia, makes the occasion doubly memorable. For even though I come from a distant land, I am one of those who hold the memory of Thomas Jefferson in unusual veneration. Not only is he a personal "hero" of mine, if I may venture so to express my respect, but my colleagues and I look upon him as probably the real founder of building research in North America, for reasons that will be appreciated by all who know Monticello. This meeting would most surely have won his approbation, here within the University which he founded and of which he said "This Institution will be based on the illimitable freedom of the human mind."1/

It may not be inappropriate to remind ourselves that, in keeping with his phenomenally wide interests, Jefferson had a lively appreciation of geology, even though this is perhaps not generally realized. Typical of his interest is this extract from his Notes on Virginia, part of a discussion of the fossil shells found on "the eastern foot of the North mountain":-

> "But such deluges as these (one explanation of the fossils then current) will not account for the shells found in the higher lands. A second opinion has been entertained, which is, that in times anterior to the records either of history or tradition, the bed of the ocean, the principal residence of the shelled tribe, has, by some great convulsion of nature, been heaved to the heights at which we now find shells and other marine animals. The favorers of this opinion do well to suppose the great events on which it rests to have taken place beyond all the eras of history; for within these, certainly, none such are to be found; and we may venture to say

farther, that no fact has taken place, either in our own days, or in the thousands of years recorded in history, which proves the existence of any natural agents, within or without the bowels of the earth, of forces sufficient to heave, to the height of fifteen thousand feet, such masses as the Andes." 2/

Those who know Jefferson's writings will recognize in this critical analysis of others' views the inquiring mind of the great President, at work upon what was probably an unfamiliar problem. It is to be noted that these words were written as early as 1781, four years before the first publication of James Hutton's "Theory of the Earth".

In view of this expressed interest in geology, it is more than strange that we find so few references in Jefferson's writings to highways and none at all, so far as my own reading goes, to the very close relation of geology to road building. Possibly this is due to the fact that the road from Charlottesville to Williamsburg, which he travelled so often, does not (as I recall it) present any vivid or striking reminders of the importance of geology to the highway builder. One can imagine him using the long rides along this ancient roadway for the meditations which find expression in his many writings.

I may console myself, therefore, for this strange ommission from the records of one of my masters by the possibility that it may have been when riding along this very road that he thought about the necessary service of scientists from other lands to the fledgling United States of America, a matter to which he refers on several occasions. May I, with all due deference, use his words in defence of my presence here today? Do you recall his writing, when he was a very old man: "I know that our pride and prejudices bristle up at the employment of foreigners, but it is science we want, and to this we must sacrifice our pride and prejudices-----we must meet the difficulty, compromise with it, and make up our minds, with the honey, to swallow the few dregs we cannot separate from it".3/ Your other speakers are providing you with honey; here are the "foreigner's dregs".

For I do come from a land far removed from this pleasant countryside of Virginia, with a climate reputedly so bad that it is probably blamed for the few "cold blasts from the North" to which you may very occasionally be subjected. It would, therefore, be invidious for me to attempt to discuss with you any of the more detailed problems which you encounter in your local highway work. May I, however, and by way of contrast, share with some thoughts about the more general aspects of geology in relation to transporation. Such generalizations can sometimes be helpful, if only in providing a background for more specific and more technical studies of particular problems.

There is, for example, an interesting tale to be told of the profound influence of geology upon the actual selection of early transportation routes. One can see that some of the oldest of the Roman roads followed the routes they did because of the local geology, and probably for no other reason. Think, for example, of the great road over the Julier Pass in the Alps which mounts to the summit in fewer zigzags than the modern road and is even said to be less exposed to the wind with consequent less trouble in maintenance even today.4_/

There is an old military canal in Canada, to give a more modern example, joining our capital city of Ottawa with Kingston at the foot of Lake Ontario. Built between 1826 and 1832, in its length of 130 miles it reflects quite remarkably the geology of the country it penetrates, one stretch (for example) being along the contact between the southern projection of the Pre-cambrian shield and the adjacent Palacozoic rocks of the St. Lawrence Valley. This is the Rideau Canal; I hope one day to be able to study its geology in some detail.5_/ I am sure that all of you can call to mind similar examples of the influence of geology upon early transportation routes.

Inland waterways and pioneer highways had to be located in conformity with local geology and topography in order to ensure suitable water levels for canals and a minimum of construction work for roads. Correspondingly, the first ports were not artificially created but were centres of population which sprang up around naturally enclosed bodies of water, or at sheltered river mouths that provided natural harbours of refuge. The development, use and eventual abandonment of the Roman port of Ostia due to the silting up of the mouth of the River Tiber is a vivid reminder, coming to us across two thousand years of history, of the fact that the great force of nature must be understood and controlled if the works of the engineer are to endure.6_/

It was the coming of the railway in the early nineteenth century, as a worldwide means of swift and economical mass transport, that changed this simple picture of merely accepting the limitations imposed by geology in the locating of transportation routes. With the spanning of the continents by railways, the routes they followed had to be, as closely as possible, the shortest distances between the cities they joined. Great bridges and long tunnels were often the answer to the limitations imposed upon the railway pioneers by the geology of the terrain they penetrated. This called for engineering and construction of a high order. In this way came some of the most notable applications of geology in engineering, many of them fortunately recorded in the annals of civil engineering, for our inspiration and guidance today.

Only within the last two decades, however, has any comparable advance been made in highway location and construction. All those present will know important roads that even today give one the impression that they are but little more than improved cowpaths. But the picture is changing- how rapidly none know better than you who have gathered here to share geological experience gained in the design and building of modern highways. Your great toll roads, such as the superb New York Thruway, have had to meet standards as exacting in their way as those used for main line railways. Even in Canada, the Trans-Canada Highway, now approaching completion as a fully paved road from Atlantic to Pacific, is being built to high standards, so similar to those in railway work that over the most difficult stretch of the route, in the heart of the Rocky Mountains, the new road is actually following the abandoned right-of-way of the Canadian Pacific Railway. This is in Rogers Pass, the old railway route having been given up when the Connaught Tunnel was opened in 1916.

Even tunnels are not unknown to the modern highway engineer. Your Pennsylvania Turnpike made ingenious use of abandoned railway tunnels. Today, you have such cities as Pittsburgh driving tunnels of no mean size to assist in solving their highway traffic problems. In other lands, too, the same thing is to be found. Last summer, when travelling in Norway, I was more than surprised to fine what extensive hard-rock tunnels Norwegian engineers had had to excavate even on such a secondary road as that from Bergen to Nordheimsund, on the justly famous Hardanger Fjord.

Even this Scandinavian work, however, fades almost to insignificance when compared with what French and Italian engineers are now actively planning, this being the construction of the Mont Blanc vehicular tunnel. With test boring complete, construction will soon be underway (if, indeed it has not already started) with completion scheduled for the end of 1960. The tunnel will be 39,032 feet long and will thus be the sixth longest in the world. Designed for two lines of traffic, it will be 27 feet. 10 inches wide and 22 feet. 8 inches high; naturally it will be equipped with artificial ventilation. It is anticipated that it will carry as many as 350,000 cars and 1,500,000 people every year. Passing directly under the Mont Blanc Massif, this great tunnel - when complete - is certain to add another significant chapter to the long record of the geology of tunnels.

Whether constructed for road, waterway or railway, or indeed for any other use, tunnels differ but little in conception or design. Consider, if you will, at the other extreme, a bridge recently completed in Canada which could not be visualized as serving anything but highway needs, even the character of which was determined by geology. This is the new crossing of beautiful Okanagan Lake in the interior of British Columbia, to give convenient access to the town of Kelowna. About 3,000 feet wide at this point, the lake has a bottom such that it gives most unsatisfactory conditions for bridge foundations. Accordingly, a floating reinforced concrete bridge was considered, to consist of ten rigid pontoons 200 feet by 50 feet and 15 feet deep. The fact that the lake water level is controlled within a range of less than four feet made practicable a floating structure. The bridge had to be built, and so the floating structure was designed and is now in service.7/ The Kelowna bridge is not, of course, unique but just the most recent of a small but notable group of floating highway bridges which includes the well-known structure at Seattle over Lake Washington and the outstanding arched-floating bridge at Hobart, Tasmania.

Bridges and tunnels are not, however, peculiar to highway engineering, interesting as they always are to the student of the interrelation of geology and engineering. Are there any aspects of what we so loosely call engineering geology that are unique, or almost so, to highway design and construction? I submit that there are, and although they do not carry with them the dramatic interest of the foundations of great bridges, for example, they are equally important and so warrant careful study. They are all concerned with the links between road structure and the local terrain.

In the first place, the location of new roads was, I believe, one of the first civilian engineering tasks to which modern techniques of aerial photo-interpretation were applied. This branch of highway work is still a very special example of the use of this highly specialized procedure. There is no need to discuss the methods now in use but it is salutory to recall that what started as a simple process of route delineation on unscaled photographs little more than a quarter of a century ago has now developed into a highly precise surveying technique.8-/ The fact that such accuracy in aerial surveying is today possible may tend to obscure the vital necessity for ground reconnaissance and location as an essential supplement to even the best of aerial photographic techniques.

Geology and topography, it is true, can be studied on the grand scale from the air, but, certainly for highway location, patient examination of a selected route on the ground itself would appear to me to be a continuing necessity. Even as we sit here discussing highway location work, tractors and oversnow vehicles are taking groups of engineers and surveyors in to campsites along what will be one of the last (and one of the toughest) pioneer highways of North America. This is the road that the Canadian Government has decided shall be built to link the extremity of the Alaska Highway, at Keno Hill, with the Arctic Coast at Tuktoyaktuk. It will have to penetrate the rugged and largely unexplored Yukon and Richardson Mountains. It will give access also to Aklavik, principal settlement of northwestern Canada, now about to be completely moved from one side of the great Mackenzie Delta to a much better site on the other, eastern side.

It was, incidentally, in connection with the survey for the new site for Aklavik that I learned my lesson with regard to aerial photo-interpretive techniques. Study of aerial photographs of the great delta (an area of 5,000 square miles) had revealed what appeared to be an ideal site for the new town on alluvial fans adjacent to the Richardson Mountains. It was only as a result of test drilling in the permafrost, as a check on this selection, that it was found that the site was underlain by frozen silt and not by the sand and gravel that everyone had anticipated. 9/ The search for the new site had to continue.

Possibly it is because of familiarity with the North that I am so conscious of the fact that terrain conditions are not always what they appear to be at first sight. In northern Canada, for example, there are vast areas of Muskeg, a word used to describe organic terrain. From the air, in summer time, and indeed even when viewed at a distance from the ground, muskeg can look quite beautiful -lush green in colour, compact and uniform in surface texture. Only when one has to traverse it, does its real character begin to reveal itself-usually waterlogged (with natural moisture contents up to 1500%), almost impossible to drain by ordinary methods, and frequently deriving what strength it has only from its surface mat of partially dried organic material.

Road construction over muskeg is therefore difficult work, requiring accurate foreknowledge of terrain conditions which is itself sometimes hard to obtain, so treacherous can be even walking over some types of muskeg. Methods have been, and are being developed for road construction over such terrain. They need not be detailed here, but reference restricted to the fact that this somewhat unusual aspect of geology in highway construction has led to an entirely new branch of terrain investigation - the scientific study of muskeg. Dr. N. W. Radforth of McMaster University, Hamilton, Ontario, has been a pioneer in this field of palaeovegetography but even his good work is overshadowed by what he recently saw in the U. S. S. R., where he visited the Russian peat research institute which has a research staff of 400 and 1600 students in residence.

Another aspect of muskeg of vital importance to the highway engineer in northern regions is the protective effect which shallow muskeg deposits have upon permanently frozen ground, the "permafrost" which is such a problem in Alaska, in northern Canada, in the U. S. S. R. and in Manchuria. If the ground beneath the muskeg is fine-grained soil, and especially so if it is silt, removal of the muskeg- the "obvious" prodedure in road building- can have permanently damaging effects. The muskeg restricts the depth to which thawing will take place by the end of each summer (thus controlling the depth of what is called the "active layer"). If it is removed, soil previously frozen solid can quickly take on the consistency of soup, with results that can readily be imagined.

This is, perhaps, the most vivid example of the effect which highway construction can have upon the terrain which it traverses, and this is surely one of the most important interrelations between geology and road engineering. Troubles with permafrost are just a reminder that the building of a road can have a permanent effect upon the thermal regime of the area covered. Soil moisture conditions are similarly affected, in some cases very seriously. As road building continues to spread over this continent, it is probable that these effects upon terrain will have to be given much more consideration than has been the case up to now. For, as my friend Prof. K. B. Woods has pointed out, approximately 2 percent of the total land area of the United States is already taken up by road surfaces, not all paved it is true, but different enough from the natural terrain to affect profoundly the subsurface regime.

There are locations, naturally, where subsurface conditions have to be deliberately changed in order to permit road construction to proceed. Drainage is the dominant requirement in this respect. The modern use of sand drains in highway work has been a notable advance- although I find myself wondering if it really is such a modern advance when I recall the record I recently came across of the use of sand drains in building foundation work in France as early as 1830. Last summer I was privileged to see what must be one of the most extensive installations of highway sand drains. This is in Finland. The construction of the first divided land highway in this friendly and courageous country, between Helsinki and the ancient capital city of Turku, has necessitated more than 500, 000 metres of sand drains being installed over two relatively short stretches of the route where the road must traverse deposits of unusually sensitive marine clay. As I stood looking at this interesting installation, with its precambrian rocky background, so familiar to me as a Canadian, I thought how vivid an example it was of the application of geology and soil mechanics to the work of the highway engineer, never thinking that I should so soon have the opportunity of describing it to you here in Virginia.

There is yet another aspect of terrain study that deserves brief mention, this being the use in situ of the soil over which a highway is to be located as road building material. Some friends were mildly amused when I included a reference to a pioneer Australian experiment of burning clay in situ- in a publication of 1939.10/ Earlier confidence has been justified by recent accounts of quite extensive experimental work in the treatment of "black cotton" soils, one of the most extensive and troublesome soil groups of the tropical world, by burning it in situ and using the resulting hard material, when broken, as a coarse aggregate in the road bed.11/ We are so profligate in the use of the road-building material, which we have generally in such abundance in North America, that it is only the occasional reference to a shortage of sand and gravel (heard even in parts of Canada) that invites attention to the possibility that the day may come when "manufactured" road building materials may become a widespread necessity.

Not only in the design and construction of highways does the local terrain play a dominant role in highway work, leading to unique applications of geological study some of which we have been considering together, but it is significant also in road maintenance, the least publicised, the least exciting and yet in some respects the most important part of highway engineering. And local geology is still of profound significance.

Geologic influence upon roadbed maintenance and drainage is obviously important. Less obvious is the significance of the geology of the terrain around the highway. In areas of incipient landslides, for example, local geology must be regularly observed. Similarly in precipitously rocky areas, it will often be necessary to study local geological features if maintenance troubles can include rock falls. Signs by the roadside indicating the possibility of danger from rock falls may sometimes be thought to be a reflection upon the adequacy of geological studies in road maintenance, and yet there are many locations where certain protection from rock falls could be achieved only at prohibitive cost. There is at least one location in North America where highway practice has had to follow railroad practice, an electrically actuated warning fence paralleling a particularly vulnerable stretch of road.

In some mountainous regions there are hazards even worse than rock falls to be guarded against in road maintenance. Avalanches sweeping on to highways in areas of high snowfall can exact terrible tolls of life and in property damage. Fortunately, there are few areas in North America where avalanche control work is a necessity but there are some. Notable work has been done in some western states. Canada now faces this problem quite critically since the finally selected route for the Trans-Canada highway through the Rockies goes through one of the very worst possible areas for avalanches. A young Swiss-trained member of our own staff is now stationed at Glacier in the heart of the mountains, at work as research engineer with the Department of Public Works, (responsible for building the road), applying in Canada the accumulated experience of Switzerland, a country pre-eminent in snow and ice research work. In one area of that lovely country, avalanche protective measures include even designing masonry buildings with wedge-shaped ends facing the avalanche slopes.

Having boldly, and without question, included snow as a geological agent I should bring this brief review to a speedy close before making further ventures into debatable territory. But one cannot mention a place with so romantic a name as Glacier without a final reminder that the inter-relation of geology and transportation routes is a reciprocal arrangement. You will recall that well over a century ago William Smith, the "Father of British Geology", gained his remarkable knowledge of the strata of Britain by his work as a builder of canals. Many were the early geological papers that described geology of the country traversed by pioneer railway lines. In more recent years, the same has been true of pioneer roads, a notable example being Charles Denny's paper on the geology of the country along the Alcan Highway.12/ Just as long as there are new roads to be build, whether in unknown country or in the heart of the continent, if the highway engineer uses geology for his good purposes and remembers also that his works may reveal previously unseen exposures of the soils and of the rocks, so will the sum of human knowledge continue to advance hand in hand with the progress of the spreading network of transportation routes covering this vast continent.

<u>References</u>

1. In a letter to William Roscoe, from Monticello, dated December 27, 1820.

2. Part of the answer to Query VI: see pp199-200 of "The Life & Selected Writings of Jefferson" ed. by Koch & Peden 1944.

3. In a letter to Charles Sigourney, from Monticello dated August 15, 1824.

4. "Engineering" by A. P. Gest, in "our Debt to Greece & Rome" series Longmans Green & Co. 1930 p. 131.

5. "Rideau Waterway" by R. F. Legget, Univ. of Toronto Press, 1955.

6. "The Development of Dock & Harbour Engineering" by Sir Cyril Kirkpatrick Vernon Harcourt Lecture, Inst. C. E. (London)1926.

7. "Canada Builds Floating Bridge, Engineering News-Record, vol. 159 p. 46-50 21 November 1957.

8. See for example: "The Appraisal of Terrain Conditions for Highways Engineering Purposes", Bulletin No. 13 Highway Research Board, Washington 1948.

9. "Rapid Disintegration of Alluvial Fan Material near Aklavik, N.W.T." by R. F. Legget and R. J. E. Brown (being submitted to G.S.A.).

10. "Geology and Engineering" by R. F. Legget, McGray-Hill Book Co.,inc. 1939 (see p. 254)

11. "Airfield Construction on Overseas Soils: Part II, Tropical Black Clays, by K. E. Clarke, Proc. Inst. C. E. (London) Vol. 8 pp 223-230, Nov. 1937.

12. "Late Quaternary Geology and Frost Phenomena along Alaska Highway, Northern British Columbia and Southeastern Yukon" by C. S. Denny, Bull G. S. A. Vol. 63 pp. 883-921 Sept. 1952

Soils and Geological Engineering Partners, Not Competitors

by Alfred W. Maner
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The ancient Greek historian, Herodotus, recorded a channel change made in 3200 B. C. in the Nile River below Memphis to straighten a bend in that famous river of history. No one knows how many centuries prior to this great feat man's first contact with soil as an engineering material occurred.

Evidences of drains, tunnels, canals, aqueducts and other kinds of structures which involved soil engineering have been found at sites of early civilizations. Many of these remnants have been found in Egypt, China, and the low countries of Europe.

We know that excellent roads existed in some of the ancient lands. Materials for the Great Pyramid of Cheops were transported over paved highways. The Crete road, from Komo to Knossos was built around 2500 B. C. of rammed earth and broken stone. In our own Western hemisphere the Inca and Maya Indians built fine highways 1500 to 2000 years ago.

The Roman engineers built some of the most famous of the ancient structures which involved the solving of many problems in soil engineering. The great Appian Way and the Roman roads in Great Britain stand out as fine examples of their ingenuity.

After the fall of the Roman Empire nearly all forms of scientific activity practically ceased for a thousand years. As the nineteenth century approached, interest in things scientific began to revive. Although it is recorded that about 500 B. C. Pythagoras remarked upon evidences of the excavation of valleys by rivers thus beginning the growth of ideas regarding the earth, geology, as a separate science, did not originate until the last quarter of the eighteenth century.

The science of geology progressed during the nineteenth century with the organization of geological surveys, the establishment of geological professorships in colleges and universities, and other developments, including the study and classification of minerals, rocks and fossils. It was during this period that geological information was recognized as a real aid to engineering.

Since man came out of the cave, structures had been built on and of the earth materials by trial and error methods. If a structure collapsed before it was completed the builders simply started all over again using larger or different materials. This method of construction continued until the structure stood up. Here was a science that classified the different materials in such a way as to help take the guesswork out of foundation problems. Engineering geology was the natural result of the attraction of the science of Geology to the art and science of engineering.

The year 1925 saw the birth of the science of soil mechanics, the branch of mechanics which deals with the action of forces on soil masses. Dr. Karl Terzaghi "the father of soil mechanics", has stated that the science originated under the pressure of necessity, almost simultaneously in the United States and Europe. Since the soil which occurs at or near the earth's surface is one of the most widely encountered materials in the civil, architechural, and agricultural branches of engineering it is easy to see why necessity sired the science. The myriad failures over the centuries resulting from inadequate foundations could have been prevented had the designers and builders known something of the engineering properties of soils. The tower of Pisa is an ever present example of ignorance of the principles of soil mechanics. Had the builders been able to predict the differential settlement that has taken place the leaning tower would not be the tourist attraction it is today and Galileo would have had to look elsewhere for an easy place from which to make his famous drop tests.

As in all sciences, soil mechanics must be applied to be of practical use to mankind. The application of the principles of soil mechanics to the practice of engineering is soil engineering.

Here, now, we have two specialized fields, engineering geology and soil engineering. What do they have in common?

The engineering geologist is concerned with the earth's crust as related to foundations of structures such as dams, pavements, buildings, and bridges, with sub-surface water, with stability of slopes, with frost action, and many other related problems.

The soils engineer is concerned with foundations of structures, such as dams, pavements, buildings, and bridges, with soil water, with stability of slopes, with frost action in soils, and many other related problems.

It appears, that the engineering geologist and the soils engineer have a lot in common.

Dr. Karl Terzaghi has said, "--the determination of the geological history of sediments has become an integral part of subsoil exploration. Since it is in many instances a prerequisite for the reliable interpretation of boring records, no engineer engaged in the practical application of soil mechanics can nowadays be considered competent unless he has at least a general knowledge of geology. On the other hand, the services which the geologist can render to the engineer are very limited unless the geologist is familiar with the fundamental principles of soil mechanics and their practical implications".

A limited survey, which consisted of examining the curricula in the catalogues of fifteen colleges offering degrees in engineering and geology, revealed that practically all of the engineering colleges require the study of soil mechanics for a degree in civil engineering and a majority have required courses in general geology or engineering geology. Two schools offer degrees in geological engineering. None of the schools offering degrees in geology or geological engineering require, or even suggest as an elective, a course in soil mechanics.

Since this is a symposium on geology as applied to highway engineering let us consider, briefly two specialized specialists - the highway engineering geologist and the highway soils engineer. Working together, these men can perform very important services for the highway departments.

The ways in which the highway geologist and the soils engineer can work as partners in producing design recommendations for a new highway are numerous. They can suggest changes in alignment based on geological studies of the area, they can classify excavation, they can suggest the best methods for control of subgrade moisture, they can recommend the best methods for erosion control of slopes, they can select the elevation of footings for bridges and recommend the type of foundation best suited for the location, and they can offer many other services which will result in a more economical design.

The location of road materials, the control of landslides, the control of subsidence of fills can be done most efficiently and thoroughly through the partnership of the engineer and the geologist. It is true that one or the other can do all of the things enumerated but a much more thorough job can be done if each contributes his specialized knowledge to the solution of the problem.

The Virginia Department of Highways established in 1947 a geological section within the Division of Tests. From one geologist and no equipment in 1947 the geology section has grown to include eight geologists, a draftsman, six drill rigs with operators, two electrical resistivity apparatus, vibration measuring equipment, and other equipment necessary for the highway geologists to perform their duties.

Our geologists duties are manifold. They work closely with the Soils Laboratory in furnishing information for pavement and bridge foundation design. They investigate landslides, make recommendations for stabilizing potential landslides, investigate blast damage claims, locate potential quarry sites, secure information on ground water movement. Electrical resistivity surveys to locate rock profile have proved to be very accurate and have saved the State thousands of dollars in excavation costs.

Our geologists and our soils engineers work in harmony but there is still a competitive spirit between the two.

Why is there an apparent rivalry, however friendly, between the soils engineer and the engineering geologist? Maybe it is because the soils engineer has not studied enough geology. Maybe it is because the engineering geologist has not familiarized himself with the principles of soil mechanics. I think that as a result of the lack of familiarity of each with the other's science the rivalry mostly stems from the difference in nomenclature and, when nomenclature overlaps, difference in definition.

The soils engineer, of course, uses the geologists basic rock names but his definition of rock is not quite the same. To the geologist rock "is the material that forms an integral part of the earth's crust. It includes loose incoherent masses, such as a bed of sand gravel, clay or volcanic ash, as well as the fresh and solid masses of granite, sandstone, limestone, and the like". To the engineer, rock is "considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces."

The engineer recognizes the term stone as indicating a certain kind of structural material. To the geologist there is no such thing as stone.

The geologist concedes that soil is the loose surface material of the earth in which plants grow. Soil, to the engineer, is a structural material and is regarded as a natural aggregate of mineral grains, plus air, water, organic constitutuents, and other substances which may be included therein. Soil has been broadly defined by some engineers as anything that can be moved with a pick and shovel, including man made dumps. This definition will probably seem extremely inane to the geologist but to the engineer who must place a structure on such material, man made dumps present just as much of a problem as do muck deposits.

If the soils engineer and the engineering geologist would get together and each strive to understand the others problems then I believe that most of the rivalry would disappear. How many times have you heard the engineer and the geologist arguing over the definition of soil?

"It is not soil", says

the geologist, "It is disintegrated rock".

"It is soil because it is unconsolidated and can be picked up with a shovel", says the engineer.

And they are both right. According to the nomenclature and definitions of geology the geologist is correct in his statement. The engineer is correct too because the conditions require that the definition be different. For instance, it is more likely that excavation on a project, be it highway, bridge, or building, will be bid at a much lower price if decomposed or disintegrated rock is called soil instead of rock.

Also the unconsolidated materials found in nature possess definite engineering characteristics that are different from their consolidated parent materials. The parent rock may have measurable plasticity, elasticity, and shrinkability, but these properties are of little concern in engineering works. These same properties in soil are very important to the engineer and must be measured accurately.

How can an understanding of each others problems be brought about between the geologist and the engineer? One way would be to start, in all the schools of engineering and geology, to teach the students of geology a course in soil mechanics and the students of civil engineering basic courses in both geology and soil mechanics. We have seen that most civil engineering curricula require both geology and soil mechanics. We have also seen that the schools of geology do not now offer courses in soil mechanics. It is particularly desirable that those schools offering degrees in geological engineering include a course in soil mechanics.

If this were done the geologist, when working with a soils engineer on an engineering problem, would not be prone to dispute the terms and definitions used by the engineer. Likewise, the soils engineer would be more likely to accept the geologists terms and definitions when working on a geological problem.

What about the thousands of practicing engineers and geologists who have not had the opportunity to study these courses in school. Each should be tolerant and endeavor to learn as much as he can about the other's basic ideas, nomenclature, and definitions.

A little understanding will go a long way towards making soils engineers and engineering geologists, in reality, partners, and not competitors.

References

Karl Terzaghi, "Origin and Functions of Soils Mechanics", Transactions, American Society of Civil Engineers, Vol. CT, P. 666 (1953).

Peck, Hanson, Thornburg, "Foundation Engineering", John Wiley and Sons, Inc. New York, 1953.

M. G. Spangler, "Soil Engineering", International Textbook Company, Scranton, 1951.

Ries and Watson, "Elements of Engineering Geology", John Wiley and Sons, Inc. New York, 1947.

A Rapid Method for Determining the Resistance of Ledge Rock to Freezing

by James M. Rice
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Introduction

The invitation to appear on your program was originally extended to Mr. J. E. Gray, Engineering Director of the National Crushed Stone Association. Due to other commitments, Mr. Gray was not able to attend this meeting. The paper to be presented is similar in some respects to one given by Mr. Gray before the 1957 Annual Meeting of the American Institute of Mining, Metallurgical, and Petroleum Engineers, and subsequently published in the Crushed Stone Journal(1).1

Effects of Freezing

The ability of an aggregate to resist the destructive effects of freezing is one aspect of durability. From the highway engineer's point of view this property is particularly important for aggregates used in portland cement concrete. It is of less importance where aggregates are waterproofed as in bituminous concrete, or where the aggregate is used in a base course where the exposure conditions are less severe. Unsound particles of aggregate may expand disruptively in concrete. This expansion may cause deep-seated disintegration of the concrete. The mechanism appears to be that the expansion of the aggregate causes disintegration of the mortar surrounding the expanding particle, and an over-all expansive of the structure. In the case of pavement slabs, the expansive effect may not be immediately apparent since the slab in cold weather is contracted. However, the residual expansion remaining may cause complete closure of all joints in hot weather. Additional expansion due to moisture gain in rainy weather may be sufficient to cause blow-ups or buckling of the slab. Associated with this phenomenon is map cracking and subsequent disintegration of large sections of the pavement (2).

Expansion of an aggregate due to freezing depends upon the pore structure, the degree to which the pores are saturated, the size and strength of the particle, and the rate of freezing. Water expands about 9 percent in volume upon changing from the liquid to the solid phase, and this expansion creates pressure if confined. In an aggregate, pressure will not be developed if the water is able to move into unfilled pore spaces or out of the boundaries of the stone. Water tends to move more freely in pores of large size than in microscopic-sized channels. Restriction of flow in fine pores permits the development of pressures exceeding the strength of the stone. Thus, it is possible that an aggregate with fine pores may expand even though not highly saturated, and an aggregate with relatively large pores may not be affected by freezing when very highly saturated. The critical pore size depends to some extent upon the rate of freezing. Some investigators have indicated that pore diameters of less than 4 or 5 microns are critical (3) (4). A more recent study indicates that pore sizes in the range of 0.1 micron are very critical (5). The size of the aggregate particle may be a factor in that it determines the distance that the water must be moved through the capillary system. It has been suggested that the critical size of aggregate particles would be in the range of an inch to a tenth of an inch (6).

An aggregate in concrete may behave somewhat differently than when unconfined. The hardened paste surrounding the particle tends to retard the egress of water since the paste is relatively impermeable. On the other hand, the paste, having a finer texture, tends to exert more affinity for the water present in the concrete. Thus, upon drying the paste may withdraw water from rock particles, and these particles are not readily resaturated (6,7).

Soundness Tests

Conventionally, the soundness of aggregates is specified in terms of a maximum loss in a sulfate soundness test. The test consists of alternate immersion of a graded and weighed sample of aggregate in a saturated solution of either sodium or magnesium sulfate, and oven drying under specified conditions. The crystallization of the salt in the pores of the aggregate exerts internal forces which may cause disintegration of the particles. The loss in weight is determined after either 5 or 10 cycles in terms of the amount of the aggregate which passes the sieve upon which it was originally retained. This test is subject to criticism in that reproducibility leaves much to be desired and because it does not actually simulate the freezing action of water. In some cases, the soundness test has led to the rejection of highly satisfactory aggregates and the acceptance of poorly performing aggregates (8).

In order to more nearly simulate field conditions, freezing and thawing tests have been investigated. The samples and evaluation of loss are similar to the sulfate soundness, that is, disintegration is measured by loss in weight. In order to develop the expected results, it has been found necessary to extend the exposure for as much as 50 cycles, which is too time consuming unless special automatic equipment is available. To accelerate the test, resort has been made to vacuum-saturation of the aggregate prior to freezing, or substitution of either brine or alcohol solutions for plain water as the freezing medium. Such freezing tests have not gained much acceptance and are of questionable value since aggregates may cause distress in concrete without any appreciable disintegration of the aggregate itself.

A more reliable method for evaluating the durability of aggregates is to subject concrete specimens containing the aggregate to freezing and thawing. Performance in the test may be evaluated by measurements of expansion or changes in frequency of vibration. By the use of air-entrainment, it is possible to protect the hardened cement paste portion of the concrete from the destructive effects of freezing. When so protected disintegration of the concrete may usually be attributed to the aggregate. At present, the American Society for Testing Materials has four concrete freezing tests in the tentative phase of standardization. The NCSA Laboratory has the necessary automatic freezing and thawing apparatus (9) for performing ASTM Designation C291-52T, Tentative Method of Test for Resistance of Concrete Specimens to Rapid Freezing in Air and Thawing in Water. The concept of these ASTM tests has been criticised by Powers (7) on the basis that they merely measure the rate of disintegration and are unusually severe. There is some justification for such criticism, however, it is believed that either a very good or very poor performance in the test is some indication of the field behavior to be expected. When the performance in the test is between these extremes, the significance as an indication of potential field behavior is subject to controversy. One disadvantage of the test is that it is time consuming due to the curing period of the concrete and the length of exposure which usually consists of two to three hundred cycles.

NCSA Core Expansion Test

The core expansion test consists of measuring the growth or linear extension of a cylindrical specimen of ledge rock caused by freezing and thawing.

PREPARATION OF SPECIMENS

Specimens are normally drilled from pieces of ledge rock perpendicular to the bedding planes, if any. Field cores may also be used if of appropriate size. The size should be not less than the maximum size of the aggregate used in the concrete. The diamond-faced core drill used in the NCSA Laboratory produces specimens 1.75 in. in diameter (Fig. 1). The cores are sawed to 2.0 in. lengths with a diamond cut-off saw, and the ends are ground plane against a glass plate using carborundum grit.

Selection of specimens is sometimes difficult due to seams and cracks. The specimens should be representative of the ledge material but open seams and cracks should be avoided since they would probably disappear in the process of blasting and crushing.

Apparatus

The principal item of equipment aside from the freezing apparatus is the core length comparator shown in Fig. 2. The comparator consists of a micrometer dial attached to a column support which is fastened to a base plate. The dial has a range or travel of 0.1 inches and is graduated in increments of 1/10,000 of an inch. Three short posts with slightly rounded tops are mounted on the base plate concentric with the axis of the dial plunger for supporting the specimen. Two small angles are mounted radially to the two rear bearing points to act as a guide for positioning the specimen. A steel cap which has three posts on one side, similar to those in the base, and a collar centered on the other side to receive the dial plunger is placed on top of the specimen. To insure that readings are not affected by disturbances of the dial mounting, a 2 by 1 3/4 in. diameter mild steel reference standard is provided.

Procedure

The core is identified with India ink and three equidistant marks are made on the top face for positioning the specimen in the comparator. The air-dry weight of the specimen is determined, and also the weight in water and surface-dry weight for the purpose of computing absorption and specific gravity. The ovendry weight may be determined after completion of test or a correction factor may be obtained from a separate sample.

The initial or base comparator reading is made after soaking, although a reading may also be made on the specimen before soaking in order to detect expansion due to soaking. Length measurements are made in a water bath maintained at the prevailing room temperature (Fig. 3). The specimen is seated upon the three base points and the steel cap is positioned on top and under the dial plunger. Readings are made at three positions, recorded, and averaged. The steel reference standard is read before and after each group of specimens. Specimens are wrapped in aluminum foil while in the freezer in order to retard evaporation of water (Fig. 4). The specimens are frozen in air at 0 F and thawed in 40F water. Prior to acquisition of the automatic freezing apparatus only one cycle was obtained daily, but now it is possible to obtain ten or more cycles daily. Cores are periodically removed from the freezer and measured for expansion. The exposure is usually terminated at 50 or 100 cycles, or sooner if the measured expansion exceeds 0.1 percent. Presently, a rate of expansion greater than 20×10^{-6} in. per in. per cycle (0.1 percent in 50 cycles) is considered excessive. Therefore, 50 cycle test results can be obtained in 5 days exposure. Allowing two days for preparation of specimens, it follows that results can be obtained in a week.

Effect of Degree of Saturation & Absorption

The saturation condition of the specimens is a controversial factor. Should the specimen be saturated by a period of water immersion which will probably not result in complete filling of permeable pore space? Or should it be highly and practically completely saturated by vacuum? Our thinking is that vacuum saturation is not realistic unless a stone is very highly saturated when quarried and used without any drying such as might be obtained by stockpiling. However, for very severe exposure conditions, where the concrete will be in prolonged contact with water under hydrostatic pressure, vacuum-saturation is believed to be justified. If the concrete is to be subjected to alternate wetting and drying as in a pavement, the specimen should be immersed in water for 24 hours.

Fig. 5 shows the effect of the initial saturation conditions on the expansion of eight ledges which are characterized by excessive expansion when highly saturated. [1] These ledges are from three Midwest limestone quarries which have been reported to be sources of aggregates with poor performance records. The textures of these ledges were fine and free from macropores. All of the vacuum-saturated cores showed high degrees of saturation and very rapid expansion - more than 0.1 percent in ten cycles or less. Companion cores, when only partially saturated by 24 hour immersion performed much better. Two of these cores (Nos. 1 & 3) eventually failed but the remaining six were sound after 80 cycles even though they became more highly saturated. These results would tend to confirm the observation that highly saturated, absorptive aggregates are not durable. However, there are notable exceptions such as are illustrated in Fig. 6. These ledges represent two Midwest limestone quarries that have good service records. Rocks in this group were coarsely textured and contained some macropores. These cores had relatively high degrees of saturation and high absorptions, yet they performed well in the expansion test.

The examples illustrated in Fig. 6 were selected to show that high levels of total porosity or absorption are not necessarily indications of poor performance. It can also be shown that rocks with low absorption may be nondurable. Fig. 7 shows the expansion curves for several ledge cores with absorption values ranging from 0.27 to 1.69 percent (by 24 hour immersion). These cores represent ledges from two limestone quarries in the Northeast. The limestones may be classified as argillaceous and carbonaceous. The textures were very fine and free from macropores. Comparison of the 24 hour and vacuum absorption values shows that these ledges were only partially saturated by 24 hour immersion, yet each of these ledges expanded rapidly. It is suspected that clay minerals may have contributed to the rapid failure.

Correlation with Other Tests

Mr. Gray's paper, previously referred to, included a comparison of core expansion results with sodium soundness losses for 38 samples of ledge rock. He concluded that "A study of the data reveals the lack of dependability of the sodium sulfate test to detect this type of unsoundness."

Additional information about the expansion test has been acquired in conjunction with a cooperative investigation of aggregates conducted by Sub-committee II-g of ASTM Committee C-9 (Concrete and Concrete Aggregates). This Subcommittee is interested in research on the pore structure of aggregates. Seven samples of ledge rock were distributed to several laboratories. These laboratories made certain measurements including the linear expansion due to freezing, porosity, internal surface area by nitrogen sorption (10), and permeability coefficient. Available results of this program are listed in Table 1. The data show a good correlation between expansion and porosity for these particular samples. The internal surface area measurements are also in agreement with the expansion values. The correlation with the permeability coefficients is less satisfactory.

The NCSA Laboratory is presently investigating the correlation of the core expansion test results with expansion of concrete specimens containing aggregate crushed from the same ledges. For both tests the stone was vacuum saturated prior to test.

These tests are not yet complete, but some preliminary observations are interesting. Of the six ledges involved, three showed high expansion in the core test, and of these three, two showed high expansion (after 100 cycles of freezing) when used in concrete. However, the rate of expansion of the concrete is much lower then than that of the cores. This could be due to the cushioning effect of the mortar surrounding the aggregate, and also to the size of the aggregate. The aggregate for the concrete graded from 1 1/2 to 3/8 inch in size, whereas the minimum dimension of the core specimens was 1 3/4 inch. The critical size for these particular ledges may be less than 1 3/4 inch. For example, if the critical size were 1 inch, particles smaller than this

Table 1. Correlation Results

Sample No.	Type of Stone	Linear[1] Expansion Percent	Permeable Porosity, Percent	Internal[2] Surface Area- m^2/g	Permeability[3] Coefficient, Millidarcys
1	Dolomite	1.54	16.9	6.05	2.8×10^{-2}
2	Dolomite	0.36	9.7	3.60	1.3×10^{-3}
3	Diorite	0.00	0.4	0.01	3.5×10^{-7}
4	Granite	0.01	1.2	0.14	1.4×10^{-3}
5	Graywacke	0.08	6.0	2.60	7.3×10^{-4}
6	Quartzite	0.00	0.5	0.00	1.0×10^{-3}
7	Quartzite	0.00	0.4	0.00	1.1×10^{-5}

1 By NCSA Core Expansion Test, 100 cycles after prolonged immersion

2 Tests performed by National Bureau of Standards (8)

3 Tests performed by Portland Cement Association (10)

size would be unlikely to expand when frozen. The third of the three ledges which showed high expansion in the core test has not shown significant expansion in the concrete specimen. The reason is not yet apparent but it is suspected that the core specimens contained a single seam of frost susceptible material - a weak plane - which caused expansion and failure by cracking in the core test. This seam was apparently eliminated in the crushing of the ledge for the concrete aggregate. This phenomenon tends to emphasize one of the limitations of the core expansion test.

Summary

The NCSA Laboratory has performed this test on many specimens representing various ledges from several quarries. The method seems particularly useful for detecting the presence of non-durable ledges in a quarry. One advantage of this procedure over soundness tests in which only disintegration is measured is that it will detect aggregates which may expand without apparent fracture. In several instances we have been called upon to investigate quarries which have stone that is apparently sound as measured by sulfate soundness and unconfined freezing tests. Yet the stone has allegedly caused excessive expansion and blow-ups in concrete pavements. By detecting the expansive ledges, we have been able to recommend selective quarrying so as to eliminate the offending stone. Thus, quarries that would have had to curtail their activities, or perhaps shut down, have been able to continue in operation.

References

1. Gray, J. E., "A Rapid Method for Determining the Durability of Ledge Rock," Crushed Stone Journal, Sept. 1957.

2. Woods, K.B., Sweet, H.S., and Shelburne, T. E., "Pavement Blowups Correlated with Source of Coarse Aggregate," Proceedings, Higaway Research Board.

3. Sweet, H.S., and Woods, K.B., "A Study of Chert as a Deleterious Constituent in Aggregates," Engineering Bulletin, Research Series No. 86, Purdue University, Sept. 1942.

4. Blanks, R. F., "Modern Concepts Applied to Concrete Aggregates," Proceedings Am. Soc. Civil Engrs., Vol. 75, P. 441 (1949)

5. Lemish, J., Rush, F.E., and Hiltrop, C.L. "The Relationship of the Physical Properties of Some Iowa Carbonate Aggregate to the Durability of Concrete", Proceedings, Highway Research Board, Vol. 37, (1958)

6. Powers, T. C., "Significance of Tests and Properties of Concrete and Concrete Aggregates," Special Technical Publication No. 169, Am. Soc. Testing Matls., (1956)

7. Powers, T. C., "Some Basic Considerations Pertaining to Freezing-and-Thawing Tests," Proceedings, Am. Soc. Testing Matls., Vol. 55(1955)

8. Bloem, D. L. "Significance of Tests and Properties of Concrete and Concrete Aggregates," Special Technical Publication No. 169 Am. Soc. Testing Matls. (1956)

9. Rice, J. M. "NCSA Laboratory Acquires Automatic Freezing and Thawing Apparatus," Crushed Stone Journal, Sept. 1954.

10. Blaine, R. L., Hunt, C. M., and Tumes, L. A., "Use of Internal Surface Area Measurements in Research on Freezing and Thawing of Materials," Proceedings, Highway Research Board, Vol. 32 (1953)

Footnotes

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1 Numbers in Parantheses refer to the references appended to this paper.

Page 27
1 The saturation values reported in this paper are based on the total pore space as determined by the bulk specific gravity and the specific gravity of powdered material.

Recent Developments in Soil Sampling and Core Drilling

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Diamond core drilling had its beginning in Europe. In 1863, a Swiss engineer conceived the idea of using diamonds in an annular ring for drilling blast holes in rock. The first machine was hand operated and the bit drilled a hole approximately 1-5/8" in diameter, and recovered a core of approximately 1-3/16". Next, a steam powered rig was used in the driving of the Mt. Cenis Tunnel between Italy and France during the year 1864. This unit had a bit speed of 30 RPM and a penetration rate of 10-12 inches per hour. In 1867, M. C. Bullock was issued a United States patent for a steam driven diamond drill. This machine operated at 250 RPM and was used to drill a 750 foot hole in search of coal near Pottsville, Pennsylvania in 1870.

Steam was the chief source of power for diamond drills until the period immediately following World War I, when the gasoline engine came to the forefront.

The early single-cylinder gasoline engines were put to use driving belt driven rigs. Diamond drilling had taken a long step forward. For the first time here was a rig that was more easily portable than the heavy and cumbersome steam rig with its high pressure boiler. Multiple cylinder gasoline engines soon replaced the old "Hit and Miss" types as users demanded more power and smoother operation. The prototype of the present day core drill with its multiple speed transmission and wire line hoist was developed about 1938.

The present day drill machines have changed very little in the past twenty years. The first major change involved the swivel head.

This is the part of the machine that provides both the rotation and the downward feed of the bit. All of the early drills had either a hand feed or mechanical screw feed arrangement. This was replaced at first by a water operated hydraulic swivel head and later by an oil operated hydraulic feed. The oil operated system required a high pressure constant volume variable pressure oil pump with a reservoir and control valves. Screw feed heads are still being used for some operations. However, the infinite degree of control over the downward rate of feed has made the hydraulic swivel head the most desirable type. It has been our experience that diamond bit life is materially increased by the smooth feed action of the hydraulic swivel head. With the development of the closed hydraulic oil circulating system other applications for the hydraulic power came into being. Hydraulically operated derricks; retracting cylinders for moving the drill assembly on and off the hole; hydraulic chucks; and the latest development, a hydraulically operated "free fall" drive hammer. All of these devices have been aimed at reducing operator fatigue and increasing the actual drilling time during the 8 hour shift.

In their search for other ways to reduce operating costs many users turned a critical eye in the direction of the diesel engines. Despite the higher initial cost of the diesel unit, compared to a gasoline engine of equal horsepower, the diesel promised substantial economy in fuel costs and reduced maintenance to offset the larger capital investment. Recently one of our customers reported operating a diesel driven rig for a full 8 hour shift on 5 gallons of fuel.

In many drilling jobs, especially in the foundation investigation phase of highway work, more time is consumed in getting the drill rig to the hole site than in actually drilling the hole. The skid type rig, although the most widely used and generally most flexible, was not ideally suited for the continuous moving and shallow hole drilling generally found in this type of work. To combat the increased costs associated with frequent moves the trailer mounted core drill was developed. Generally, this is a two wheel pneumatic tire trailer with provisions for a derrick. Screw jacks are provided to level the drill platform and stabilize the unit. Recently we developed a low bed trailer for use with a standard skid mounted rig. In this set-up we have achieved the portability of the conventional trailer mounted rig and still retained the flexibility of the skid mounted unit. The advantage of the low bed unit is that if the hole location can be reached by the trailer, the drill remains on the trailer and the hole is drilled from the mounted position. On the other hand, if the hole cannot be reached by trailer, the drill is dismounted and skidded "on location".

In some areas of the country most drill hole locations are easily accessible by truck. This was a logical reason for the development of a truck mounted drill rig. The size and depth of hole and the type of drilling generally dictate the size of the truck mounted unit required. Fourwheel drive trucks have proved very satisfactory in moving over rough terrain.

Recently we developed another completely different type of mobile drill. A Fordson Major Diesel tractor provided both mobility for the drill unit and power for the drilling operations through the power take-off. Two such units are currently in the field and the purchaser reports that they have surpassed their fondest expectations. We think that there is a good market for this unit. The drill can be easily dismounted from the trailer and temporarily replaced by a back hoe, or other rear end attachments. Or, the drill can be left in place and attachments, such as a front end loader, added to the unit. This feature should appeal to a contractor, who already has a tracter, and would like to have a drill unit.

In a study of the major costs of any drilling operation, it is readily apparent that direct labor charges, plus supervisory costs, constitute the largest single item. Second to labor is the cost of diamond bits. Actually the entire time allotted for this paper could easily be spent in a discussion of diamond bits. Sprague & Henwood was the pioneer in the development of "oriented" bits. In this process, the individual diamonds are oriented in the bit, in such a way that the hardest part of the diamond is in contact with the material being drilled. Recent studies have indicated that three major factors influence the performance of a diamond bit:

1. The grade of diamonds.

2. The size of diamonds.

3. The type of matrix.

Orientation of the diamonds, together with the right combination of the three major elements of the bit, must be worked out in order to give the user the best bit for his particular type of drilling. When this is done with care, and followed by field testing, the final result can be a lower cost for each foot of drilling. For many years, Sprague & Henwood has maintained a consulting service on diamond bits to assist the user to either reduce his cost per foot, if this is at all possible, or to hold his cost at the lowest practical level.

I would like to mention that at this moment we are actively engaged in the development of Masonry diamond bits, and a special machine for taking concrete cores in connection with highway pavement testing. The use of Masonry diamond bits for drilling holes in ceramic tile, stone, brick, plastic, or reinforced concrete, has been expanding rapidly.

The newest development in drilling accessory equipment other than sampling devices, is the "W" series drill rods.

The Diamond Core Drill Manufacturers Association was responsible for the development of this new

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series of rods. The "W" designation was selected to represent "World Wide", as it is hoped that the specifications for these rods will be adopted on a world-wide basis. The chief advantages of the "W" series rods are:

1. The rods have a larger cross sectional area. This means a more rigid drill string with less tendency to deflect. The closer fit in the hole tends to lessen vibration resulting in reduced wear on the machine.

2. Improved thread design makes it easier to couple and uncouple the drill string.

3. Larger openings in the couplings allow more water to reach the face of the bit. This is important in preventing the bit from being burned, and also to facilitate the removal of the cuttings.

4. The larger outside diameter of the rods means a smaller annular space in the hole. This means an increase in velocity of the discharge water because of the smaller space, the increase in velocity results in better cutting removal.

Without a doubt, the most recent developments have been in the soil sampling or foundation investigation phase. To get the sample so that it can be examined and tested is not always as easy as it would seem. Gone are the days when all that was necessary was to drive down an open end pipe, fill the end with whatever was at the bottom of the hole and bring it up for examination. This was, of course, disturbed sampling in its crudest form. Today the emphasis is more and more on the recovery of a sample in as nearly its natural state as possible.

Each type soil has its own characteristics and its own sampling problems. For some time we have had the well known split barrel samplers, solid barrel samplers and other modifications of this basic type. These are all driven into the soil by a drop weight. Plastic materials on the other hand required a different approach, and we have seen the development of the "thin wall" Shelby tube and Piston type samplers, both of which are designed to be pressed into the material to be sampled.

A modification of the original thin wall piston type sampler was recently marketed under the name of Greer & McClelland Hydraulic Piston Type Sampler. This sampler is forced into the soil in a semi-explosive manner by means of hydraulic pressure. Water under pressure is forced down through the drill rods. When sufficient pressure is built up, a pin shears and the pressure drives the sample tube forward. This method eliminates the need for the extra line of piston rods required with the earlier type sampler.

Another item of interest

is the 1" Retractable plug sampler. This highly portable, self-contained sampler is designed to be driven by a hand operated drive weight. In actual operation the sampler is driven down to the depth at which it is desired to sample. The plug in the end of the sampler is withdrawn and the sampler is driven ahead for approximately 36". The sampler is then withdrawn and the 6" brass sample tubes containing the soil are removed. The sampler, as its name implies, recovers a soil sample 1" in diameter. It has a practical limit of about 50 feet in depth.

Soils Engineers have long considered the possibility of some method for accurately measuring the bearing strength of soil in situ. This possibility has now become a reality in the Vane Shear tester. Vane shear testing is a mechanical method of measuring the torque required to shear a cylinder of soil place. On the Sprague and Henwood Vane Shear Tester the force measuring instrument is a Dillon Strain gauge, which works on the principle of measuring the deflection of a cantilever for load calibrations. The strain gauge measures the input force on a known movement arm and the torque is computed in inch/pounds. By using a prepared chart of applied torque we can determine the shear strength of the soil. The handle, or crank, of the tester is designed to turn at 60 RPM causing the Vane to revolve approximately .2 of a degree per second. The high reduction between the input handle and the vane, together with the slow speed of rotation of the vane, reduces the input force to a negligible amount. This was done deliberately to reduce human error insofar as possible.

The sampling device, which has currently aroused the greatest interest, is the Swedish Foil Sampler. In order to better appreciate the advantages of the foil sampler it is necessary that we review the limitations of other samplers.

Generally speaking, the properties of soil change much more rapidly in the vertical than in the horizontal direction, and, thus, it is logical to assume that the spacing of samples in the bore hole is far more important than the spacing of the bore holes themselves. The fact that often a very thin layer of soil may be of tremendous importance dictates that the samples of soil be taken at relatively close intervals, or, as often is the case, continuously.

In present day, continuous sampling operations using the piston or "Shelby" samplers, while referred to as continuous, in practice, it is never just that. Accuracy in the measurement of the depth at which a sample is taken is not constant. It is very easy to have an error of an inch or more for each sample taken. The accumulation of such errors can present an incorrect picture as to the actual depth at which one particular sample, or portion thereof, was recovered. Another problem is that the lowest part of a sample may fall out and get lost during the withdrawal of the sampler. It can be seen that every soil layer which comes between two samples may appear on the boring log to be a few inches thicker or thinner than it really is. A thin layer may, under some conditions, entirely escape detection.

The upper and lower portion of each sample taken with conventional samplers is generally disturbed. Only the middle section can be considered as undisturbed. The upper section of the sample is injured by torsion, or tension, when the previous sample is separated from it. Additional damage is done by friction against and adhesion to the inside of the sampler when the top section is forced through the entire length of the sample tube. Furthermore, the lower part of the sample is also damaged by torsion, as it is separated from the subsoil.

When a conventional type of sampler is forced into the ground, the tube will, to a degree, pull the core of soil formed inside it downward. The downward movement of the sample is a result of friction and adhesion on the inside of the tube. As a consequence, part of the soil immediately below the cutting edge of the sampler will be forced away laterally instead of entering the tube. The extent to which the soil is forced away from the mouth of the sampler will depend largely upon the softness of the material, and the length of the harder core already formed in the tube. It is understandable than that the natural strata represented by the core will be present at a thickness reduced to an unknown extent, and with a structure disturbance to an unknown degree.

The main disadvantage of existing samplers is that they can take only relatively short samples. This is due to the slide resistance between the core sample and inside wall of the sample tube. As the length of the sample increases, the slide resistance becomes greater with increasing excess pressure at the cutting edge. When the safe length of sample is reached, the excess pressure is so high that part of the soil, under the mouth of the sampler is pushed aside.

The addition of the tight fitting piston to the piston type sampler has aided materially in increasing sample recovery because the suction created assists in keeping the sample in the tube during withdrawal, and at the same time reduces disturbance of the sample to a marked degree.

In some cases, samplers have been given a small inside clearance by turning in the cutting edge of the tube. The premise was that this increase in the clearance between the cutting edge and the inside of the tube would reduce the slide resistance by reducing the pressure in the sampler. However, it must be kept in mind that the total pressure, which in the first place is reduced, has no appreciable influence on the slide resistance.

The basic principle and the one which has contributed to the phenomenal success of the foil sampler, and the one from which it derives its name, is the method of insulating the core, or sample, from the inside of the sampler by means of a number of thin axial strips of foil. The upper ends of these foils are attached to a piston placed in the sampler immediately above the core. The piston, which, by the way, is not tight fitting, is attached by means of a chain or a rod to the driving scaffold at the surface. In this way the piston, together with the foils, are kept on a constant level while the sampler is forced down. During the advance of the sampler, the inside walls slide against the foil. The resulting friction causes a pulling force in the foils and their anchorage, but does not affect the core. There is no slide whatsoever between the core and the foils which encase it. If for some reason that core should tend to more downward or upward, this movement is immediately prevented by the friction and adhesion between the foils and the core. Thus, the recovery ratio is automatically kept equal to 100%. It can be seen that the friction against and adhesion to the surface of the core, which are the main causes of difficulty in other samplers, have in this way been turned into useful forces, which keep the original thickness and structure of each soil layer unchanged on its way into and upward inside the sampler. During the withdrawal of the sampler, the foils follow the movement and each part of the core hangs on its part of the foils by friction and adhesion.

The lower end of the sampler is shaped into a sharp cutting edge. This bottom portion is a double wall construction, providing a magazine for the foils. The foil passes from the magazine through a horizontal slot in the inner wall into the interior of the sampler and runs from there upward between the core and the sampler wall to the piston. As the sampler is forced down, each coil of foil unwinds and covers a portion of the core as it enters the sampler.

The sampler has 16 rolls of foil 7/16 inches wide, and the foils cover nearly the whole circumference of the core. Foil thicknesses of .0025 inch to .008 inch are available. The maximum length of the foils vary according to their thicknesses but approximately

70 feet of .006 inch foil can be inserted.

In a stress free state, the foils lie flat in the sampler. They are, as a rule, bent laterally as they pass from the magazine into the inside of the sampler. This bending exerts an elastic radial pressure on the core. The lateral elasticity of the foils serves as an excellent regulator of the lateral stress in the core and, hence, also of the axial stress in the foils. Without these regulators, even a small swelling of the core, or small variations in the inner cross-sectional area of the tube would cause the foils to get stuck and even break.

It is well to note that at no time is the sampler revolved for even a part revolution--not even to break the core. Torsional movement of any kind would tend to quickly break the foils.

The construction of the sampler tube is somewhat unique. The tubes are connected by couplings having right hand flat threads. The couplings are split longitudinally into two halves kept together by an upper and lower conical ring, locked in position by set-screws. Due to this design, the tube sections can be disconnected from one another without turning. After the sampler is withdrawn, the coupling can be removed, the foils cut, the sample or core severed, and both ends of the tube capped for shipment to the laboratory. The foils are not cut off flush with the upper end of the tube. This provides a means of refastening the foils to an auxiliary pulling device in order to remove the core from the tube for further testing. It should be noted that this method of division prevents the core from subsiding within the sample tube.

After considerable experience it has been found that this new sampler will give the user much better information about the ground in one operation than that obtained by any other previous method in a reasonable number of operations. Coring with the foil sampler is more expensive than intermittent sampling with very wide spacing, but generally, it is less expensive than intermittent sampling with close spacing. At the present time, it is difficult to establish a break-even point between the cost of continuous sampling with the Foil Sampler as compared to intermittent sampling with other methods. Local conditions will greatly determine the value of foil sampling. Where important structures are involved, a wide spacing of samples is out of the question, and for many reasons it seems clear that the new sampler will work faster, and at a lower cost, than the other types.

Where a thin layer of soil could be fatal, it must be searched for continuous sampling and here is where the Swedish Foil sampler excels.

Ammonium Nitrate As an Explosive

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Ammonium Nitrate has been used as an ingredient in explosives for many years. During the last several years Ammonium Nitrate has been used as an explosive without compounding and packaging. The large tonnages of this product produced for fertilizer use provided a ready and inexpensive source once the technology was advanced enough to take advantage of its availability and low cost.

Ammonium Nitrate, in the past, has been used as an explosive in two ways:

1- As an extender and gas former to supplement a high explosive, such as TNT or nitroglycerine.

2- As a low-speed, low-cost explosive.

In the second instance Ammonium Nitrate was mixed with carbon black, sulphur, sawdust, or other such combustibles to form more gas from excess oxygen available in an Ammonium Nitrate decomposition reaction. When this second type of explosive is detonated, the speed of detonation is approximately that of Ammonium Nitrate.

A totally different manner of using this product as a high explosive is now being practiced. When fuel oil (approximately 5% wt.) is mixed with Ammonium Nitrate, the decomposition reaction is changed. The fuel oil is apparently in such intimate physical contact with the decomposing and reacting Ammonium Nitrate that it enters into the primary or first decomposition reaction.

We draw this conclusion for two reasons:

1- The speed of the decomposition reaction is raised from 2,000 M/sec. for Ammonium Nitrate plus dry additives to 4,000 M/sec. for Ammonium Nitrate and fuel oil.

2- The self-propagation limit can be lowered from 9 inches minimum diameter to approximately 2 inches diameter when fuel oil is used.

This property of fuel oil to change Ammonium Nitrate from a low-value explosive to a high-value explosive at virtually no extra cost explains the popularity and acceptance of the method. This product and method combination is now widely used in construction work, mining and quarrying.

The term "overdrive" describes that property of explosives which enables them to detonate at a rate greater than their self-propagating detonation rate. If, for example, a mass of explosive which detonates at 7,000 M/sec. is set off in contact with another explosive which detonates at 4,000 M/sec. then this 4,000 M/sec. explosive will decompose at a rate higher than 4,000 M/sec., but somewhat lower than 7,000 M/sec. for a given distance. Therefore, when Ammonium Nitrate is used, as much of the Ammonium Nitrate must be as close to a higher rate detonator as possible, especially in small diameter holes. This is done by attaching the detonator to the primacord or dropping a cartridge of dynamite into the hole at definite intervals until the proper percentage of detonator by weight is obtained. The preferred method is to use a continuous strand of heavy primacord designed specifically for this purpose of overdrive detonation.

If we use this principle to detonate Ammonium Nitrate, we can take the detonation rate from approximately 4,000 M/sec. to over 6,000 M/sec. Hard rock usually needs a shock of approximately 5,000 M/sec. in order to shatter well. For maximum value from Ammonium Nitrate and fuel oil, an overdrive detonating system should be used. This method of detonation gives excellent results in a variety of strata from granite or hard limestone to soft shale. We have shown that by varying primer strength and percentage of primer by weight, Ammonium Nitrate performance can be varied to act characteristically as the primer acts. That is, Ammonium Nitrate will give results comparable in most respects and better in many, than a hole shot with all primer. By using a continuous column detonator such as a heavy Primacord,* the entire mass of explosive detonates at the highest possible rate.

A great many explosives users categorized explosives into fast explosives with low gas production or slow explosives with high gas production. To some extent, this was a valid working premise. However, the categorization by these characteristics has caused some confusion when Ammonium Nitrate is described.

Ammonium Nitrate can be detonated at a high rate, and this high rate enhances its already unusual gas production, since the speed of the detonation largely determines its peak temperature and the temperature determines the volume. Ammonium Nitrate and fuel oil detonated at a high rate therefore provides both exceptional shattering power and the lifting action of a high gas production desired in most blasting operations. The explosives user is able to obtain for the first time this unique action in an explosive which delivers the best function of both types of fixed explosives. By using this very inexpensive but effective detonating method developed by Monsanto Chemical Company, the diameter of the hole ceases to be a limitation on the use of Ammonium Nitrate and fuel oil mixtures. We do notice, however, that the larger the mass of nitrate, the more efficient the detonation. We suggest the use of this system in holes down to approximately 3 inches in diameter.

Monsanto has done substantial work on those physical factors which affect explosive efficiency and has designed its product to take advantage of the optimum in physical properties. Specific physical properties of Ammonium Nitrate prill which give definite value are: Prill size distribution range, Average prill size, Oil content, and Density of the prill mass. It can be safely said that density (per se) is not to be desired. High mass per unit volume is of value in fixed explosives especially in smaller holes for the purpose of obtaining maximum work from each foot of hole drilled. In fact, it is very difficult to obtain reliable detonation from dense Ammonium Nitrate in oil mixtures. A relatively light, porous prill will perform substantially more work than a dense Ammonium Nitrate. In Ammonium Nitrate and oil mixtures there is a definite efficiency curve determined by the prill surface, to available air, ratio. Specific prill size is not as important as the prill size distribution range. While there is an optimum prill size that we are capitalizing on, the largest gain in explosive performance comes from taking out those sizes over and under the optimum prill size range. A uniform, relatively small prill seems to be optimum for explosives use. We have found that the amount of fuel oil retained by the prill is very important. There is a rather sharp curve peaking at 5% by weight of fuel oil calculated on the weight of prilled Ammonium Nitrate. Fortunately, most prilled nitrate will retain just about this amount of fuel oil if it is wetted by 10% fuel oil and then allowed to drain for a half hour or so before being poured into the hole. There is a definite damping effect on the explosion if 10% or more of fuel oil is trapped in or retained on the nitrate at the time of detonation.

The explosives users now utilizing the data just discussed have shown marked savings over all other systems of blasting. The Ammonium Nitrate is used to full advantage in that more work is performed per dollar spent for explosive material. We believe this method and product combination utilizes to the fullest extent the potential value of Ammonium Nitrate.

In order to prevent possible mishandling of such a common chemical a review of some basic safety factors pertaining to Ammonium Nitrate fertilizer is as follows:

> Ammonium Nitrate should be handled much as you would handle materials such as gasoline. By this, we mean that under all normal circumstances and due care, the material is safe to handle.

Under conditions of extreme heat, (several hundred degrees F.) Confinement, or open flame, Ammonium Nitrate can be dangerous.

Ammonium Nitrate is not particularly flammable, but burns intensely when set on fire.

It should be stored in a building with good ventilation.

It should never be stored in a building capable of confining gases, such as a stone or concrete building with small windows.

It must never be contaminated with unknown materials or with materials not specifically recommended by the manufacturer of the nitrate. This is extremely important as a number of rather common substances may cause unpredictable reaction conditions if mixed with Ammonium Nitrate.

The dangerous mass limit of Ammonium Nitrate is 123 tons. At this level a fire can spontaneously change to a detonation. We, therefore, suggest that each and every stock of 40 to 50 tons be separated by at least six feet of space and a light metal partition such as corrugated iron sheet.

This material can produce relatively toxic oxides of nitrogen while burning. An automatic sprinkling system with an overhead storage tank for water is advised, or at least some hose with a pressure great enough to reach the fire without unduly exposing the fire fighters to the fumes generated by the burning nitrate.

Flooding with water is the only effective way to fight a nitrate fire as the nitrate has its own oxygen supply built in.

This material is not hazardous if you will keep three things in mind.

1-No contamination unless specifically recommended.

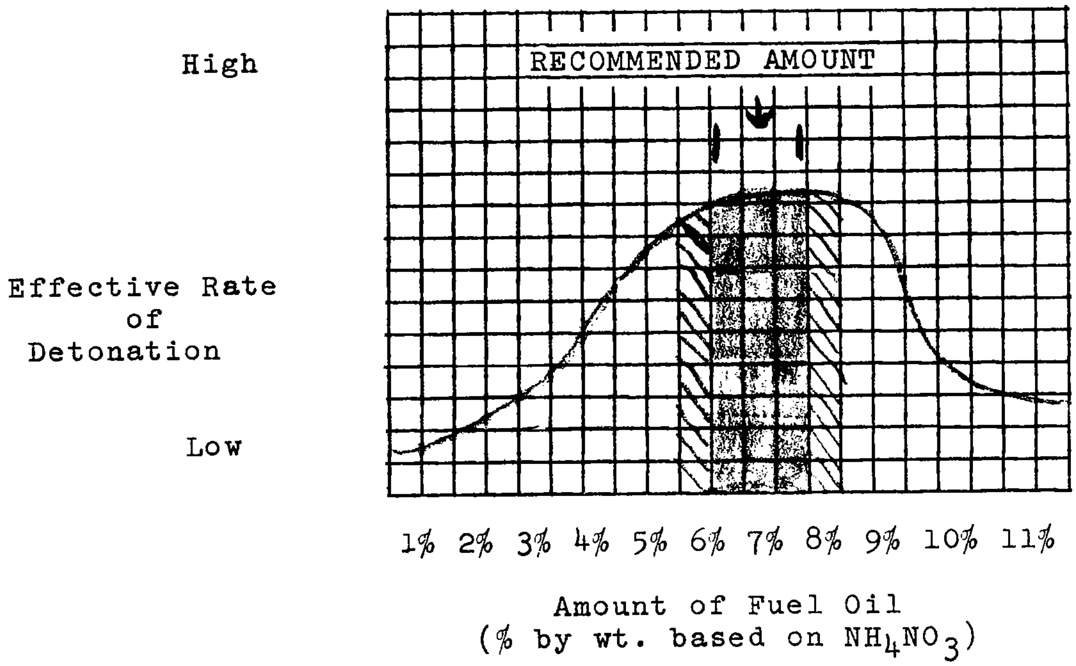
2-No confinement or excessive heat (flame).

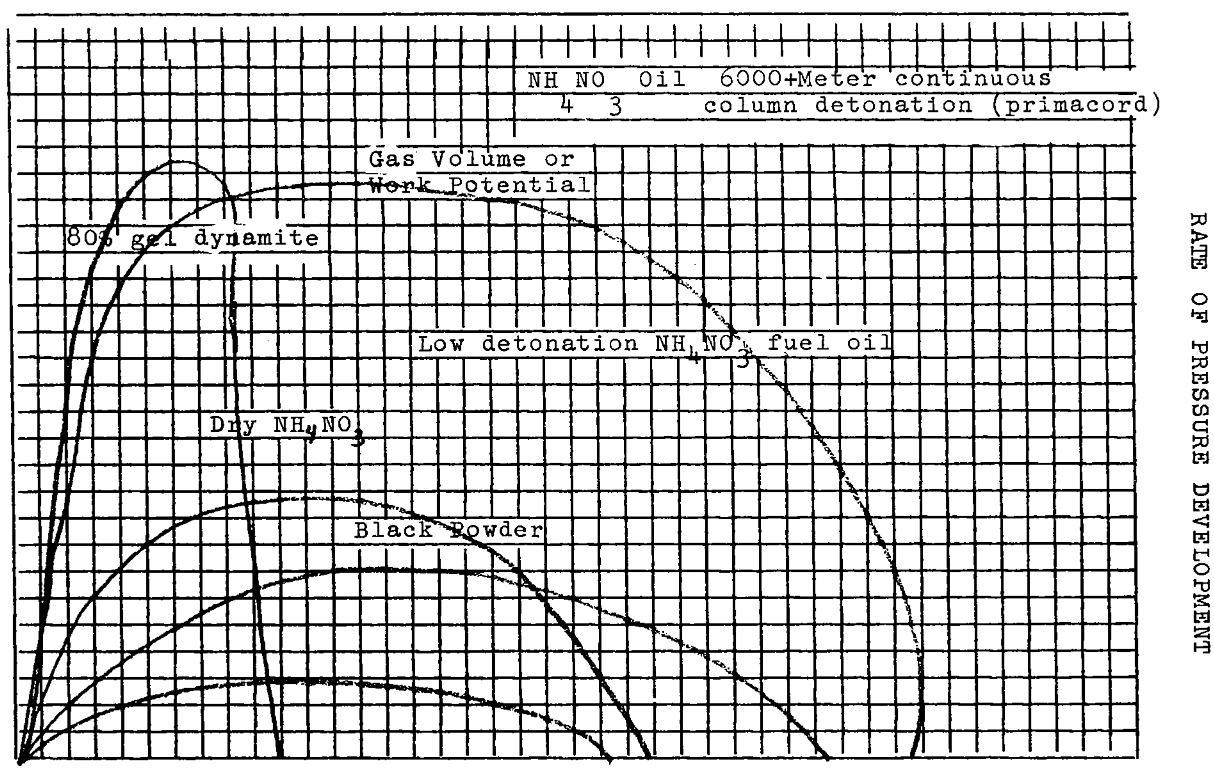
3-No storage near the critical mass (123 tons).

Always keep loose nitrate from broken bags carefully swept up so Ammonium Nitrate dust is not present in the storage area. The dust, of course, is flammable.

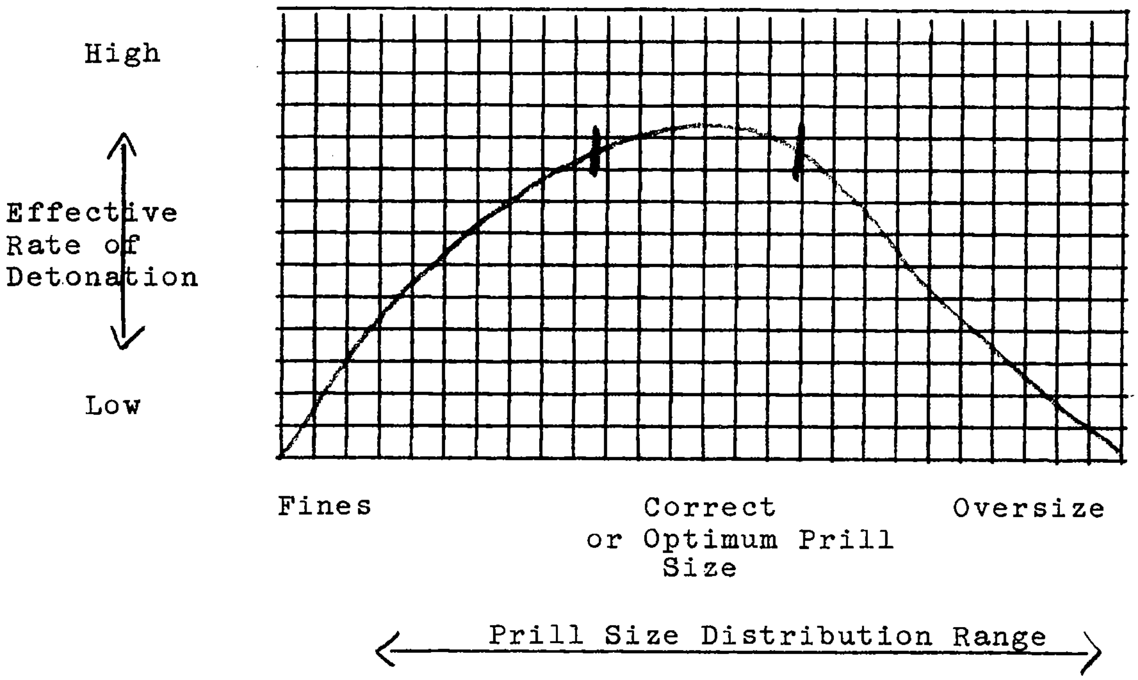
We recommend the use of the bulletin on Ammonium Nitrate published by the Manufacturing Chemists' Association (1625 Eye St. N. W. Washington 6, D. C.) for further information on handling and storage.

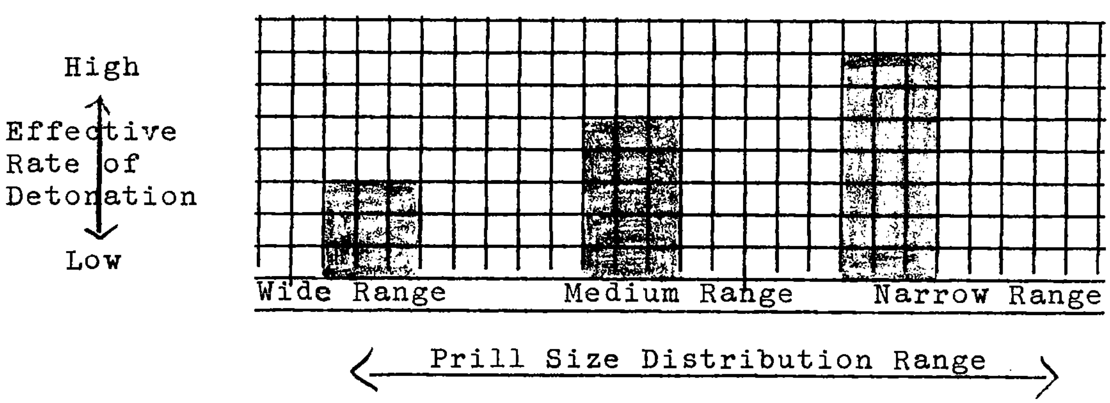
Fuel Oil Content and Detonation Rate





PRILL SIZE DISTRIBUTION RANGE
and Effect on Explosive
Efficiency

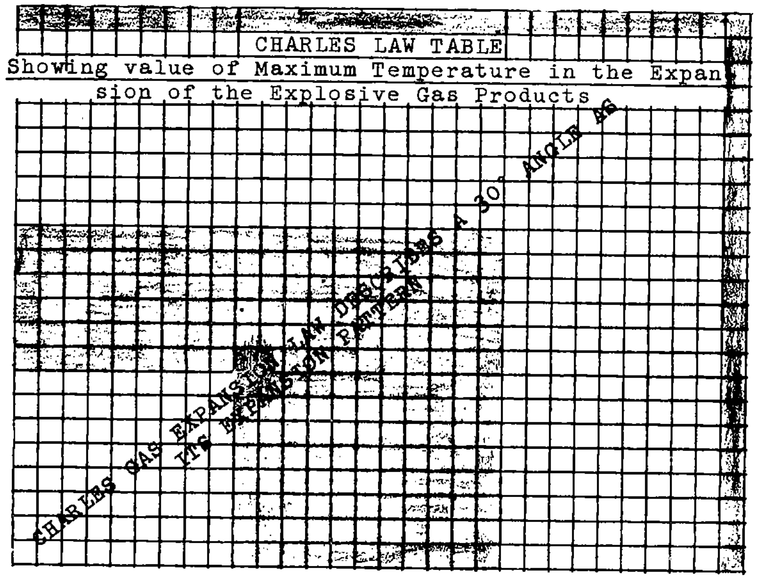




Temperature

Gas Volume after
Addition of 273°
K of Temperature

Original gas
Volume



For every increase of 273°K, the gas volume is doubled.

Volume

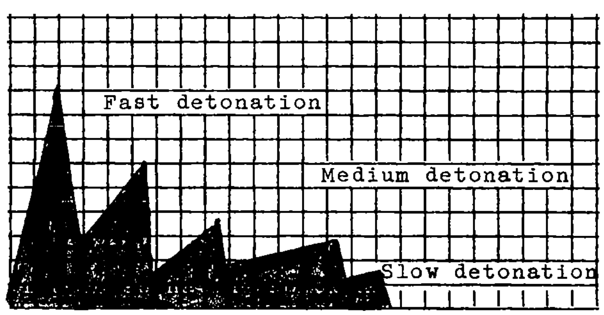
This entire area represents the gas volume after addition of only 546°K of temperature. This is four times the original volume.

DEMONSTRATION OF TEMPERATURE-TIME RELATIONSHIP OF HIGH SPEED DETONATION

This performs maximum work through production of highest possible temperature and resultant maximum gas volume.

T
Temperature
5,000°C.

2,000°C.

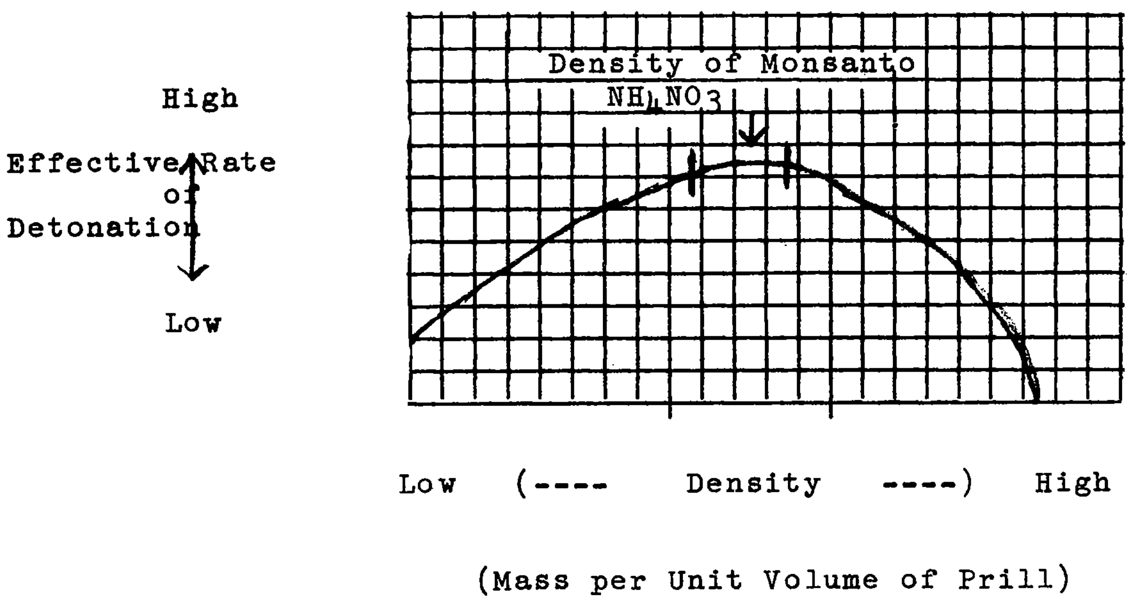


T'
Time in milli seconds

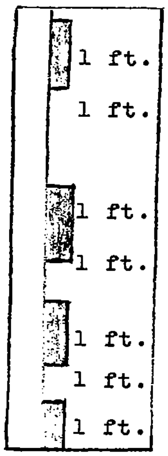
Relative Rates of Detonation Showing Time Versus Temperature Relationship Which Affects Gas Volume as Shown Above.

Fast Detonation can add up to 2,000°C. temperature to a given amount of explosive products.

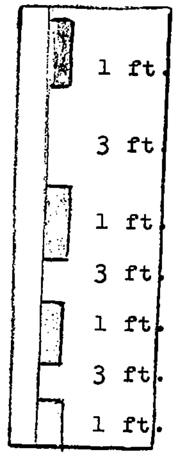
Physical Properties of NH_4NO_3 Prill and Their Relationship with Explosives Effectiveness, Demonstrating Optimized Characteristics of Monsanto's Prill



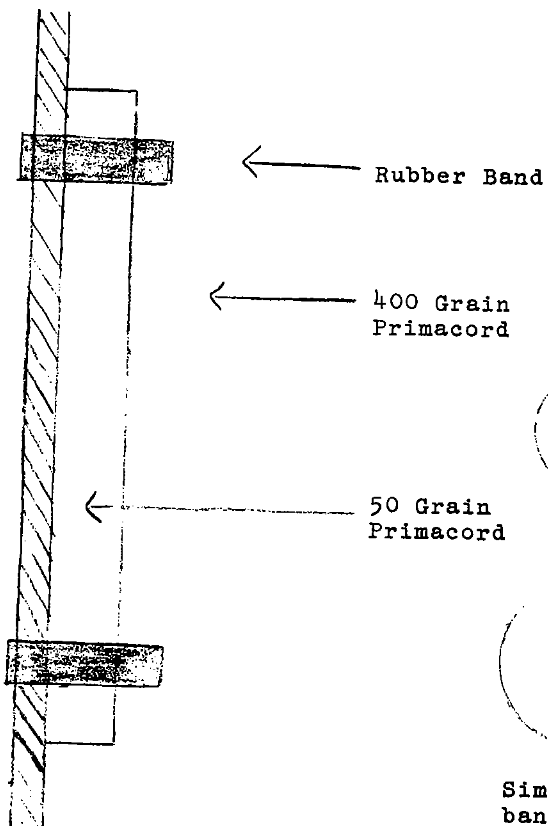
METHOD OF USING PRIMACORD LENGTHS AS DETONATOR



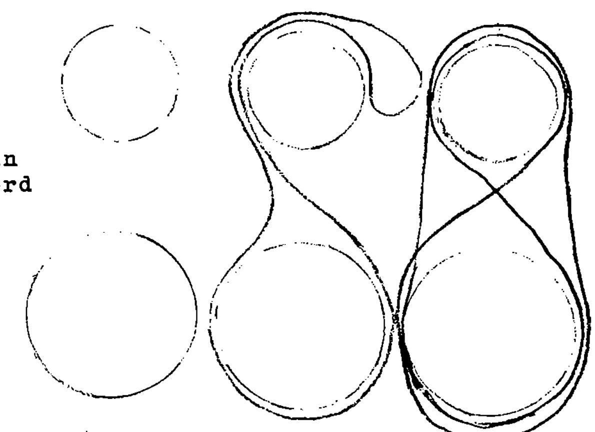
Damp hole and
2-3in. hole



Normal dry shooting
and holes larger than
3 in. in diameter

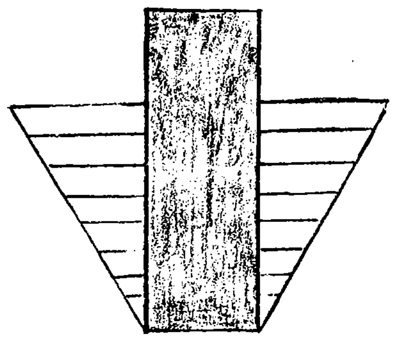


Position of 400 grain stick of Primacord attached to standard 500 grain Primacord by looping a rubber band about the both of them

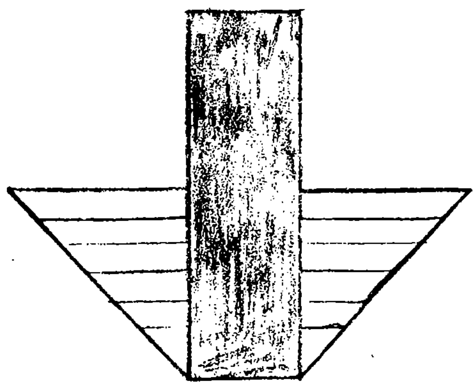


Simply loop one end of the rubber band around the 400 grain, pass the band around the 50 grain, then loop the remaining end of the band around the 400 grain again.

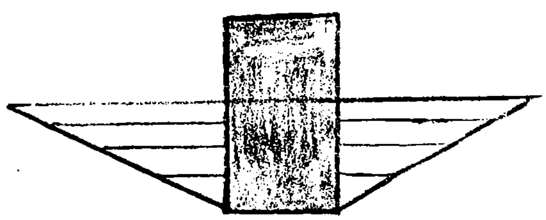
The higher the rate of detonation, the flatter the angle of effect at the bottom of the hole, all other factors being constant.



3-5,000M/sec.



5,6000M/sec.



6,700M/sec.
Recommended

One of the advantages of the Primacord detonation method is, therefore, a more even quarry floor and less toe which would detract from good shovel operation.

(The above angles are approximate)

Application of Seismology to Highway Engineering Problems

by Curtis R. Tuttle
Engineering Geologist
United States Geological Survey
Boston, Massachusetts

Introduction - Patrons of geophysical methods frequently exhibit one of two opposite attitudes, (1) either they believe all sub-surface design problems can be solved with mysterious instruments, or (2) they relegate such methods to the status of witchcraft and ouija boards. Geophysical methods are not design panaceas and seismology is no exception. However, seismology is being used by 5 state highway organizations in addition to other agencies. The degree of success increases as the method is employed by technicians using the group-study method. Group-study insures proper consideration of the closely related fields of physics, engineering, soil mechanics and geology. The degree of success increases as the seismic method is employed in areas of simple geologic environment. However, even at such sites identification borings are often necessary to permit quantitative interpretation of seismic data in terms of depth to concealed bedrock or of physical soil and rock properties.

This paper is based on 4 years experience in applying seismic methods to highway design problems in Massachusetts. The studies were made as part of a continuing Co-operative Geologic Project between the US Geological Survey and the Massachusetts Department of Public Works. My purpose is to present an unbiased appraisal of the seismic method for highway design purposes, therefore, geophysical problems and velocity effects are mentioned without explanation. Those interested in details are referred to the references cites (Reiland 1946, Leet 1950.) Four points will be discussed in the following order:

1. The function of the seismograph as an engineering tool

2. Highway design problems suitable for seismic interpretation

3. Design details derived empirically from group-study data

4. Comparison of seismic versus boring methods.

Function of the Seismograph as an Engineering Tool

The principal function of the seismograph is to detect essentially horizontal

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elastic discontinuities in the earth's crust. In practice an elastic wave field is established in the crust by detonating a small charge of dynamite confined in a shallow hole. The reaction of sub-surface soil and bedrock units to this field is observed by measuring the time of first arrival of the refracted wave for various distances of explosion source (shot point) and detecting instrument (seismometer) along a tangent on the surface of the ground. From time and distance, speed is calculated. Since the speed has the direction of the tangent it becomes a velocity. Velocity discontinuities can usually be identified and correlated with soil and bedrock units whose presence and physical characters have been established by geologic observation, borings, or other independent means. In addition, the thickness of each soil unit and the total depth to bedrock are calculated directly from observed times, distances and velocities. Since velocity is a function of the elastic moduli, the stress-deformation character of earth materials can also be measured and/or inferred at sites having nearly ideal, theoretical sub-surface conditions.

Derived Functions of velocity data are the distribution, discussions, shapes and attitudes of soil units; the occurrence of lensing and other horizontal discontinuities; the topography of the concealed bedrock surface; and less frequently the massive versus stratified character of bedrock. All derived functions should be used with caution until the soundness of interpretation has been established independently. Seismic data are interpreted mostly by empirical, highly subjective methods; usually the accuracy of derived observations increases with experience, and particularly if the experience is limited to an area consisting of only two or three basic geologic environments. In Massachusetts these environments, with respect to highway relocations, are:

a. one soil unit above massive, crystalline or metamorphic bedrock

b. one soil unit above fractured, "blocky" crystalline or metamorphic bedrock.

c. one soil unit above metamorphic rock containing schistocity or other planar structure, normal to which an increase of volume and separation of discrete platy unit has occurred.

d. one soil unit above sedimentary bedrock.

Additional layers of soil above bedrock and shallow (4 to 10 feet) versus deep (greater than 10 feet) depths to bedrock modify this basic classification and increase the difficulty of interpretation.

Usually granular and plastic materials such as sand, gravel, clay and glacial till are characterized by velocities of 600 to 6,000 feet per second. Rigid materials such as shale, sandstone, and the crystalline and metamorphic rocks are characterized by velocities of 7,000 to 20,000 feet per second. Velocity is also influenced by degree of cementation and compaction, increasing geologic age, degree of weathering, attitude of bedding and schistosity, and the frequency and distribution of fractures and joints.

Although there is marked success in correlating velocity with material this derived function must also be used with caution. A water saturated outwash (sand and gravel with some coarse silt and virtually no clay) will exhibit the same velocity as that of water alone: about 5,000 feet per second. Further, glacial till and glacial clays frequently exhibit the same velocity. Geologic study may admit of any or all of these physically dissimiliar soils possible occurring at a single site. The same outwash, when tested at normal water content usually has a velocity of only 3,500 feet per second. A trial of electrical resistivity using the Lee partitioning configuration, shows that this method sometimes offers important aid for interpreting velocity-identification, and depth effects, at sites where seismic data alone do not afford a unique solution.

<u>Highway Design Problems Suitable for Seismic Interpretation</u> - a report by the Missouri State Highway Commission of geophysical methods used by state highway departments, shows that in 1957 only California, Massachusetts, New York, New Jersey, and Ohio were using the seismic method. Use by these organizations as shown in Figure 1, was primarily for classification of excavation. Suitable design problems are those which can be related to functions derived from velocities, and for which correlation of velocity and material is established. These problems are stated, the site conditions affecting interpretation and reliability are listed, and a specific appraisal of reliability in Massachusetts is made. The following problems have been reliably solved in Massachusetts and are suitable for application in other areas of similiar geologic environment. They are listed in order of most frequent use, they are of similiar reliability, and they represent the limits of applicability for normal site investigation:

a. depth to bedrock, when this depth is at least 10 feet.

b. the number, approximate thicknesses and identification of major soil units overlying bedrock.

c. identification of bedrock and detection of major planar surfaces which are distinctly related (1) to the direction of maximum relief due to blasting, and (2) to the azimuth and slope ratio which results; a factor of rock slope design.

d. preparation of subsurface bedrock contour map.

e. determination of the altitude of the water table in coarse-grained, unconsolidated soils.

Because Massachusetts' soils of glacial origin are physically dissimilar from those of the Middle Atlantic States, and since my experience with geophysics excludes the latter States, any extrapolation of interpretation details would be presumptuous. However, the following conditions are common in a wide variety of geologic environment:

Reliability of seismic methods increases as the actual subsurface geological conditions approach the ideal, theoretically desirable conditions. The theoretical conditions are a prerequisite for the absolute analytical solution of depth to bedrock, and thickness of soil layer problems. Site geology in Massachusetts flagrantly violates the theoretical conditions in about 50% of the area which has been studied. In a general sense, reliability is represented by design, practice of the Massachusetts Department of Public Works: bedrock profiles and material identi-

Common Site Conditions Affecting Interpretation and Reliability

A. Quantitative data obtained from site inspection or velocity measurement:

Aspect	Optimum Condition	Limiting Condition
Velocity ratio	1/2.4 (or greater)	1:1.35
Slope of original ground	6.6:1 (or less)	3.5:1
Depth to bedrock	18 feet (or more)	10 feet

B. Qualitative data, sometimes suspected, and difficult to evaluate:

1. Three-dimensional effects	
a. side refraction - effect:	calculated depth to bedrock less than true depth
b. angle of emergence-effects:	calculated depth and material identification
c. true rock velocity -	erroneous measurement affects depth calculations
2. Velocity inversions - effect:	creates "hidden layer" which remains undetected; depth error is equal to thickness of the hidden layer
3. Geophysical beterogeneity of soil units-	example: boulders of 10 cu. yd. volume sparsely distributed, or boulders of 2-5 cu. yd. volume compactly 'nested'
4. Geophysical beterogeneity of bedrock units -	example: weathered bedrock layer with irregular horizontal and vertical distribution; irregular areal distribution of 'blocky' or 'platy' versus massive bedrock

fication derived from seismic data are preferred to the former practice of "guessing" excavation quantities.

Specific reliability has been related only to those sites at which a direct point-by-point comparison of depth to bedrock is available from top of bedrock surveys taken for excavation quantities. As of February 1957 our engineering geology unit has made seismic studies at 216 sites. Only 38, or 17.6% of these sites have provided comparison data of the required quality. At the remaining sites highway alignment was radically changed, unclassified excavation was specified, or comparison data was never obtained. The following types of comparisons are recognized from data obtained at the 38 sites:

a. seismic interpretation 100% correct: altitude of subgrade much higher than underlying deep bedrock

b. seismic interpretation 85% correct: for example; a deviation of 0 to 3 feet above or below the actual depth of 20 feet

c. total failures: due to 1. bedrock less than 10 feet deep, range of observed error is 88% to 300% above or below the actual surface 2. unique site conditions, such as sub-surface elastic dikes, and false velocity effects
3. temporary seismologist, unfamiliar with local geology.

The percentage of each type of comparison is shown in Figure 2. Failures could be reduced to 16 percent by including a factor to show that the over-all estimate of excavation quantities at these sites is usually within the 20 percent range acceptable to the Department of Public Works. At non-comparison sites acceptable results known qualitatively could be used to increase the over-all accuracy. However, Figure 2 shows the current correct order of magnitude based entirely on a small quantity of precise survey data, and represents a high degree of success.

DESIGN DETAILS DERIVED EMPIRICALLY FROM GROUP-STUDY DATA - restating the idea of group-study (Beers, 1950) total sub-surface data are fully known at a site by considering integrated geophysical, soil mechanics and geologic data. This requires cooperation among specialists in the several fields. In Massachusetts detailed group-study material consists of seismic, soil mechanics and geologic investigations at some 8 sites. Data from electrical resistivity (LEE partitioning configuration) and identification borings are available for 14 additional sites. These studies show that there are empirical relationships between seismic velocities and some of the physical properties of soil. Two examples will be given for which these relationships are tentatively established and the inferred soil properties have been reported for design use. Secondly, earth slope design details are discussed.

1. Till with a velocity range of 5,000 to 6,500 feet per second has the following properties which are based on closely grouped data from three sites:

a. density ..130 lb/ft^3

b. passing U. S. Std. No. 200 Sieve....40%

c. liquid limit...10%

d. bearing capacity .....10 $tons/ft^2$ minutes.

e. excavation treatment .....requires scarification for efficient removal with self-powered excavation equipment (pans)

f. material use.....unsuitable for compacted fill in wet areas, unsuitable for fill directly below subgrade.

2. A till of restricted geographic distribution (Worcester plateau) has a velocity range of 3,000 to 4,000 feet per second, contains objectionable quantities of soil particles passing the No. 200 sieve and exhibits "bedding". On a 3:1 slope active solifluction scarps occur within a few months of final grading and mulching. Thick mulching should be avoided since this aids development of high pore water pressures at the slope face. Bituminous ditches normal to the toe, and collecting run-off from a citch at the heel, are rapidly undermined and destroyed. These scarps are caused by ground water seepage through sand and coarse silt layers included in the till. Such slopes can be cut to a 2.5:1 ratio, and provided with boulder or crushed rock paving from the toe to the lower third point of the slope.

These correlations of velocity and group-study data are still incompletely known: further study might show them to be simply coincidence. The examples are offered as observations, not as claims, for the seismic method.

Earth slope design details
(1) granular soils: sand and gravel identified from velocities usually has sufficient moisture to exhibit some apparent cohesion. The angle of repose for this material is less than the angle of shearing resistance. For conservative design the angle of repose is suggested as the slope ratio for a proposed cut. (2) cohesive soils: particle bond in till of 5,000 to 6,500 feet per second velocity range is primarily related to changes in water content, therefore the material has cohesion and shearing resistance, (Terzaghi, 1943). Shearing resistance is stated by Coulomb's equation which contains the term: tangent of shearing resistance. Both the triaxial compression test for soil, which also measures the tangent of shearing resistance, and laboratory determination of Young's modulus, measure axial strain or functions having the same dimensions. Seismic velocities are related to the elastic moduli. So there appears to be a relationship between velocity and tangent of shearing resistance. This approach might lead to the estimation of shearing resistance directly from geophysical observations. In Massachusetts this

would be a distinct advantage. The problem is a current field of investigation. Finally, using the methods of Evison 1956, Leet 1950, and Outenberg 1949, Young's modulus of a till on Cape Cod was determined to be 526,000 pounds per square inch. Density was assumed as 125 pounds per cubic foot and Poisson's ratio was calculated as 0.28. This assumption of density, which is a linear function of Young's modulus, introduces only a small error therefore the value of Young's modulus is of the correct order of magnitude. This method has distinct advantage for the design of footings for grade separation structures and bridges.

COMPARISON OF SEISMIC VERSUS BORING METHODS - Reliability and limits of applicability have been discussed for the seismic method. Familiarity with these aspects of boring methods is assumed, therefore only cost and speed need consideration. Daily operating cost for the standard crew employed in Massachusetts is $170. This includes wages, per diem and transportation for 2 engineers and 1 geologist-seismologist; wages and transportation for 4 maintenance laborers obtained from the local highway district; and expendable supplies including dynamite. Assuming a site is 1,300 feet from the point of closest vehicle access and that clearing of minor amounts of brush is necessary, an average day's work consists of 5 reverse-profile traverses. Preliminary velocity data are plotted in the field to determine data quality and to insure adequate areal coverage of the site. The length of each traverse is a function of the depth to bedrock and ranges from 110 to 550 feet. The amount of areal coverage depends on the number of roadways proposed and/or the presence of a center mall. Standard coverage consists of three parallel series of traverses: one on the centerline of the mall and one half way between the toe and heel of each proposed side slope. For comparison with boring costs the following data are instructive; 5 traverses produce depths at 6 shot points or "holes":

Further economy results from rapid mobilization demobilization and "move" time, and the fact that water is not necessary as for core drilling.

Speed of the field operation is influenced by some of the factors above. Access to sites without necessity for building roads is easily made by the

average depth to concealed bedrock	depth to bedrock x6 determinations	daily cost, $170 divided by total "footage"	cost per linear foot
20 ft.	120 lin. ft.	$170/120	$1.42
100 ft.	600 lin. ft.	$170/600	0.28

Aspect	Relative Ratio Seismic: Borings	Remarks
Original equipment cost	3:1	-
Yearly maintenance cost	1:1	-
Daily operating cost	0.7:1	-
Ease of access to site	10:1	-
Moving time hole-to-hole	1:5	minimum
Mobilization-demobil time	1:10	-
Safety hazards	8:1	dynamite
Highly trained personnel	10:1	
OVERALL SPEED	8:1	minimum
ACCURACY OF DEPTH TO BEDROCK	2:1	related to Mass.* boring practices
ACCURACY OF MATERIAL IDENTIFICATION	0.7:1	related to Mass.* boring practices

*Note: 1-inch pipe samples acquired with a 140 pound hammer and driven to "refusal" are provided by contract. Since highways are very narrow and exceedingly long only this size and type of boring is economical. However, the author retains his aversion to samples except those acquired with 3-inch diameter samplers and a 340 pound hammer, with "refusal" proved by at least 20 feet of NX core from series X core barrels.

7-man crew; it is not uncommon, though wearisome, to carry equipment 3,500 feet from the vehicles. All equipment is carried in one station wagon including room for one extra passenger and the dynamite. Because seismology is a line technique as opposed to boring methods, a point technique, this interpretational advantage permits rapid accumulation of data for a sub-surface bedrock contour map or a continuous bedrock profile. That is, the direction and qualitative slope of local bedrock irregularities can frequently be obtained with moderate accuracy, by interpreting relative velocities along the rock line segment of the time-distance graph from reversed profiles. Concealed bedrock topography in Massachusetts is highly irregular so this technique is commonly used.

A comparison of seismic and boring methods can be shown in tabular form:

References

Beers, R. F. (1950) Geophysical problems in Applied Sedimentation: Applied Sedimentation, P. D. Trask, ed. John Wiley & Sons.

Evison,F.F. (1956), The Seismic Determination of Young's Modulus and Poisson's Ratio for Rocks In Situ: Geotechnique V. VI,n,3,Sept. 1956, pp. 118-123.

Gutenberg, B. (1949), Internal Constitution of the Earth: Dover Publications, Inc.

Heiland, C. A. (1946) Geophysical Exploration: Prentice Hall, Inc.

Leet, L. D. (1950) Earth Waves: John Wiley & Sons, Inc.

Terzaghi, K. (1943) Theoretical Soil Mechanics: John Wiley & Sons, Inc.

Proceedings of the
Tenth Annual Symposium on

"GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING"

February 20, 1959

GEORGIA INSTITUTE OF TECHNOLOGY
ATLANTA, GEORGIA

Symposium on Geology

as Applied to Highway Engineering

at

Georgia Institute of Technology

Atlanta, Georgia

on

February 20, 1959

This meeting was the tenth assembly of these Symposia which had its initial start in Richmond, Virginia, in 1950. This Symposium was under the joint sponsorship of the Georgia State Highway Department and the Georgia Institute of Technology. The Committee trusts that the publication of these papers will be of benefit to the many interested in the related problems of Geology and Highway Engineering.

On the evening previous to the Symposium, a social gathering was held in the Lounge of the Architects and Engineers Institute Building in Atlanta. This gathering was made possible through the interest of Mr. Nelson Severinghaus of the Consolidated Quarries, Inc., of Decatur, Georgia.

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GEOLOGY AS AN AID TO RIGHT OF WAY

By

George D. Felix, State Right of Way Engineer,

Virginia Department of Highways.

You are no doubt wondering what in the world Geology has to do with the purchase of Right of Way for highways. Or, if you are in the highway field, you may say: "These Right of Way guys are always getting in on things, and this is another way of doing it." Being one of the "Right of Way guys," I can say that we do seem to get drawn into a lot of things that make me wonder - "How did we get into that?"

To establish a connection between Geology and Right of Way, let's see what Mr. Webster might say for each of these in layman's language. For Geology, he says it is "The science dealing with the structure of the earth's crust and the formation----of its various layers; it includes the study of individual rock types." As to Right of Way - "A strip of land acquired for a public road; right of passage over another's property." So we find that in getting Right of Way, we are securing "right of passage," or buying land, and the Geologist is studying how the land came to be, including any rocks that may be on or under it. Now, I sneaked up on you with that one!

In acquiring Right of Way we deal with all kinds of people in all walks of life. Most people only want what is fair for their property, or right; that is needed. Occasionally, you encounter one - - - but, that is another story that we won't go into here. There are times when an owner believes, or so he tells us, that he has a veritable gold mine in gravel, sand, marble, or just a special kind of dirt! And, of course, there are springs destroyed, houses shaken apart, and slides - numerous slides - all the fault of the Highway Department.

These claims are hard to combat. The local people who might know the real answers are neighbors, possibly friends, of the property owner and loath to testify against him. Many of these claims require scientific investigation and real proof to upset them. I mean real proof; for again the Commissioners, or Jury, hearing a case are local people and their sympathies are apt to be with the property owner. It is in combating these claims, either before they go to Court or in Court, that the Geological Section of the Highway Department has done some really marvelous work.

One of the most unusual cases we have encountered was just North of Winchester where we were preparing to made a road four-lane, divided, by the addition of another lane parallel to the existing roadway. This threw the new lane close to an apple candy factory. Part of the process of making the candy was the coating of it with a supersaturated sugar solution, and jarring would cause this solution to crystalize, rendering it worthless. The claim was, when we went to deal for the right of way, that by moving the road closer to the factory, the vibration from the many trucks using the road would put the candy company out of business.

You may, or may not, know that Winchester is in the Valley of Virginia, and a great part of this area is underlaid with limestone often at, or close to, the surface. On the face of it, the claim appeared to have merit. To determine just how much merit, we appealed to Mr. W.T. Parrott, Highway Geologist. He knew of a company specializing in vibration studies and solicited their help. Their equipment proved that there was more vibration, and of greater intensity, from the normal operations within the building than from the traffic on the highway. Thus, this claim went out the window, and we purchased the right of way at a reasonable figure. Incidentally, after the road was completed the vibration tests were repeated, and the original findings confirmed.

Virginia is proud of her historic past, and cherishes buildings and other items connected with it. That these have often been bought and restored by persons of wealth from other areas doesn't lessen their value. About two miles from an historic old home, restored, the Highway Department opened up and operated a stone quarry. The owner of the house claimed extensive damage to plaster and cracking of the walls. The vibration unit registered no vibrations when test shots were put off in the quarry, and the case collapsed.

As stated earlier, owners have convinced themselves that they have valuable deposits or formations and don't hesitate to assert all kinds of claims when some part of these are included in a proposed right of way. Such a situation arose on a secondary road we were planning on widening, and at the same time we were going to ease some of the curves. What looked to us as just so much more rock was asserted by the owners to be marble, and a claim for a $70,000.00 settlement was asked. Careful geologic investigation and mapping showed that the material was a limestone that could be polished and would sell as a marble. Only a relatively small amount was actually taken in the right of way, and the access was better, if affected at all. A settlement in the neighborhood of $2,500.00 left everybody happy.

Cases of damage by reason of slides or damage to wells and springs are too numerous to enumerate. Where claims of this nature come in, we are anxious to know the true answers. If the owner has suffered damage by reason of the highway location or construction, we want to compensate him fairly for it. On the other hand, if there has been no damage by reason of the Department's action, we don't want to make any "gifts." Faced with these claims, we ask the

Geological Section to supply us the answers. In order to be in a better position to investigate these matters, the Geological Section has been instrumental in getting records of springs and wells near a project, kept by the project inspector. With this, they don't have to be in the position of "trying to lock the stable door after the horse is stolen."

I don't know how other States handle it, but in Virginia the securing of land for offices and shops, and the planning and erection of these buildings, is a part of the work of the Right of Way Division. With the larger, multi-storied buildings, such as District Offices or the proposed new Central Highway Office in Richmond, it is essential that we have accurate information on foundation conditions. Again, we call on the Geological Section and by borings, test drillings, soil evaluation, they supply our architects with what they need.

So, gentlemen, I hope these necessarily rather brief summaries of some of the ways Geology has helped Right of Way have proved the point. We are still finding new ways that the two Divisions can be of help to each other. I feel that I would be remiss if I did not mention that I believe a large measure of our successful relations is due to the man at the head of the Geological Section. He is ever alert to find ways to help us, and when we approach him with a problem, he is cheerful to undertake to find a solution. With a co-worker of this type, I believe any Right of Way Division will find Geology a very valuable aid in their work.

THE DISTRIBUTION AND CHARACTER OF STONE FOR AGGREGATE IN GEORGIA

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INTRODUCTION

This paper is planned to give a comprehensive, but very brief, review of the distribution of rocks suitable for aggregate, gravel, or crushed stone. The stone industry is next to the top mining industry in Georgia, and it has a promising future. Discussion is keyed to the State Geologic map of 1939. A map which accompanies this paper has been prepared to indicate the major regions of the state from the point of view of the distribution and character of aggregates. In order to prepare this later map the 107 formations and rock types of the State Geological map are here reduced to 11.

Some parts of the state have no quarryable stone, but Georgia compares favorably with other states in varieties and distribution of stone, and useful stone for aggregate can be obtained in at least eight of the eleven districts mentioned above. There will be no attempt here to go into detailed descriptions of the rock types which are discussed, but reference lists will be given which supply available information regarding the location, distribution, petrographic description, and chemical composition of the rocks. All of these references, with two exceptions, are either publications of the Georgia Geological Survey or have been actively sponsored by this Survey. The Georgia Geological Survey which is the Department of Mines, Mining and Geology, has pioneered the study of stone at no expense to the producers of the state. Representatives of the stone industry are encouraged to consult with our geologists who welcome the opportunity of assisting them with their problems in the field and in the laboratory.

References

Georgia Geological Survey, Circular 2, 10th Edition, Directory of Georgia Mineral Producers, 1959.

Vallely, James L., and Peyton, Garland, The Mineral Industry of Georgia, U. S. Bureau of Mines Minerals Yearbook, 1958.

PRE-CAMBRIAN AND PALEOZOIC

(Crystalline Rocks)

Granites, Granite Gneisses and Migmatites

These rocks in central Georgia and southeast central Alabama extend further south than anywhere else in the country except in the central granite area of Texas. For this reason they are well located for quarrying, and for distribution, not only in Georgia and more northern states, but in south Georgia and Florida; and with the development of water transportation on the Chattahoochee and Savannah rivers, the stone industry may be expanded through foreign trade with the West Indies and South America. As you will see from an inspection of the state geological map, these rocks have wide distribution over the crystalline area of Georgia and geologists realize that we have an almost unlimited supply. Large tonnages of visibly undeveloped stone are now known to exist.

The commercial operations at present are generally in the granites, foliated granites and migmatites, and the large crushed stone operations are near the principal centers of population. For example, Columbus, Macon, and Augusta, which are on the Fall Line, all have quarries not far away in the crystalline area east, west, and north. Atlanta is practically ringed with large crushed stone industries found in Cobb, Douglas, Henry, and DeKalb Counties. No attempt will be made here to list the names and locations of these industries in the state. They may be obtained from any current edition of Georgia Geological Survey Circular 2, Directory of Georgia Mineral Producers. The large monumental stone industry of Elbert County serves distant markets; thus, this industry does not depend upon a local market. The porphyritic granites are foliated and contain large crystals or porphyroblasts of potash feldspar which are usually pink in color. They are quarried around Augusta, in Rabun County, near Palmetto in Fulton County, and in Coweta County. The pyroxene granites, better known as charnockites of Upson County have not yet been used much.

Two large belts of granites and migmatites occur in the crystalline area but large and small bodies of such rocks are scattered about in many other localities. A western belt, west of the Brevard formation (see State Geologic Map) has important outcrops in Carroll, Douglas and other counties in this belt, extending in a northeast direction through Habersham and Rabun Counties. A very prominent belt of granites and migmatites enters Georgia in Heard County and follows the eastern side of the Brevard belt, practically through the state and into South Carolina.

Quarryable granite is more abundant than rapid observations would indicate because at the present time those areas which are covered with overburden, and where there are not very many outcrops, are not used. Two types of granite outcrops are common to the state. The boulder type, which is known from tropical and subtropical regions all over the world, is quite common but is not favored by quarriers because the granite, even though the boulders seem to be solid, is fractured and weathered for considerable and variable distances beneath the outcrops. The bare granite domes and pavements constitute the most favored localities for quarry sites.

The foliated granites and migmatites constitute the greatest single supply of stone in the state. In all cases where quarries are to be opened, the stone should receive standard tests such as are given to it by the Georgia State Highway Department. Locally there have been found some stone which for mineral or chemical reasons, contains defects which might not be detected by the usual physical tests. For example, the occurrence of titanite where hornblende rocks have been saturated and converted into migmatites is an undesirable element and leads to rock decay. Also, there are areas where the granites have been hydrothermally altered and where they contain sulphide mineralization and such stone is liable to rapid decay.

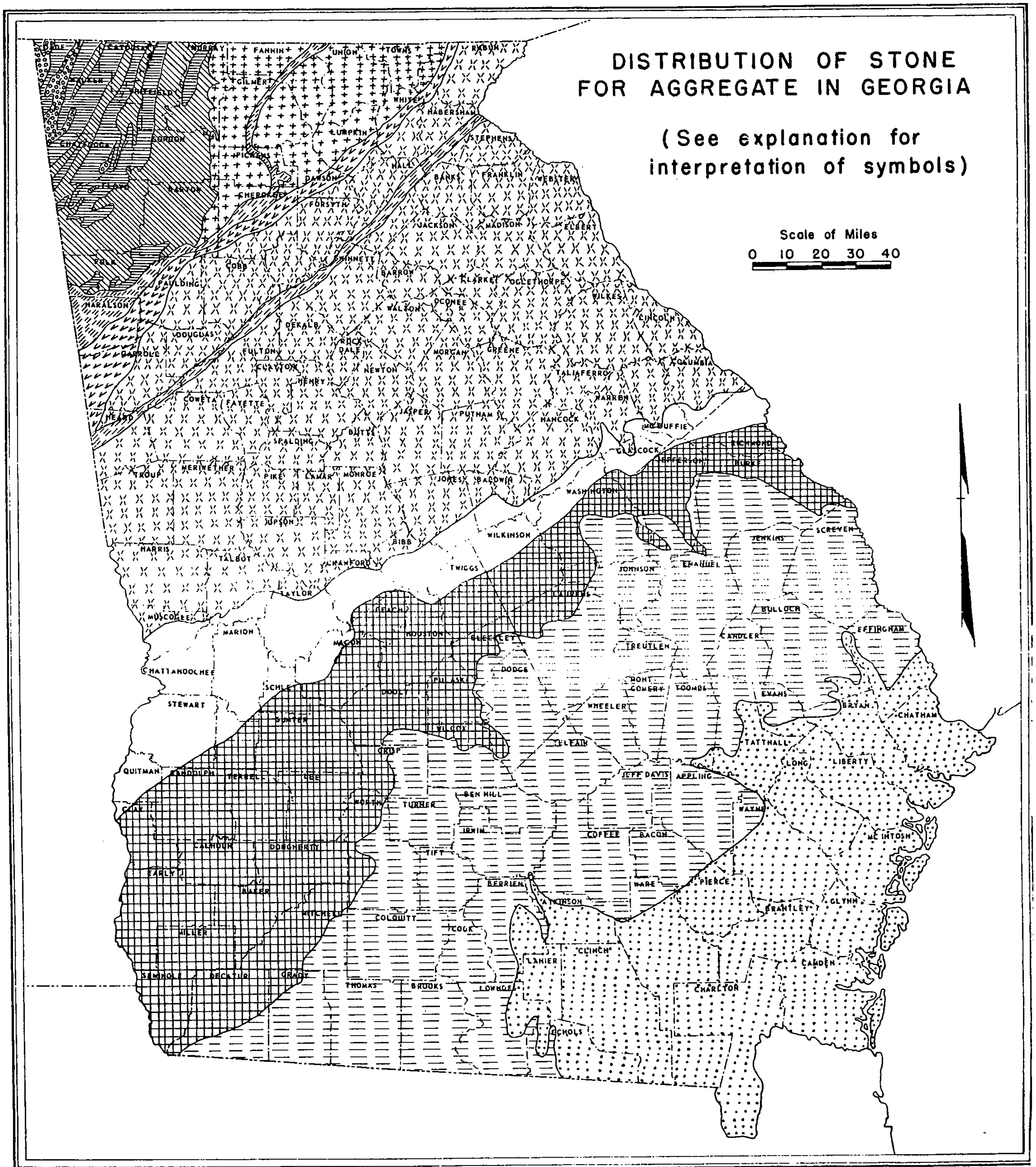
Hornblende Rocks

Metavolcanics which include hornblende gneisses are widespread over the crystalline area, but have not been much used. The most extensive district for such rocks is found in Carroll, Douglas, Paulding, Cobb, and Cherokee Counties where the volcanics are thick and massive and in some of these counties quarries have been opened in them especially where they have been converted into migmatites by granitization.

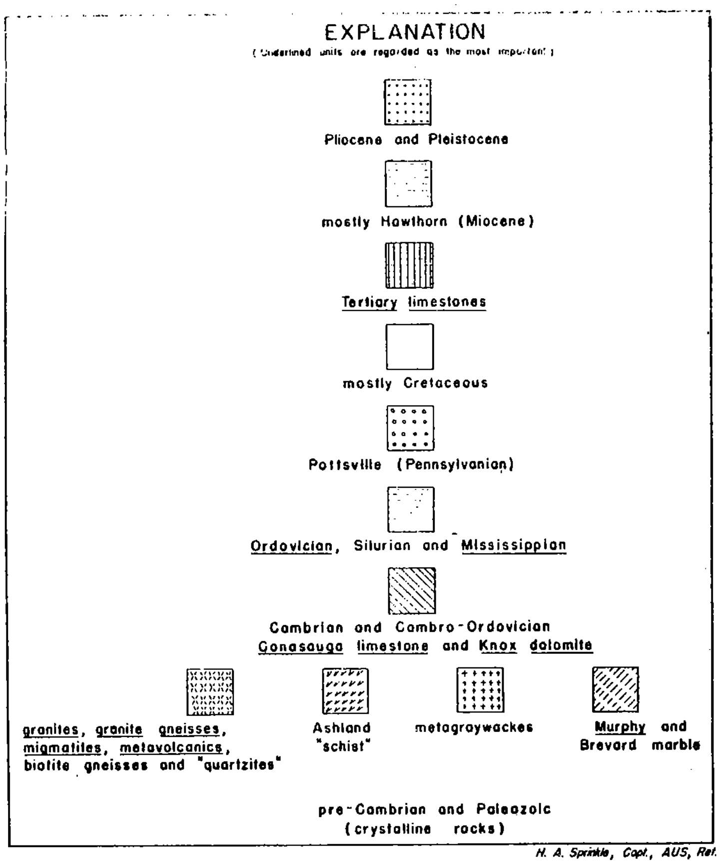
The Little River series (see State Geologic Map) although locally highly hornblende-bearing, contains many other types of volcanic rocks. This belt is undoubtedly much more extensive than is indicated upon the State Geologic Map, for volcanic rocks occur quite generally in the belt of crystallines adjacent to the Fall Line, all the way across the state. Several quarries have been opened in facies of these rocks and they promise to be locally important sources of stone.

Biotite Gneiss

Much of the crystalline area of the Georgia Piedmont is composed of



H. A. Sprinkle, Capt., AUS, Ret.



H. A. Sprinkle, Capt., AUS, Ret.

an old sedimentary biotite gneiss into which other rocks are intruded. Local quarries have been opened in scattered localities; as a rule the rock contains intrusions of granite.

Quartzite

Several types of quartzite occur over the crystalline area. In the extensive biotite gneiss and schist series, quartzites associated with muscovite schists are extensive over very much of the Piedmont area, south of the Ashland schist belt. They are particularly prominent in Douglas, Cobb, Fulton, Forsyth, Hall, Jackson, Barrow, Gwinnett, Rockdale, DeKalb, Troup, Meriwether, Harris, Talbot, Upson, Lamar, Baldwin Counties, and others. This type of quartzite has been used locally where it is dug out and thrown upon clay roads. It is quite friable and in many cases weathered to sand; thus not serviceable as aggregate. There is still a different type characterized by the Tallulah Falls and Toccoa quartzite beds. These are graywackes, very high in quartz and which contain also biotite and muscovite. These beds are massive; they have been quarried near Tallulah Falls and quite extensively in numerous places along the ridge just west of Toccoa, in Stephens County. In the latter case, the rock rather closely resembles a granite.

ASHLAND SCHIST

These rocks composing Ashland schist with inclusions of rocks mapped as Wedowee formation and Canton schist, occurs in a N.E.-S.W. belt in the northern Piedmont of Georgia. Later mapping of the Piedmont will undoubedly break this belt up into sediments and volcanics. The rocks are mostly altered sediments which are too micaceous to be sources of crushed stone. Graphite-garnet schists are quite common in the belt. In Carroll and Paulding counties, granite gneiss intrusions are mapped locally and some of these might supply quarry stone. Metavolcanics with their high hornblende content are abundant, and where massive, could supply local needs for aggregate. There is, for example, a quarry in this type of rock where road stone was removed in White County a short distance west of Cleveland.

METAGRAYWACKE AND GNEISS

This district (see accompanying map) is underlaid by great series of metasedimentary rocks—arkoses and shales, now metamorphosed to metagraywackes and slates. Metamorphism increases eastward and in the eastern part of the area, the rocks are definitely gneisses within the kyanite isograd. The region is practically devoid of granite intrusions, but there is some granite and coarse granite gneiss in Cohutta Mountains, east of Chatsworth (Furcron and Teague, 1947). In the southern part of the district, coarse granites occur which have been described as the Corbin granite (Watson, 1902). Thus far these granites have not been used for highway construction. The leuco-granites east of Chatsworth are quite local and good quarry sites are difficult to find. The Corbin granite further south, is injected into slates and graywackes, is inclined to be schistose and thus far has not been used.

Scattered over the district however, quarries have been opened in massive metagraywackes, particularly where the bedding or foliation is nearly horizontal and where the outcrops are in stream valleys. Metagraywacke is hard, massive, is a good stone for road work, but is abrasive and produces considerable wear upon equipment.

<u>References</u> (<u>Crystalline</u> <u>Rocks</u>)

Brent, W. B., Toccoa quartzite and adjacent rocks in Stephens County, Georgia, M. A. Thesis, Cornell University, 1952. Sponsorship and field direction by Georgia Geological Survey.

Clarke, J. W., Geology and mineral resources of the Thomaston quadrangle; Bull. 59, 1952, 99 pp., geologic map.

Crickmay, G. W., Geology of the crystalline rocks of Georgia; Bulletin 58, 1952, 56 pages.

Georgia Geological Survey, Geologic map of Georgia, 1939.

Grant, W. H., Geology of Hart County, Georgia; Bull. 67, 1958, 75 pp., geologic map.

Hermann, Leo A., Geology of the Stone Mountain-Lithonia district, Georgia; Bull. 61, 1954, 139 pp., geologic map and map of quarries and outcrops.

Hurst, V. J., Geologic map of Kennesaw Mountain-Sweat Mountain area, Cobb County, Georgia, 1956.

----------, Stratigraphy, structure, and mineral resources of the Mineral Bluff quadrangle, Georgia; Bull. 63, 1955, 137 pp., geologic map.

LeGrand, H. E., and Furcron, A. S., Geology and ground-water-resources of central-east Georgia; Bull. 64, 1956, 164 pp., geologic map.

McCallie, S. W., Roads and road-building materials of Georgia; Bull. 8, 1901, 264 pp., 32 pl., 4 maps.

Severinghaus, Nelson, Development of a crushed stone operation near Lithonia, DeKalb County, Georgia; Bull. 60, 1953, pp. 67-73.

Teague, K. H., Mineral resources of Union, Towns, Lumpkin and White Counties, Georgia (a map of mines, quarries and prospects) 1951.

Teague, K. H., and Furcron, A. S., Geology and mineral resources of Rabun and Habersham Counties, Georgia (a geologic and mineral resource map).

Watson, T. L., Granites and gneisses of Georgia; Bull. 9, 1902, 368 pp., 27 pl., 1 map.

MURPHY AND BREVARD MARBLE

Two belts of metasedimentary rocks strike generally northeast and southwest through the upper and middle Piedmont region of the state. The rocks of these belts differ in composition from the enclosing older paragneisses and migmatites and they contain the only known marbles from the large crystalline area of the state.

Murphy Marble Belt

A long overturned syncline of these sedimentary rocks extends from Hewitts, North Carolina, southwestward, entering Georgia near Culberson, N. C., in Fannin County, Georgia, through Mineral Bluff, Blue Ridge, Ellijay, Tate, Marble Hill, and Ball Ground. The rocks extend through the Allatoona district in southeast Bartow County, the belt widening to be correlated with the Talladega of Alabama. Marbles in this belt occur locally in numerous places, from North Carolina southward nearly to Canton in Cherokee County. Southwest of that district there is one occurrence of marble north of Buchanan in Harralson County. The marbles or crystalline limestones are extensively quarried in the vicinity of Tate and Marble Hill for dimension and monumental stone and also as crushed stone for various uses. Numerous deposits of these marbles throughout the belt ranging from white to blue and from high calcium marbles to dolomites have been recorded. The white dolomitic marbles in the vicinity of Whitestone have been extensively quarried. In Fannin County, there are two marble belts. These marbles are of excellent quality for highway construction. Details of the localities where outcrops may be observed or quarried can be obtained from the references cited below.

No fossils have been found in these rocks and their geologic age is uncertain. However, all geologists who have written upon this topic recently believe that they are of Paleozoic age.

Brevard Belt

A belt of metasedimentary rocks, very similar in composition and general appearance to the belt described above, enters the state from Oconee County, South Carolina, extending through northern Stephens, Habersham, Hall, Gwinnett, Fulton, southern Cobb, southern Douglas, Carroll, and Heard Counties into Alabama. The stratigraphy and age of these rocks is less understood than those of the previously described belt. Detailed work may definitely indicate the structure of the belt and may differentiate between the formations which compose it. On the state geologic map a long thrust fault is tentatively placed approximately along the southeastern side of this belt and present knowledge suggests that the rocks of the belt represent an overturned syncline thrust faulted along its southeastern side. Marble deposits have been recognized and described by the Georgia Geological Survey in this belt as far south as Flowery Branch in Hall County (Furcron, 1942). These marbles are of good quality for highway construction, but have been very little used. For specific location of deposits, see references given below.

References (Murphy and Brevard Marble)

Bayley, W. S., Geology of Tate quadrangle, Georgia: Ga. Geol. Survey, Bull. 43, 1928, 170 pp., 22 pl., 2 figs, geologic map.

Furcron, A. S., Dolomites and Magnesian limestones in Georgia: Ga. Geol. Survey, Info. Circular 14, 1942, (Brevard belt, pp. 6-11).

Hurst, V. J., Stratigraphy, structure and mineral resources of the Mineral Bluff quadrangle, Georgia: Ga. Geol. Survey, 1955, 137 pp., 56 figs., 12 tables, 6 pl., 2 diagrams.

LaForge, L., and Phalen, W. C., U. S. Geol. Survey Atlas, Ellijay folio (No. 187), 1913. (Out of print, see your libraries)

Maynard, T. Poole, Limestones and cement materials of north Georgia: Ga. Geol. Survey Bull. 27, 1912 (Piedmont marbles, pp. 109-128).

McCallie, S. W., Marbles of Georgia (Revised Edition): Ga. Geol. Survey, 1907, 406 pp., 34 figs., 10 plates, geologic map.

Pruitt, Robert G., Jr., The Brevard zone of northeasternmost Georgia: Unpublished Masters thesis in library of Emory Univ., 1952, 71 pp., geologic map (deals with marbles in Stephens & Habersham Counties). Sponsorship and field direction by Georgia Geological Survey.

CAMBRIAN AND CAMBRO-ORDOVICIAN

(Conasauga limestone and Knox dolomite)

As you may see from the Paleozoic column on the state geologic map about one-half of the entire Paleozoic sequence consists of Cambrian and Cambro-Ordovician rocks. There are three limestone and dolomite-bearing formations present which are from oldest to youngest; the Shady dolomite, limestone in the Conasauga formation and the Knox dolomite. The Shady dolomite is deeply weathered and poorly exposed and close to the Cartersville fault.

The most important rock for aggregate in the Lower Cambrian is the blue limestone of the Conasauga formation. This formation is widespread in the eastern and southern third of our exposed Paleozoic sequence. The formation consists of shale with limestone layers which are thin to thick. These beds can be traced with detailed work and constitute an important mineral resource of the district here indicated. Some of the beds are slaty and not entirely suitable for aggregate. Thus far, geologists who have mapped our Paleozoic rocks have not been sufficiently economic minded to map these important limestone beds in the Conasauga. We are now doing this and we hope that when we eventually prepare and publish the next state geologic map that we will have a good differentiation of rock types for this formation. The L & N Railroad from Cartersville into Tennessee follows important outcrops of this limestone. In general, they have not been developed and they represent a potential mineral resource of considerable importance. Also, they are north of Cartersville, our easternmost Paleozoic limestone, and along the Cartersville fault the crystalline schists and metagraywackes are thrust directly over them.

The Knox dolomite of Cambro-Ordovician age also is a very thick formation and is very widespread. It contains a considerable amount of chert and over the entire area it is usually deeply weathered, but may be mapped by the high chert content of the soil. This chert, with the accompanying fine siliceous material has been extensively used to improve unpaved roads. The Knox dolomite is a good stone for aggregate, but because of its deep weathering, there are few quarry sites available.

ORDOVICIAN, SILURIAN, AND MISSISSIPPIAN

These Paleozoic rocks contain numerous formations where limestone suitable for aggregate may be obtained. In the Ordovician, the Newala limestone is usually pure, is thick bedded, blue to blue-gray and finely crystalline. It crops out in narrow belts, for example, along the sides of Missionary Ridge, and occurs over the Rockmart shale north of Aragon, Polk County. The Murfreesboro limestone is quite variable in composition, generally blue, and compact, and locally contains some chert. It crops out notably around Chickamauga. The Mosheim limestone is compact, and dove-colored, pure, persistent, but rather thin. The Lenoir is a dark-gray, coarsely crystalline limestone, locally cherty. The Lebanon is thin-bedded, locally argillaceous. The Holston, often referred to as the Holston marble, enters Georgia from Tennessee, in a narrow belt just east of State Highway 71. The Lowville is a fairly pure, blue limestone, but the Moccasin is argillaceous and in many cases a mud rock. The Trenton limestone is very fossiliferous, thin bedded, blue-gray, and coarsely crystalline. The best exposures are reported from Lookout Valley (for distribution see Butts and Gildersleeve, 1948).

Limestones are essentially absent in Silurian rocks where the dominant rock types are sandstones and shales. There is very little Devonian in Georgia, the Armuchee chert being only about 50 ft. thick.

The limestones of Mississippian system are perhaps the most important of northwest Georgia for the purposes under discussion, and are at present generally classified most with the Bangor, and have not been separated in geologic mapping. They are rather similar in character, pure, generally gray to bluish-gray, coarsely crystalline, and thick bedded, representing the principal important source for aggregate. They are particularly well-exposed beneath the Pennington shale and Pottsville rocks around the edges of Lookout Mountain and in such places are quite accessible for quarrying.

POTTSVILLE

In the extreme northwestern corner of Georgia in Dade and Walker Counties are two outliers of the Allegheny plateau, known as Sand and Lookout Mountains. These mountains are capped with sandstone and shales of Pottsville age, which have been separated into several formations (Johnson, 1946). They are underlaid by the Mississippian Pennington shale beneath which is the great sequence of Mississippian limestones.

The Pottsville is composed of thick sandstone conglomerate and shale beds and the only rock which might be used for highway purposes would be massive sandstone and conglomeratic sandstone beds and flagstone, but they would not be suitable for aggregate.

References (Paleozoic Rocks)

Butts, George, and Gildersleeve, Benjamin, Geology and mineral resources of the Paleozoic area of northwest Georgia; Ga. Geological Survey, Bull. 54, 1948, 176 pp., 8 plates, geologic map and map of quarries, mines and prospects.

Furcron, A. S., Mineral resource survey of Floyd County, Georgia; Ga. Mineral Newsletter, Vol. XI, No. 1, Spring 1958, pp. 1-5.

Georgia Geological Survey, Geologic map of Georgia, 1939.

Johnson, V. H., Coal deposits on Sand and Lookout Mountains. Dade and Walker Counties, Georgia, U.S.G.S., 1946 (geological map).

Maynard, T. Poole, Limestones and cement materials of north Georgia;Ga. Geol. Survey, Bull. 27, 1912, 296 pp., 22 pl., 1 geologic map.

McCallie, S. W., Roads and road-building materials of Georgia; Ga. Geol. Survey, Bull. 8, 1901, 264 pps.

Smith, W. L., Geology of the Conasauga formation in the vicinity of Ranger, Georgia, 1958, M.S. thesis, Emory University sponsored by Georgia Geological Survey.

Stuart, Alfred W., Dolomitic limestone and shale on the Fairmount area; Ga. Mineral Newsletter, Vol. IX, No. 3, Autumn 1956, pp. 85-89 (colored geologic map). Resume of M.S. thesis, Emory University. Sponsorship and field direction by Georgia Geological Survey.

CRETACEOUS FORMATIONS

A band of upper cretaceous rocks crosses through the central part of the state, producing the inner marginal formation of the Atlantic Coastal Plain. The Tuscaloosa, Cusseta and Providence formations contain an abundance of sand and a great deal of sand and gravel, are taken from the Tuscaloosa formation between Butler and Columbus, especially in the vicinity of Junction City. The Tuscaloosa formation is famous for its bodies of white kaolin, but no quarry sites are known from this belt of Cretaceous rock which could produce a usable crushed stone. The Fall Line cities, Columbus, Macon, and Augusta depend upon quarries in the crystalline area. They could draw upon the Tertiary limestones also which are just south of the Cretaceous belt.

TERTIARY LIMESTONES

Introduction

The Coastal Plain is composed of Mesozoic and Cenozoic rocks. The oldest exposed rocks occur at its northern limits where the Upper Cretaceous Tuscaloosa formation crops out (see Georgia State geologic map). Above this formation are numerous other upper cretaceous rocks which are exposed particularly towards the southwestern part of the state. Thus, the Cretaceous outcrop is wedge-shaped and looses its belt-like character in the middle and northeastern extension where the Tuscaloosa formation is exposed beneath Tertiary rocks only in stream valleys and in narrow belts right against the crystalline rocks. These Mesozoic rocks contain valuable resources of white kaolin and sufficient deposits of sand to last us indefinitely, but they do not have consolidated rocks suitable for the preparation of aggregates for highway construction.

The remaining portion of the Coastal Plain is directly underlaid by Tertiary and Pleistocene formations. Over a large part of the central district, the Miocene Hawthorn formation is exposed. Its rocks consisting of fullers earth, sands, sandy fullers earth, cherts, thin beds of limestone and other types, do not seem to contain good limestones for the opening of quarries, and thus far no stone has been quarried in this district to my knowledge.

Pleistocene deposits (sands and clays) occur at the surface in a belt along the coast which is at least 40 miles wide, but extends up the major stream courses to points as far back as the fall line. Thus, our discussion of commercial limestones will now be confined to certain Tertiary formations where they are good enough and thick enough to quarry.

Useful Deposits

Although most of the known commercial deposits of limestone thus far discovered belong to only one of the Tertiary formations, several other formations are known to contain limestones which may at some date be used for road construction. The distribution of the following mentioned formations may be obtained by referring to the geologic map of Georgia.

Paleocene deposits represent the oldest Cenozoic rocks, thus they crop out fartherest West. They are represented in Georgia by the Clayton formation, which however, is usually very completely weathered and oxidized upon the outcrop. The lower part of the formation is a sandy limestone, but the unweathered limestone of this formation is mostly buried just southeast of its outcrop by later formations. Outcrops in southwest Georgia may be excellent limestone but rather soft for crushed stone.

The lower Eocene-McBean formation, estimated to be roughly 100 ft. thick, is generally sandy, but there are localities such as Shell Bluff, in Burke County where rather unconsolidated limestones and shell rock occur.

The important limestone producing formation in south Georgia is the Ocala limestone of Eocene age. Its outcrop is wide in southwest Georgia and it extends northeast as far as Wilkinson County. It is usually a white or cream-colored, pure limestone, the lower part of the formation somewhat sandy. The total thickness of the Ocala is not yet definitely known. In Twiggs County the extension of the Ocala is classified as a Twiggs clay and contains valuable deposits of Fuller's Earth. In the vicinity of Sandersville in Washington County, an equivalent of the Ocala is referred to as the Sandersville limestone (Cooke, 1943, p. 62). This is a light-gray, chalky limestone, somewhat sandy near the base.

The Flint River formation of Oligocene age occupies a considerable district in Georgia, forming a long, irregular outcrop belt just east of the Ocala limestone. It originally consisted of impure limestones and chert, but it is poorly exposed and deeply weathered. Thus, masses of fossiliferous chert residual clay, sand, and gravel, usually indicate the presence of the underlying Flint River formation. There are few outcrops available and at present no known quarries in this stone.

The Tampa limestone of Oligocene age is indicated by Cooke (Cooke, 1943, geologic map) as a narrow belt cropping out along the prominent escarpment which separates Flint River formation from the Hawthorn and which extends in Georgia from the junction of Flint and Chattahoochee Rivers, northeast, to the vicinity of Sylvester. The formation has a possible thickness of 100 ft., is generally weathered down, but where observed, consists of soft to hard limestone, locally sandy.

Lower Miocene limestone regarded as Tampa crops out along the major streams south of Turner, Irwin, Coffee, and Ware Counties. In general, these limestone beds are reported to be thin and not adapted to the opening of large quarries.

In many cases, both hard and soft limestone may be encountered in the the same quarry. In former years, there was no use for the hard limestone, but the softer varieties were used, combined with the natural composition of soil roads to form a sort of macadam. Where quarries are opened for the purpose of obtaining aggregate, it is well to determine the expected proportion of hard and soft limestone available and this is one of the important reasons for drilling such a deposit. Another reason for drilling, especially if the limestone is desired for purposes other than aggregate is to determine if the lower sandy part of the formation is present. In such deposits, naturally the hard lime will crop out and the areas of soft lime may not be observed until the deposit is drilled on centers.

Teritary limestones of south Georgia are excavated near Bridgeboro, in Mitchell County, at Clinchfield for the manufacture of Portland cement in Houston County, and at Armena in Lee County. When we consider the need for agricultural lime and for aggregate over this very large area of Georgia, it is surprising that there has been so little development of these extensive limestone deposits, particularly, the Ocala limestone.

In developing the limestone, it is best to select a locality where the stone has a minimum of overburden and is at or close to the surface. Good stone may be seen around the margins of sinks in very many places, but at such localities, the limestone is very porous and full of solution cavities, many of which contain soil. Outcrops are abundant in the banks of rivers and creeks, but as a rule these are not desirable localities because these limestones are all quite soluble and will contain solution cavities to water level. Thus, if quarries are opened too near the stream large inruptions of water may be expected. By checking the depth of overburden away from stream courses in may cases it would be possible to select good quarry sites and also have available water supplies. Such properties should be carefully prospected down the slope. In prospecting for limestone, the many new roadcuts made by the State Highway Department often disclose the presence of good stone. Where limestone is found under will-drained areas between streams and close to the surface, it can be removed for considerable depth before water is encountered, and below the water table only a reasonable amount of seepage water might be expected.

References (Tertiary Limestones)

Brantly, J. E., Limestone and Marls of the Coastal Plain of Georgia: Ga. Geol.

Survey, Bull. 21, 1916, 300 pp., geologic and limestone location map. (Out of print, see your libraries).

Cooke, C. Wythe, Geology of the Coastal Plain of Georgia: U.S.G.S., Bull. 941, 1943, 121 pp., geologic map. (Out of print, see your libraries).

Furcron, A. S., and Perry, Eugene C., Jr., Mineral resource survey of Crisp County, Georgia: Ga. Min. Newsletter, Vol. XI, No. 2, Summer 1958, pp. 37-44.

----------, Limestone of Lee County Georgia: Ga. Min. Newsletter, Vol. XI, No. 4, Winter 1958, pp. 111-118, illustrations and limestone location maps.

Georgia Dept. of Mines, Mining and Geology, Geologic Map of Georgia: 1939.

Veach, Otto, Geology of the Coastal Plain of Georgia: Ga. Geol. Survey, Bull. 26, 1911, 463 pp., 30 pl., 2 maps. (Out of print, see our libraries).

HAWTHORN FORMATION

Much of the central portion of the Coastal Plain of Georgia is covered by this formation which is regarded as having about 200 ft. maximum thickness. Its composition is not favorable for quarry sites or for limestone deposits. Much of the formation consists of sandy fuller's earth, sands, sandstones and some chert. In the lower part of this formation limestones occur and these limestones crop out in southern Georgia along the major southward flowing streams which have cut through the upper part of the formation to expose the limestone. Thus, in this section of the state prospecting for limestone can be done along the stream courses. In most places prospecting will be necessary in order to determine the nature and thickness of the stone which may range from hard to soft in the same outcrop. There is no important production of stone at present from this district.

PLIOCENE AND PLEISTOCENE

The eastern coastal area of Georgia covered by rocks of this age do not offer opportunities for the opening of quarries. The deposited materials are argillaceous, calcareous, phosphatic clays and sands, but no hard limestones are known. Beneath these deposits, the Hawthorn formation occurs. Some thin beds of soft marls and limestones have been described (Brantley, 1916) along the St. Marys River in Charlton and Camden Counties.

CONCLUSIONS

In the Coastal Plain of Georgia which covers approximately 60% of the state, the principal source of stone will be Tertiary limestones, particularly the outcropping belt of Ocala. These limestones are not as hard as present requirements for aggregate generally demand, but with judicious planning, a very large supply of limestone can be obtained for all of the common uses to which this rock is applied.

In the crystalline and highland area of the state (approximately 30%) granites, foliated granites and migmatites are producing at present most of the commercial crushed stone. Along the southern marginal belt of the Piedmont, metavolcanic rocks also are supplying a considerable amount of aggregate. The granite rocktypes will remain in the future a dominant source of supply for this district.

In the Paleozoic area which includes all or parts of about ten northwestern counties (10% of the state) the principal rocks have been and probably will remain the Conasauga limestone, Mississippian limestones, and to a lesser extent, Ordovician limestones.

DESIGN CONSIDERATIONS IN THE TREATMENT OF SOFT FOUNDATIONS

By

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OHIO DEPARTMENT OF HIGHWAYS

This paper will deal with treatment of peat and associated soft clays which are common to the glaciated areas of the United States. Our concern with these problems has been greatly increased in the past several years because of the necessity for construction new high type roads on all new locations such as the Ohio Turnpike across the northern part of the State and the new Interstate Route from Columbus in the central part of the State to Cleveland on the shores of Lake Erie. The high geometric standard necessary for roads of this type and the desire to locate them in such a way as to do the least damage to existing homes, factories, and the other high value facilities has tended to push these locations out into the hills, brush, and marshes. On the new Interstate road between Columbus and Cleveland, treatment of soft foundations, peat deposits, etc., is costing in the neighborhood of $2,000,000.

ORIGIN OF PEATS

Peat and soft clay foundation in Ohio are associated principally with the glacial drift of the most recent Ice advance, namely, the Wisconsin, Fig. 1. In Ohio, this covered the northwestern three-fourths of the State. The glacial drift in Ohio and in neighboring States had a general tendency to fill up the valleys and level off the hilltops, destroying the old established drainageways and leaving a constructional type of terrain which in many areas is made up of hummocky ground and undrained depressions. The formation of peat in these undrained areas is well illustrated in the following sketches taken from Ohio Geological Survey Bulletin No. 16 on Peat Deposits by Alfred Dachnowski.

Figure 2 illustrates a lake with open water at the right and with a vegetative encroachment which has already filled the shallower portion of the lake at the left. Each type of vegetation has its preferred habitat depending on depth, turbulence, and chemical composition of the water. In the open water there is practically no plant growth; moving into the shallow water and across the filled portion of the lake we find first, the marginal plants such as the lilies, the shore succession consisting of submerged and semi-aquatic plants such as the reeds which project their single leaves above the surface of the water, and then the Bog Succession consisting of bog meadow, bog shrub, and bog forest.

Through the normal processes of growth, partial decay, and disintegration, the bottom of the pond is gradually filled with decayed vegetative matter. These deposits are acted upon by the normal agencies of currect, wave action, and sedimentation. During periods of turbulence, some of the finer portions of decayed matter are carried out into the open water, together with silt and clay mineral soils washed in from nearby higher ground to be deposited as sedimentary peat and organic clays. Local or seasonal variations in run-off, current, and wave action will greatly alter the type of material being deposited on the lake bottom, thus explaining the considerable complexities and variations which are associated with these deposits.

As open water depth is reduced by sedimentary deposition, the habitat is modified to a point at which aquatic and semi-aquatic plants may thrive and these plants, therefore, migrate into what was once open water; in the meantime, these plants have been growing, dying, and building up a mat of semi-fibrous and sedimentary peat, thereby reducing the water surface in their locale. This reduction modifies the habitat and provides a condition acceptable to the Marsh Vegetation. These marsh grasses, reeds, and cane in turn build a deposit of fibrous peat, modify the habitat, and provide a condition favorable to the bushes, conifers, and eventually the deciduous trees which form the woody peat. This repeating process gradually fills the lake.

It should be emphasized here that the undrained depression is still in existence. It has been loosely filled and covered but has not been destroyed. The lake can be restored by removing the peat such as would be the case if peat were excavated to make way for a highway embankment. The accumulation of peat occurs because the plant remains are protected from the normal weathering processes by submergence. Because they are submerged, they are protected from oxidation and from aerobic bacterial action. Also, the acid contamination of the lake, resulting from the decaying vegetation, reduces the anaerobic bacterial action, thus extending the time required for decomposition to many thousands of years. Recognizable forms of plant and animal remains are common and, in at least one case, a well-preserved human being was found.

CLASSIFICATION OF ORGANIC MATERIALS

Considering the origin of these organic materials, it is evident that many variations in type can be expected. For purpose of identification, Ohio has adopted the following classification:

Organic Clays -

These are mineral soils containing some interspersed organic materials. The organic content ranges from 4 to about 12 per cent and the liquid limit from 35 to 100. They are usually soft, gray to black in color, and have water contents equal to or greater than their liquid limits.

Sedimentary Peat -

This is a sedimentary deposit in open water of an amorphous residue of partially decayed organic matter together with a variable amount of mineral soils. As in the organic clays, horizontal and vertical variations are explainable by the laws of sedimentation. The organic component of the material ranges from 12 to 50 per cent, liquid limit is usually on the order of 100 or more, and water contents are usually higher than the liquid limit. The color ranges from gray to green and occasionally black.

Fibrous Peat -

This material is an accumulation of the recognizable remains of semi-aquatic and shore succession plants. It is characterized by the presence of short fibers derived primarily from the alteration of the grasses and sedges. Atterburg limits are usually not run of this type of peat. Water contents are usually high, ranging from 100 to 1000 per cent. The color is usually light to dark brown depending on the degree of decomposition.

Woody Peat -

Woody peat is derived primarily from trees and brush and is identifiable on the basis of the large amount of partially decomposed wood fragments found in it. The color is usually black and the water contents have about the same high range as the fibrous peats.

Fine Textured or Compact Peat -

This is a dark brown to black decomposed vegetative material in which the decomposition has advanced far enough to obliterate nearly all traces of the former plant. The color is usually darker than that of the fibrous or woody peats from which it was derived. Water contents are usually in the 100 to 1000 per cent range as they are for the fibrous peats. This structureless material constitutes the largest percentage of our usual peat deposits.

Marl -

In its pure form, marl is fresh water calcium carbonate derived from plant and animal remains. It is usually associated with various amounts of organic material or mineral soil.

Often the above described organic soils are covered by mineral soils which have been washed in from adjacent higher ground. The presence of these more normal soils over the soft organic materials presents a special problem both in the treatment and in the identification of the soft foundation area. Depth of this wash cover may range from 0 up to 20 feet.

TREATMENT OF SOFT FOUNDATIONS

There are a number of possible ways in which the soft foundation area may be handled. The selection of the most appropriate treatment will depend on the type of improvement being made and on the specific nature of the bad foundation material. Briefly, these alternates are as follows:

Avoidance -

Needless to say, the most desirable procedure is to adopt an alignment which will miss all of the more serious problems. This is of course not always possible; however, in the early stages of plan development consideration should be given to possible soil conditions and if an area is suspected to include some poor foundation materials, it should be investigated sufficiently to get some idea of the magnitude of the problem.

Floatation -

Where a very low embankment loading is possible, or where the general characteristics of the road do not warrant the high expense required to stabilize the soft material, the road may simply be built across the deposit using a light weight fill material placed directly on the existing ground or on a timber mat. Figure 3 is a photograph of a recently constructed roadway consisting of a cinder fill about 3 feet high which lies on about 40 feet of fibrous and sedimentary peat. In this case the old road did go around the peat deposit, but it had four right angle turns in a distance of less than 1,000 feet. A short distance west of this spot, the present road is floated over deep peat for a distance of 2300 feet. Although the surface is a little wavy, the alignment is straight and the traffic is not confronted with hazardous curves. It was, therefore, considered worthwhile to straighten this east section by going across the bog. The scalping which is usually done before placement of a new fill was waived here, the ground surface was cleared of brush, but the grass and other vegetative cover which would not project through the fill was left in place. As is to be expected, there has been some settlement of the new fill, about 0.3 of a foot to date. However, we think it will be more economical to do a little surface maintenance in this area from time to time rather than to spend the $150,000 which we estimated would be required to treat the peat.

The example given above applies to a secondary road and to a deep peat deposit. We also have the condition of a shallow layer (2 to 5 feet) of compressible organic soils overlaying sand and gravel. Even on primary roads in this case, we have been successful in treating these soils simply by using free draining granular material in the lower two feet of the fill. The presence of granular material above and below the compressible soil permits rapid drainage of the unstable soil and complete consolidation prior to placement of the pavement.

Total Excavation -

When soft materials such as peat occur immediately beneath the surface of the ground in depths on the order of 10 to 15 feet, it can be totally excavated and replaced with suitable fill, see Figure 4. Since there is likely to be open water in the hole from which peat was removed, it is our usual practice to use granular material for the backfill and to permit placement of this material by the method of end dumping.

Partial Excavation and Displacement -

This is sometimes referred to as the Michigan Method. It is applicable to the peat soils since they are usually sufficiently fluid to be displaced under a suitable fill loading. The procedure consists of excavating a trench in the peat ahead of the advancing fill to a depth of 10 to 15 feet. When depth of peat becomes greater than the trench the weight of the granular fill plus surcharge displaces the soft material under the fill into this trench from which it can be removed by excavation. When the trench has been cleaned out, the fill is brought forward into it by end dumping of granular material. Thus, by alternate trenching and filling, the embankment is advanced across the deposit. Peat deposits up to 45 or 50 feet can be successfully treated by this method. The method has also been used on deposits with depths up to 70 to 75 feet, which appears to be about the maximum depth at which peats occur in Ohio. However, there has been some movement of fills placed by this method on these deeper deposits. Our experience indicates that 50 feet is about the maximum depth of soft material which we can totally displace. However, we have not found an entirely suitable alternate method.

For both the total excavation case and for the case of partial excavation and displacement, we have found it expedient to determine the volume of material excavated by measuring the volume of granular fill required to fill the hole. Our plans provide two items of granular borrow in the case of the excavation and displacement method.

(1) Per cubic yard of granular borrow including the cost of excavation.

(2) Per cubic yard of granular borrow.

The total quantity of granular material is determined by cross sectioning the borrow pit from which it is taken and the quantity of granular borrow above the original ground line is determined by cross sectioning the fill. The amount of granular borrow including the cost of excavation of unsuitable soils is found by subtracting the amount used in fills from the total quantity.

Vertical Sand Drains -

Vertical sand drains, see Figure 5, can be used in deep soft clays and sedimentary peats which are not fluid enough to be handled by the displacement method, but are too soft to support the fill as they are. The sand drains tend to expedite drainage of water from the soft foundation by providing closely spaced filter wells through the foundation. As the fill load is added, water which is squeezed out of these soils can escape upward through the sand drains into the sand blanket which is provided at the base of the embankment and thence out to the toe of the fill slope. By this method, rate of consolidation of the soil is hastened and the strength of the soils increases so that the foundation can support the final loading. Naturally, this procedure requires thorough laboratory testing to develop the design and careful control of the rate of fill loading during construction, plus a surcharge and waiting period after the embankment is completed to assure that the ultimate settlement is obtained before the pavement is placed.

Structure -

The final alternate is simply to bridge the area with some sort of structure supported by piling driven to firm subsoil. Figure 6 is a view showing the pile and concrete cap structure used to bridge a soft foundation area near Mansfield, Ohio, on a recently completed project. A structure similar to this was also used at one of the deep peat bogs on the Ohio Turnpike.

EVALUATION OF CONDITIONS

With the data available on foundation conditions and the above described alternate treatments in mind, the designer must select the method of treatment which is most appropriate for his particular problem. Factors which will have to be considered in the selection are:

Type of Improvement being Made -

Naturally, a little more tolerance can be permitted in allowable settlement on a low type, secondary road than on a high type expressway. Further, a failure of the secondary road, although undesirable, is by no means as completely unacceptable as the failure of a prime arterial highway.

Cost of Method of Treatment -

All other factors being the same, the least expensive treatment which will give satisfactory performance under the conditions of service should be selected. To give a rough idea, we have computed cost of construction of a section of four-lane highway over a soft foundation 30 feet deep and 500 feet long. For this problem, we have assumed that the foundation could be successfully treated by either excavation, vertical sand drains, or by building a bridge. Cost of the alternates is computed below. Unit costs used in this analysis are based on Ohio experience in the past year.

A. Vertical Sand Drains -

28,200 Lineal Feet 16" Vertical Sand Drains

at $1.70 Per Foot = $48,000

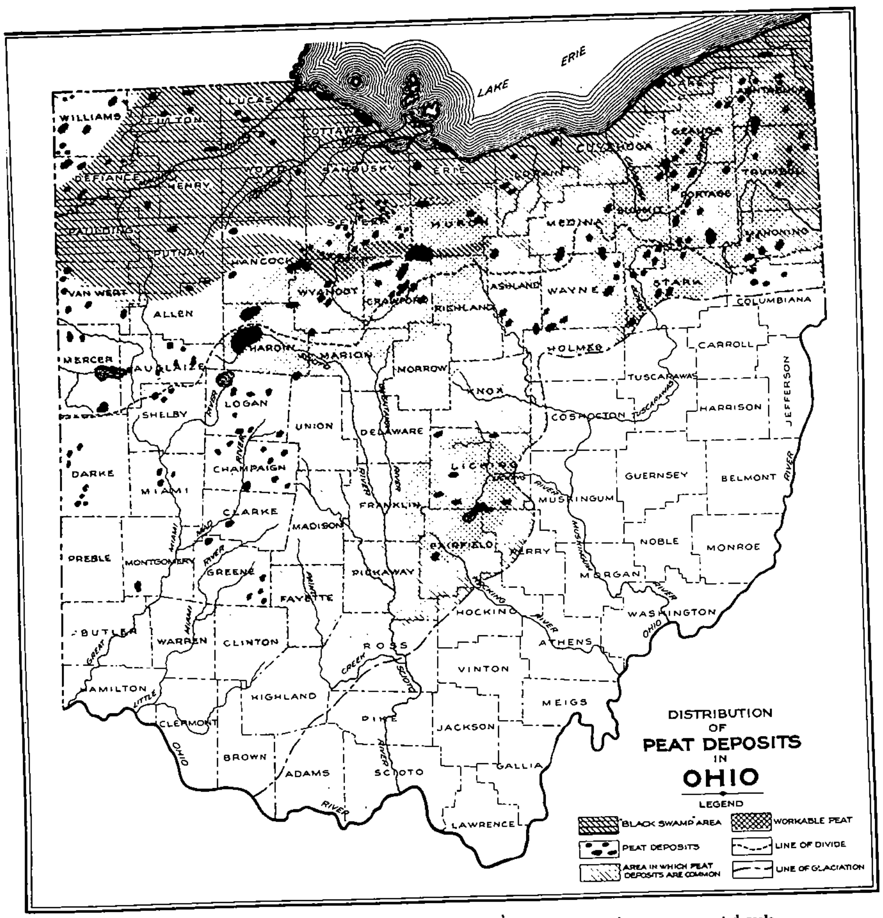
	3' Deep Sand Blanket, 15,000 Cubic Yards at \$3.80 Per Cu. Yd.	= \$57,500
	10' High Surcharge Place and Remove 20,750 Cu. Yd. at \$1.20	= \$24,900
	Total Sand Drains	\$130,400
B.	Partial Excavation and Displacement -	
	113,325 Cu. Yd. Granular Borrow Including the cost of Excavation at \$2.00	= \$226,600
	106,670 Cu. Yd. Granular Embankment Minus cost of Regular Embankment at \$0.60	= \$64,000
	20,750 Cu. Yd. Surcharge Place and Remove at \$1.80	= \$37,400
	Total Partial Excavation and Displacement	\$328,000
C.	Structure -	
	Twin Bridges 33' Wide out to out, structure cost Per Sq. Ft. of Deck	= \$18.00
	33 x 2 x 500 = 33,000 Sq. Ft. at \$18.00	= \$595,000
	Less cost of normal embankment 106,670 Cu. Yd. at \$ .60 = \$64,000	
	Less cost of pavement, paved shoulders, etc. 500 x \$45 = \$22,500	
	Less cost of guard rail 1000' at \$3.00 = \$ 3,000	
	Total Savings by Bridge \$89,500	\$89,500
	Total Structure	\$506,500

Adaptability to Soil Conditions Encountered -

Most important of all, the method selected must be applicable to the specific questionable foundation soil. A surface deposit of fibrous or woody peat would be so weak that drainage by the use of filter wells or sand drains would be relatively ineffective. Building an embankment of moderate height, even at a very slow rate, would undoubtedly result in failure by lateral displacement. This extremely weak material would either have to be replaced by stable foundation soil using one of the excavation methods or the area bridged. Figures 7, 8, and 9 illustrate an area of deep peat treated by the partial excavation and displacement method.

On the other hand, a deep deposit of sedimentary peat and organic clay, which is overlaid by washed-in mineral silts and clays is likely to be so stiff that treatment by partial excavation and displacement would not be satisfactory, since it is probable that much of the material would not displace and would leave irregular masses of compressible material in the foundation thus permitting large differential settlement. Under these conditions, vertical sand drains would perhaps be the most effective treatment, since they would expedite the foundation drainage and permit construction within a reasonable time limit without failure of the foundation by partial displacement. Figures 10 and 11 illustrate a deposit of organic clays, and sedimentary peats treated by the use of vertical sand drains.

Only with a thorough knowledge of the extent and type of material, a careful evaluation of the economics of the possible alternate treatments, and of the allowable settlements in the facility being constructed, can the best treatment for the specific problem be selected.



Map showing distribution of peat deposits in Ohio. The deposits are represented on an exaggerated scale.

FIGURE 1 - Map showing distribution of peat deposits in Ohio and the glacial boundary. From Ohio Geological Survey Bulletin No. 16 Peat Deposits.

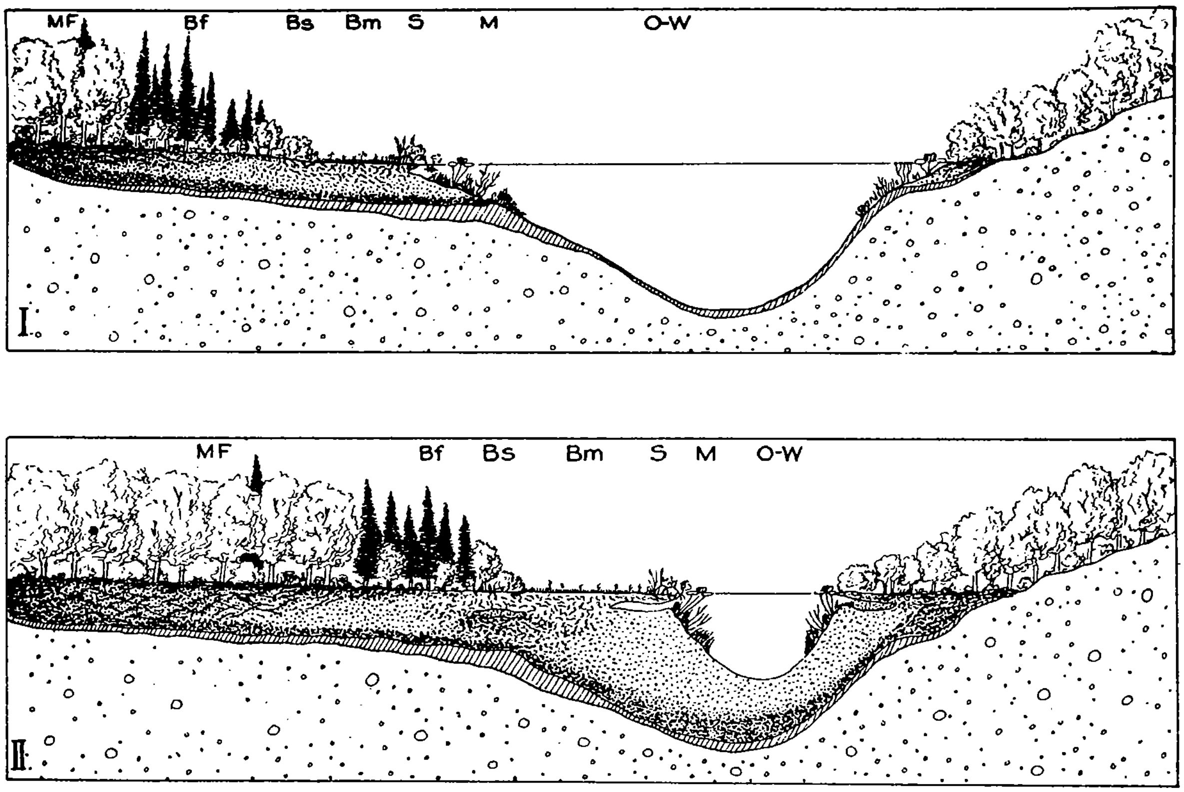


FIGURE 2 - Showing formation of peat in an open Lake by succesive encroachment of various types of plants. From Ohio Geological Survey Bulletin No. 16 Peat Deposits.

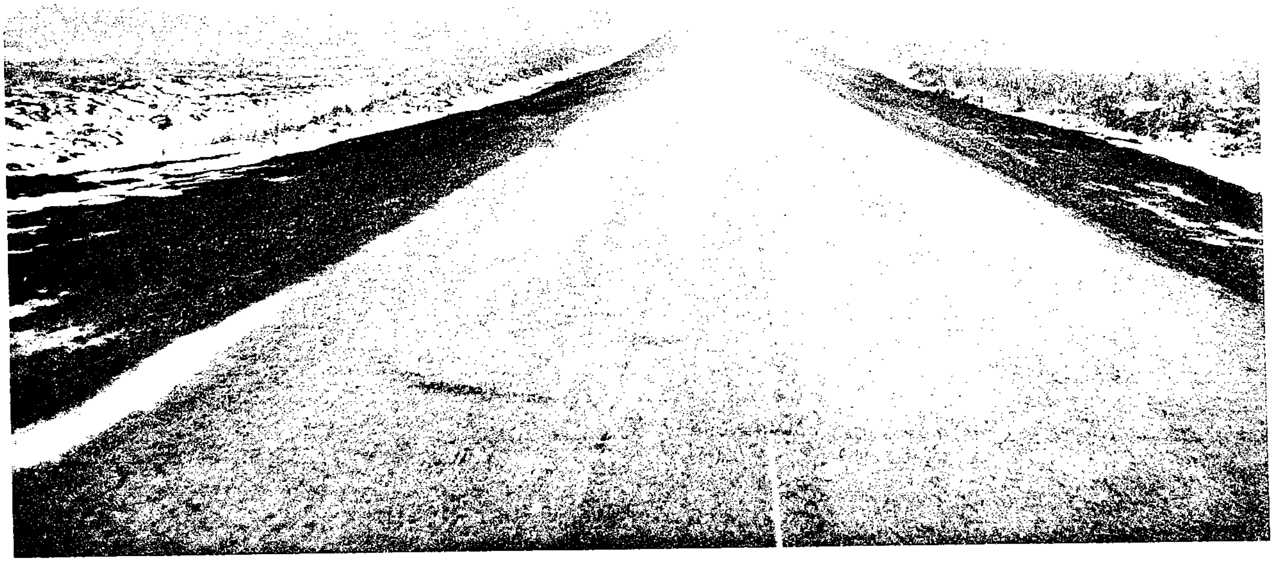


FIGURE 3 - Secondary highway "floated" over 40 feet of fibrous and sedimentary peat. Note low height of fill and the use of light weight cinders.

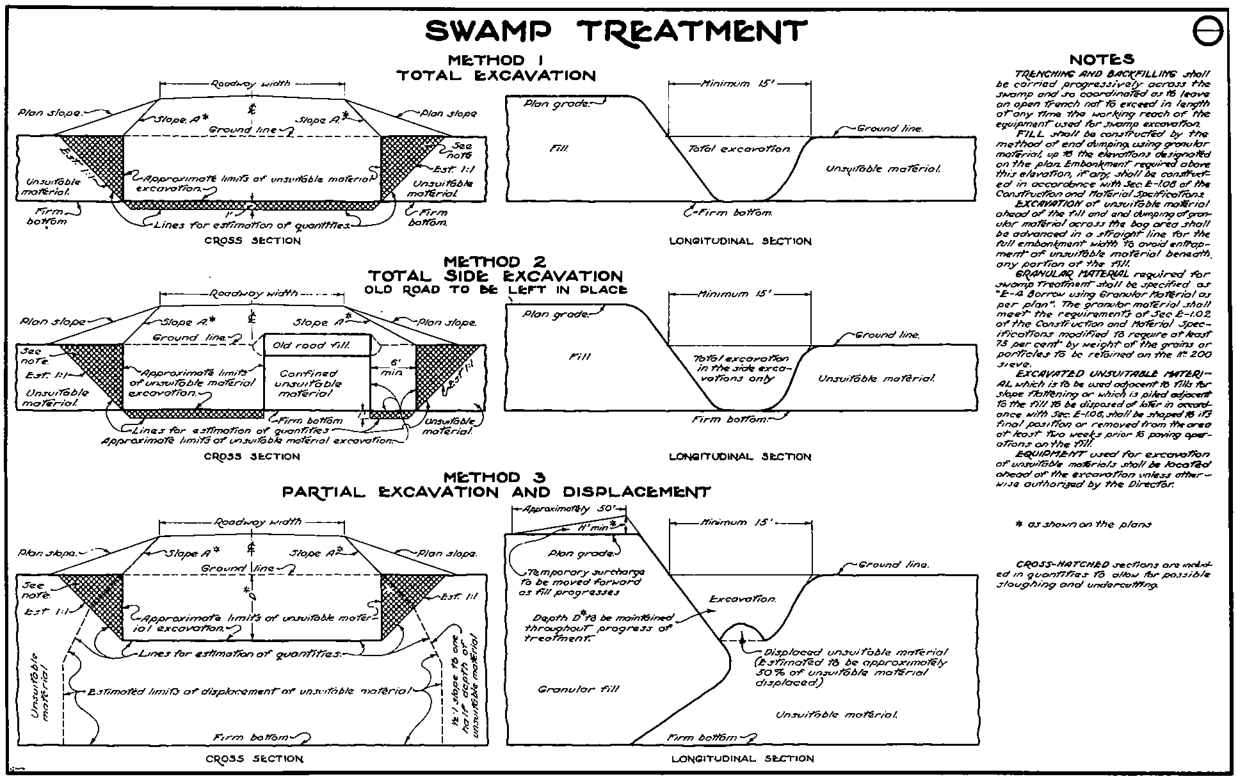


FIGURE 4 - Standard swamp treatment drawing covering total excavation and partial excavation and displacement methods.

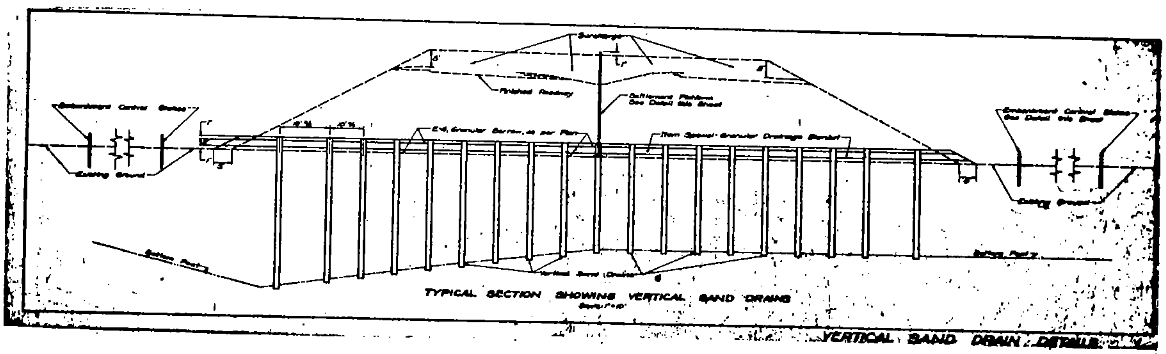


FIGURE 5 - Typical section showing principal features of the sand drain Method of treating a soft foundation.

FIGURE 6 - Pile and concrete cap structure used to bridge a deep deposit overlain by mineral soils on an important urban by-pass job.

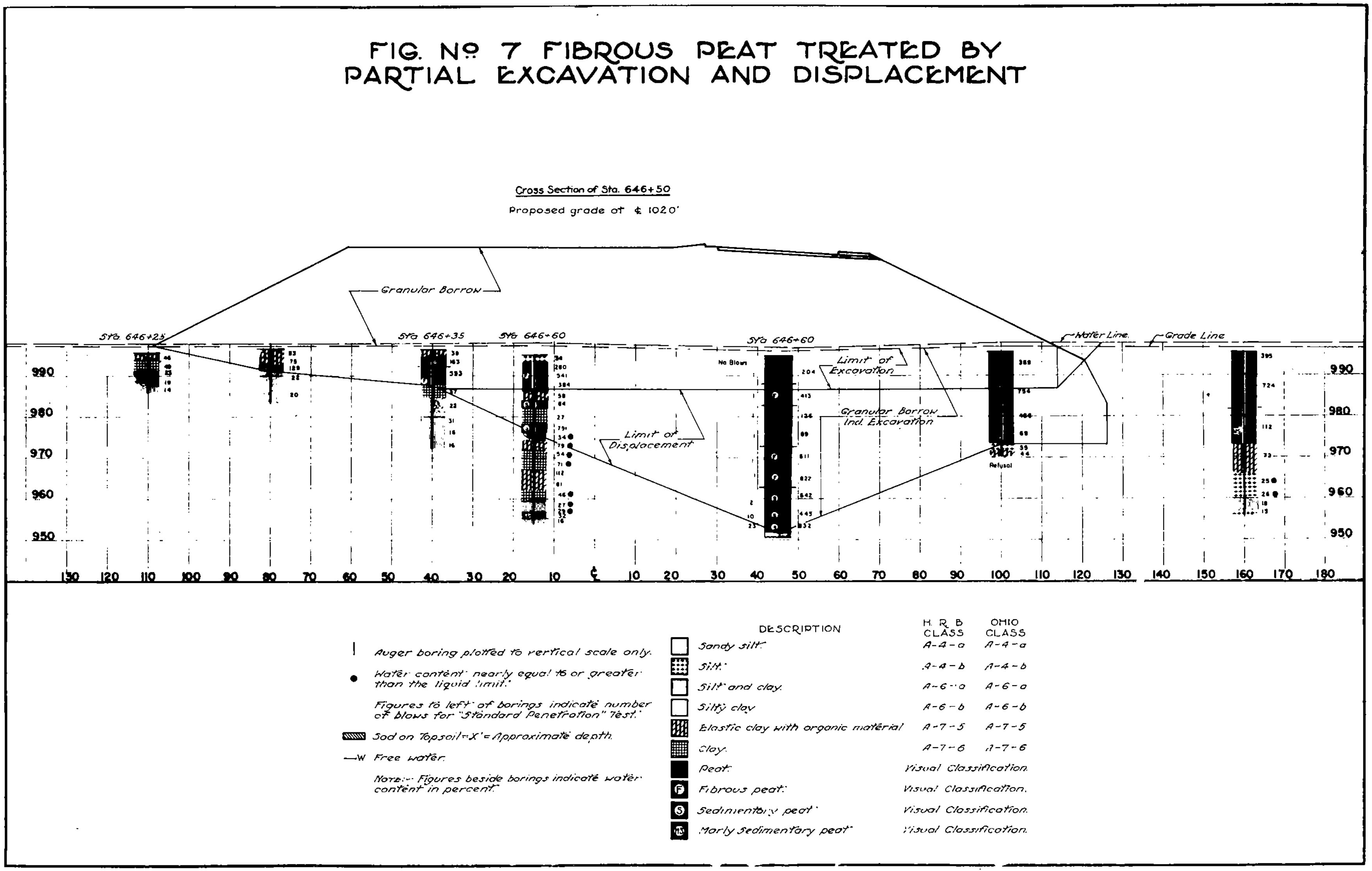
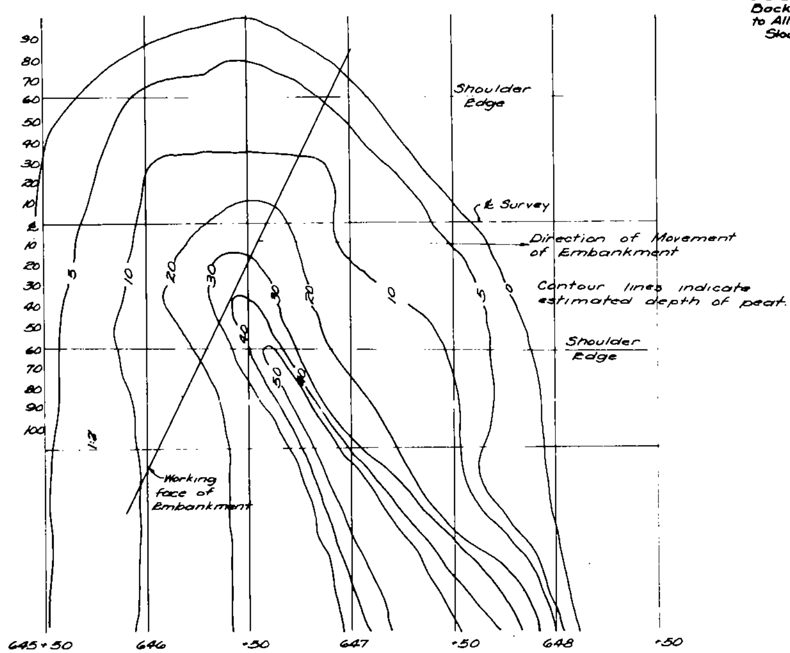


FIGURE 7 - Test boring logs and construction template showing treatment of deep peat deposit by method of partial excavation and displacement.

THE TOTAL EXCAVATION METHOD WILL BE USED BETWEEN STATIONS 645+5C AND 646+CC AND STATIONS 647+5C AND 648+00, WITH LIMITS TO BE VARIED DURING CONSTRUCTION TO FIT FIELD CONDITIONS.

TO ASSURE CONSOLIDATION OF UNDERLYING SOFT MATERIALS, NO PAVEMENT SHALL BE PLACEC BETWEEN STATION 645+5C AND 648+CC WITHIN 9C DAYS AFTER COMPLETION OF THE FILL.

WITHIN SEVEN DAYS AFTER COMPLETION OF THE FILL, THE STATE HIGHWAY TESTING LABORATORY SHALL BE SO INFORMED SO THAT BORINGS MAY BE TAKEN TO DETERMINE THE EFFECTIVENESS OF THE PEAT DISPLACEMENT.



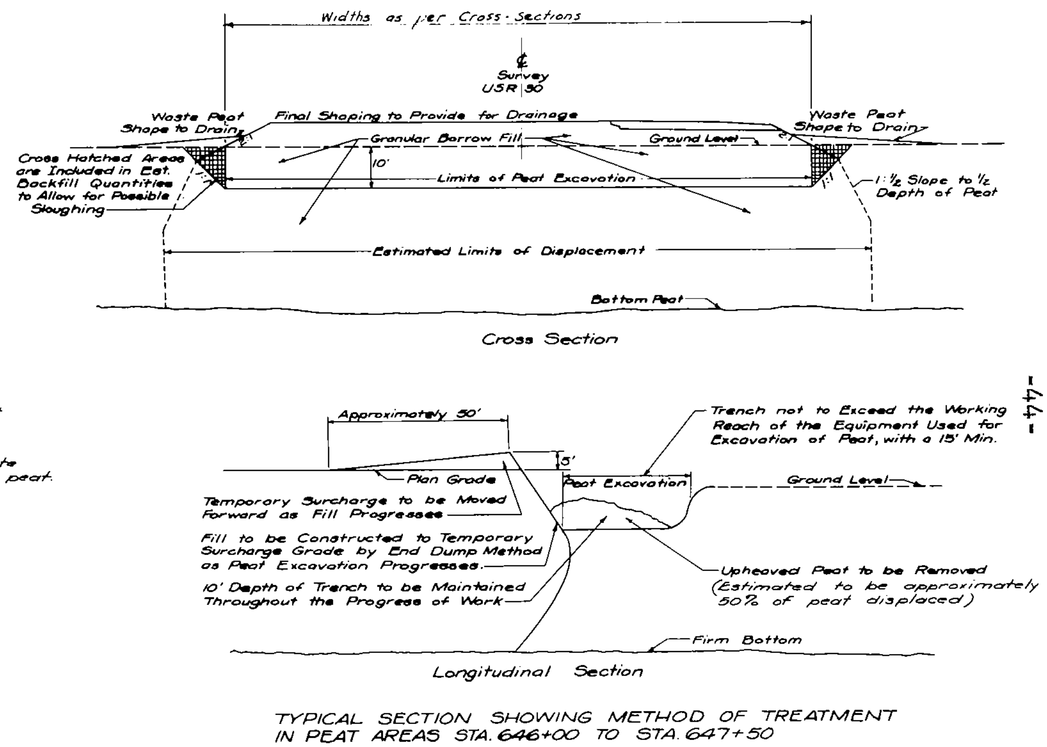


FIGURE 8 - Contours on bottom of peat (left) and details of embankment construction (right) for peat deposit shown in Figure 7.

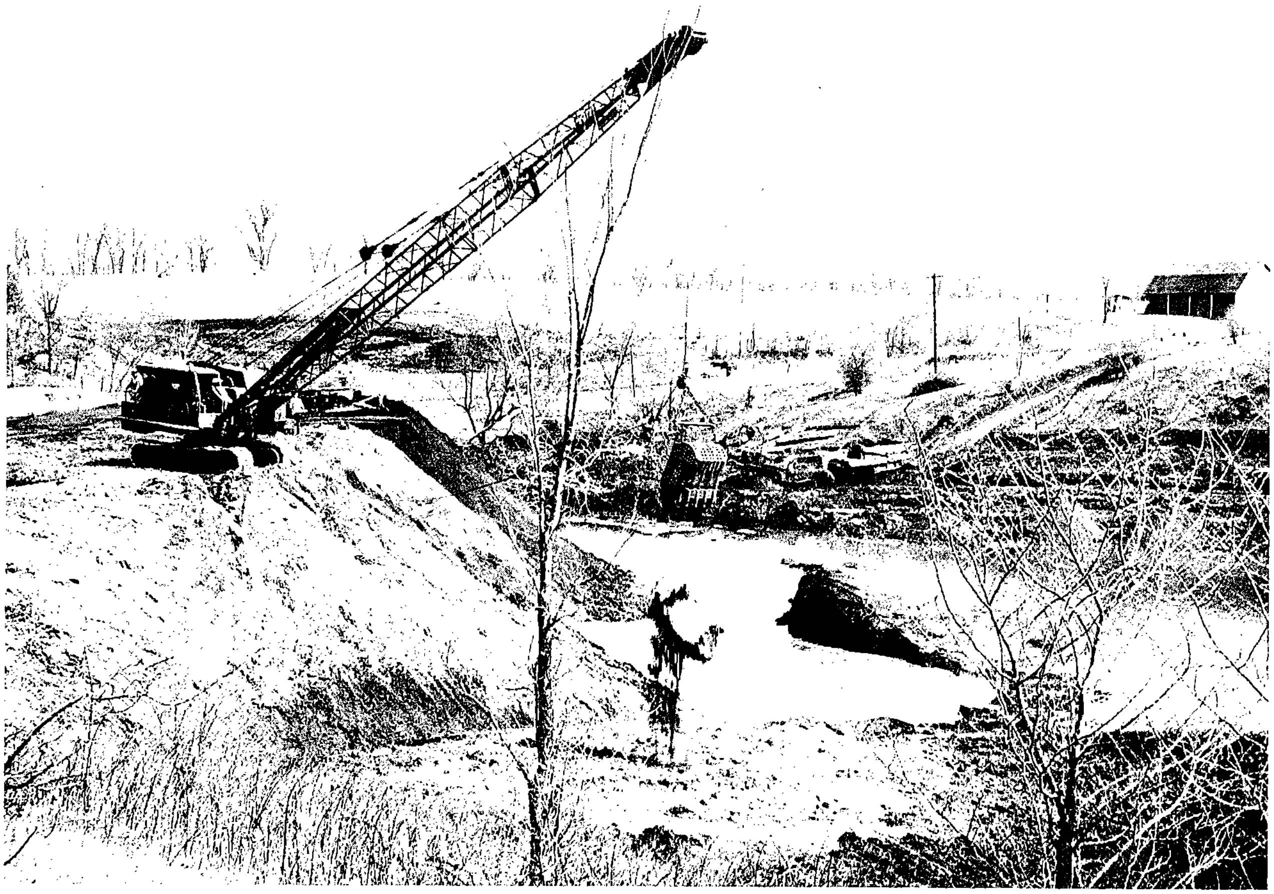


FIGURE 9 - Excavation of peat ahead of fill (foreground) and advancement of fill by end dumping granular material (background).

FIG. No. 10 SOFT FOUNDATION AREA TREATED BY VERTICAL SAND DRAINS

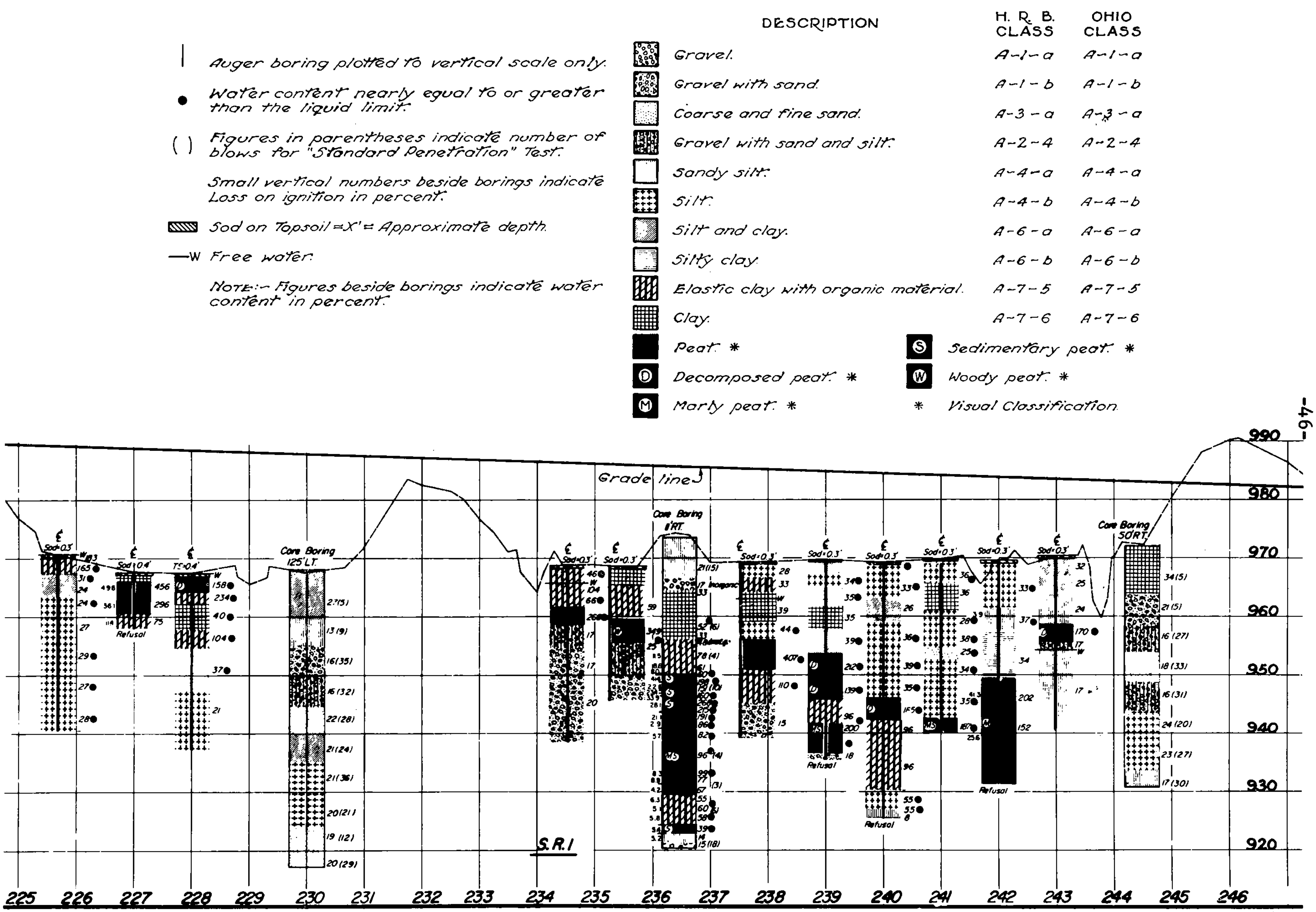


FIGURE 10 - Soft foundation area treated by the use of vertical sand drains.

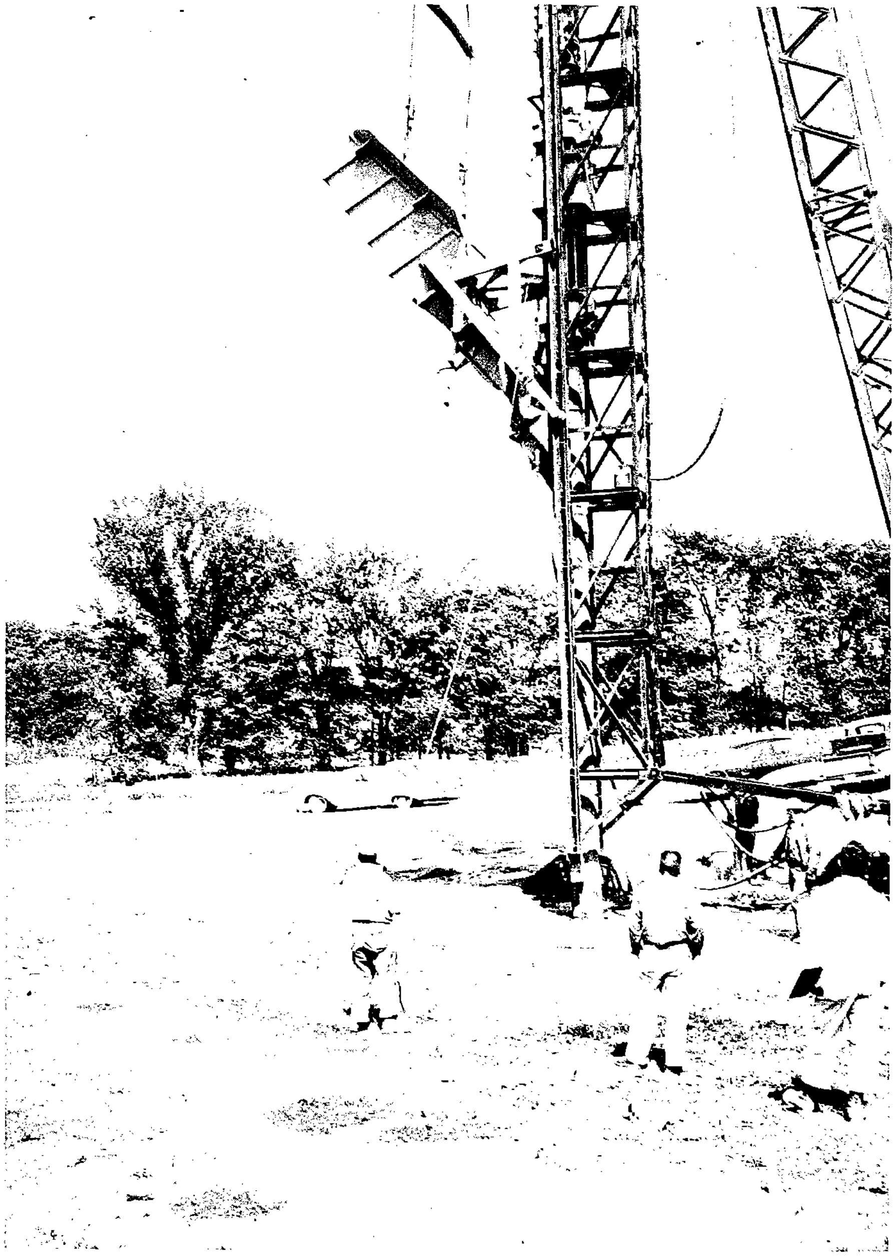


FIGURE 11 - View of mandrel and skip used for driving and filling sand drain.

THE USE OF GEOLOGICAL INVESTIGATION IN FOREIGN CONSULTING WORK

By

Charles M. Upham, Consulting Engineer

The title of this paper connotes two things: the first geology and the second roadbuilding.

In reference to road building, there are certain standard procedures that are followed regardless of whether the work is done in this country or in a foreign land. After a program has been outlined with certain roads to be built, the first step is the location of the road. The next step is the consideration of what materials will be used in the construction of the road.

The geologist is important in both of these steps. The importance of the use of geology in road construction is very evident by the fact that about one half of the State Highway Departments in this country have from one to ten geologists on their staff. All but one State has the equivalent of a geological survey, although the names of the organizations vary. All State Highway Departments utilize the geological maps and data produced by these State Organizations as well as those of the U. S. Geological Survey. The geologists employed by State Highway Departments augment, rather than duplicate, this geological information.

This is not a new role for the geologist, although at present it is being greatly expanded, and rating greater importance. Much of the data now being furnished by the geologist was previously furnished the Highway Engineer in a cruder and less accurate manner.

The efficiency and economy of the earth work involved in any road program will depend to a large degree upon the accuracy and extent of the information supplied by the geologist to the Highway Engineer and Contractor.

Geological materials are used directly in the construction of many highway parts, such as foundations, bases, surface courses and structures. The ability to locate suitable materials in sufficient quantities from information supplied by the geologist will effect both the progress and economy of a road program.

In foreign countries, it is the exception, rather than the rule, to have any geologic maps and this information must be obtained and produced by the geologist. We have found considerable activity in the preparation of maps by the Army Map Service and these, of course, have been very helpful for location designations.

One of the most interesting participations of the "geologist in a road program" was in Egypt where one of your own members, Mr. W. T. Parrott, was in charge of obtaining, recording and presenting the geologic data that was to be used in connection with the comprehensive road program of the country.

Most of you who have had experience with deserts know that as a rule the desert is a road builder's paradise and practically almost any kind of road material can be found in the desert. The Delta area of Egypt was bounded on each side by deserts and material that has recently served exceptionally well in road building had been available in these deserts for the past hundreds of years - yet hardly ever used. The probable reason for this was because the geologic information as to its quality and location had not been presented to the road engineer.

A comprehensive geologic study of the desert area showed that there was an abundance of aggregates that were perfect for road construction. A thorough knowledge of this area showed that in many instances the aggregates had occurred in nature all classified and ready to use. In some locations there were graded gravels from 2" down to sand. In other areas there were pieces of gravel large enough to produce crushed stone, and in many areas there was fine gravel ranging from 1/2" to 1/10" that was perfect for bituminous cover material and this had already been classified and prepared when it was originally laid down or by the subsequent winds. In addition, there was sandstone and limestone.

A geologic study of this area showed that we were working on the West Terrace of the Nile and the only thing lacking to furnish practically all of the road material that Egypt would ever use was that of transportation. In order to reduce transportation, a further study was made for aggregate materials and it was found that the East terrace of the Nile, together with a flood plain, contained even more and better materials than had been found on the West terrace. Samples of all of these materials were sent to the laboratory for confirmation and a map was produced by Mr. Parrott showing the location of all materials throughout the Delta and adjacent deserts that were satisfactory for road building.

As you may already know, the Delta of Egypt resulted when a deep estuary had been filled up with the silt from the Nile and being located a long distance from the source only the finest material was deposited in this area. It was decidedly a silt and clay, and while it happened to be excellent for farming purposes, it was practically useless for road building without the addition of coarser aggregates. It was found in the Laboratory that this material, when mixed with the proper amount of coarser sand or gravel that could be found in the adjacent desert, made a satisfactory mix for road bases which when protected by a bituminous surface gave excellent results.

The geologist came into play again and showed that the river deposited the coarser sand further up stream, or somewhere near the south end of the Delta. So, instead of hauling material all the way from the desert, it was possible to dredge this coarser sand and mix it with the silt and thus form a fair sand-clay base, which when topped with a couple of inches of gravel or stone from the desert and protected with a bituminous cover, made an excellent road.

In addition to this, several studies were made of limestone quarries. It was quite evident that the highway engineers were utilizing specifications that were drawn in some other country for if a limestone had more than 35% Los Angeles Rattler loss, it was difficult to get the engineers to use it. It was found through experiment that an excellent base could be made of limestone with as high as 60% - 65% Rattler loss and in this dry climate and when topped with a better wearing surface made an excellent base. Many of the old quarries in use during the time when the pyramids were constructed had thousands of tons of what they call refuse rock. This material made an excellent base and reduced the cost of road construction. All of this information was furnished by the geologist.

While geology was not used in the first step of road location in Egypt, it was used throughout in the finding and obtaining of materials for construction. The next step which will include roads to the various oases will probably be located through photogrammetry and based on geologic findings. There is no question but what aerial pictures will show the geologic formations so that proper and economic locations of the highways can be made.

The durability of the roads we are building will depend to a large extent upon the bases, subbases, and subgrades which support the surfaces. Again, the accuracy and extent of the information we have on the geological materials which comprise these supports will be an important factor in their design for maximum service life of these roads.

In the United States, some 375,000 structures will be constructed, or improved, under the present interstate highway program. The extent and accuracy of our knowledge of the geological materials which will serve as foundations and aggregates for these structures will be an important factor in their future service behavior. The best and most economical locations for our highways can be selected if adequate geological data is available. Future drainage problems can be minimized if geologists provide geological guides for the Highway Design Engineer.

These are a few of the reasons why the role of the geologist in roadbuilding has increased in importance. There are other reasons, too, which are very significant because of some of the characteristics of our present U. S. roadbuilding program. Congress declared, in the Federal Aid Highway Act of 1956, that it was in the national interest to accelerate the rate of highway construction. As a consequence, highway departments went all out in an effort to streamline procedures to increase the flow of projects into the construction stage. In less than three years, 42 State Highway Departments, many consulting engineering firms and the Bureau of Public Roads have integrated the electronic computer into their operations to greatly speed up processing and presentation of engineering data. Here at Georgia Tech, the Rich's Electronic Computer Center contributed to this effort by furnishing the State Highway Department excellent computer service.

Highway departments, including Georgia, have rapidly expanded their use of aerial surveys and modern photogrammetric techniques to expedite the collection and processing of engineering data. And, in keeping with their role in roadbuilding, geologists are also extending their use of aerial surveys to expedite the collection and presentation of geological data.

The detailed stereoscopic study of aerial photographs, for the purpose of evaluating the soil pattern elements, can vastly simplify the mapping of soils in the field as well as the office. Since soil patterns are of a recurring nature, the interpretation of their basic air-photo patterns, along with the identifying elements, is an economic method for efficiently covering large areas in a relatively short period of time. Under the same climatic conditions and relative topographic position, any two materials derived from the same parent material or deposited in a similar manner will have similar profiles, properties, and air-photo patterns.

In air-photo analysis, the identifications of each of the natural elements of the soil pattern determines certain physical properties of a deposit that should all agree, providing the geologic events were such that the evaluation can be correct. The land form or the general relief, topographic position relative to the surrounding area, shape and distribution of physical features, even road and field patterns, all combine to determine the land form of a soil area or deposit.

Soil color, the various shades of gray, ranging from white to black, appearing in air-photos reflect the photographic soil colors and are known as color tones. In general, organic and fine materials photograph as dark color tones, while the coarser, better drained materials photograph much lighter. Differences in elevation, soil moisture, and soil texture are all shown by a contrast in color tones.

The proper and careful interpretation of gullies will aid in determining soil texture and profile development, since uniform, deep soils have uniform gully cross sections and gradients, while stratification or variations in the soils develop cross sectional and gradient changes in the gully, Sands and gravels have short V-shaped gullies, while the cohesive soils (silts and clays) contain broad, softly rounded V-shaped gullies with long, uniform gradients extending well into uplands. The semi-cohesive silts and sand clays, with their short U-shaped gullies, often have severe headward erosion.

One of the oldest problems known to the civil engineer is the location of good granular material. The engineer is expected to know the location and extent of deposits of sand and gravel for successful construction and performances. The usual method of field reconnaissance, although time consuming and expensive, often proves inaccurate and unreliable. This type of surveying is often hampered by lack of roads and cooperation on the part of the owner, dense vegetation, and other visual handicaps that prevent great sight distances and recognition of land forms and topographic position.

The use of air-photos enables the observer to study from above large areas, determining the aerial extent and sometimes the quality of granular deposits. For example, I know of one instance where the completion of a 25-mile highway project had been delayed several years because ground searches had failed to locate gravel or other suitable material within an economical haul distance. After a study of aerial photographs was made, several locations were selected for new field explorations. One proved to be an alluvial fan containing 50,000 cubic yards of suitable base course material. The fan was almost bisected by the centerline of the road. Other selected locations proved to have ample supplies of suitable material to complete the project.

Geologists are also making wider use of the soil resistivity equipment and technique developed by the Bureau of Public Roads to expedite the collection of geological data. This method utilizes the difference in electrical resistivity of different geological materials forming the earth's surface. The instruments are low in cost and portable. The method is simple and fast. Many applications have been developed which have resulted in substantial savings in time and money over conventional core drilling operations.

More important are demonstrated savings in the cost and speed of road building. On the Washington-Baltimore Parkway, for example, the Bureau of Public Roads made soil-resistivity surveys before construction on the 12-mile southern end of the project. The purpose was to locate select borrow material within the right-of-way. These surveys were responsible for providing enough granular topping material for a 12-inch layer to be placed over the entire length of this four-lane divided highway from sources within the right-of-way limits. Several hundred thousand dollars were saved in construction costs and progress on this project was accelerated.

This method has many other useful applications. It can expedite the procurement of geological data related to landslide conditions, design of slopes, bridge foundations and other subsurface explorations. It can be used as a guide in developing core drilling programs and not only as a quick check on results, but also to augment those results with additional data below the depth of the core holes.

Geologists are also taking advantage of improved equipment for making subsurface explorations by the seismic method. New instrumentation has reduced the cost and increased the speed with which these explorations can be made. Here are a few demonstrated applications of this method of obtaining geological data which have already effected economies and expedited the rate of road building. For instance, blasting is a normal road construction operation. Safety is a primary consideration in this operation and it is closely regulated, particularly in urban areas. Usually, the amount of explosive that a contractor may use in a charge is regulated by the American Standard Table of Distances from homes, buildings, and other vulnerable installations. Sometimes, the contractor is handicapped needlessly and cannot use enough explosive to get good breakage or an otherwise efficient and at the same time safe operation.

At least one State has revised its blasting regulations to provide that where seismic data has been obtained in field tests, it may be used to determine the amount of explosive that may be used.

This application of the seismic method will enable contractors to carry out this phase of their operations more effectively, efficiently, and economically without creating any undue hazard.

This method has also been used to reduce another problem faced by contractors. They must decide on each project what equipment they should use for the earthmoving involved. Inspection of the site and "guesstimates" do not always produce the most economical and efficient results. Siesmic surveys can remove much of the guesswork from decisions as to whether material can be scraper-loaded, whether a ripper can be used to make scraper-loading feasible or whether the material must be blasted and shovel-loaded. The expense and delay of putting equipment on a project only to find that it will not do the best job will be minimized.

It can be seen that the geologist plays an ever increasing part in the modern highway program, especially from the standpoint of progress and economy.

No doubt in the early highway programs qualitative geologic data were furnished the road builder. This information came from various sources; many times it was incomplete and of questionable value.

It was not until the geologist became an integral part of highway engineering that the real benefits and savings in road work were realized. At the present time, it is absolutely essential to use the reports and studies of the geologist to realize the maximum efficiency in the progress and economy of a highway program.

HIGHWAY MATERIAL SURVEY IN WEST VIRGINIA

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An aggregate survey or inventory may be defined as an organized program to locate the maximum quantity of aggregate materials suitable for construction purposes within an area of minimum haulage distance to its point of use for the lowest possible cost of extraction and processing. To realize the optimum use of money spent on an aggregate survey its coverage should be broad. Statewide surveys meet this requirement. Since geology is the foundation for all aggregate surveys and since it has no respect for man-made boundaries, coordination of information on aggregate locations and materials is best accomplished over large areas.

Many benefits to the state can be provided by a materials survey. It can be beneficial to the design engineer in planning new projects. Knowledge of material available in the vicinity of a proposed project can influence actual design of that project. Project design is then tailored to fit the materials at hand. Information furnished by a materials survey can be used in location planning for roads of all types. By making information on aggregate locations available to contractors lower bid prices on contract work will be forthcoming because the burden of exploration has been removed.

As far back in history as man's first attempt at road construction the reasons for having an aggregate survey or inventory were present. These very same reasons are present today; however, economics are more important than ever before. Statewide aggregate surveys have become a necessity. Several states realized this fact some time ago. Other states have not yet realized the benefits that such a survey can provide. The Federal-Aid Highway Act of 1956 has pointed out the real need for aggregate Surveys.

This act provides for the construction of 41,000 miles of limited access highways across the United States. This network of highways is now commonly called the Interstate Highway System. Under this Act, the Federal Government agrees to pay ninety per cent of the cost of construction; the remaining ten per cent being paid by the individual states within whose boundaries the highways lie. Each state will be responsible for all phases of construction, from initial planning to final completion and afterwards maintenance of the completed highways. Constantly available for counsel and guidance is the U. S. Bureau of Public Roads, coordinating the over-all program.

In a general way, the specifications for the Interstate System in West Virginia will provide for a limited access, four-lane, divided highway. This will take the form of two twelve-foot wide travel lanes in each direction, with a ten-foot wide shoulder on the right hand side of each strip, and a three-foot shoulder on the left side. Where the gradient is such that truck speeds will be lower than 30 miles per hour, climbing lanes will be provided. Median strips will be forty-feet wide, except in difficult terrain and across bridges, where a four-foot raised strip will separate lanes. In certain cases, opposing lanes will have different elevations and may even have slightly different routes. West Virginia's share of the Interstate mileage is about 300 miles at the present time. From specifications calling for a base course thickness of fifteen inches, to say nothing of the wearing courses and shoulder dressing, it can be seen that tremendous quantities of aggregate material will be required for this undertaking. Average rock quantities for base course construction will be about 31,000 cubic yards per mile.

Because of the vast quantities of material required for the Interstates, as well as an accelerated intrastate road construction program, it was realized that for West Virginia the only logical approach to the problem of obtaining aggregate material was to conduct a statewide inventory. In order to locate and record new sources of material, to keep the locations within economic hauling distances, and to coordinate and disseminate the information obtained it would be necessary to have an organized approach to the problem. As a result of this thinking the Highway Aggregate Research Project was formed as a joint venture between the State Road Commission and West Virginia University with the support of the U. S. Bureau of Public Roads. Its purpose is to conduct an aggregate survey or inventory of West Virginia. In the interest of economy, an attempt has been made to limit the maximum haulage distance between aggregate sources to a five-mile radius for the entire state. Wherever possible this distance has been reduced to a radius of two miles in the vicinity of the proposed Interstate Highways.

Prior to the work done by personnel of the Highway Aggregate Research Project, searches for Highway aggregate materials were conducted only as the immediate situation required. You can well imagine the confusion and duplication of effort arising from spasmodic surveys at widespread locations by persons of varied backgrounds. Thus, only limited areas were surveyed and no over-all picture of aggregate resources was available.

Highway contractors are usually required to locate their own materials and to submit them for testing and acceptance by the State Road Commission. This additional cost of exploration for aggregate materials is passed on to the State in the form of higher construction costs. The Highway Aggregate Research Project was established to economize on this factor by enabling the State and its contractors to proceed by having information on aggregate sources available beforehand. Prior knowledge of locations of suitable material is an important factor in planning future highways. In West Virginia this inventory will be available for use on the Interstate System as well as primary and secondary routes throughout the State.

To accomplish its purpose, the Project is utilizing all material and information available. A knowledge of local geology and other related subjects is a prime requisite to any material survey. Therefore, studies of geology, aerial photography, soil maps, highway and topographic maps, and consulting engineer's reports are made. Field trips provide on-the-spot study and the opportunity to collect rock samples for testing, to determine suitable locations for quarry sites, and to substantiate assumptions made from literature surveys.

Having no precedent within the State upon which to base its findings it was necessary for the Project to design its own maps, data sheets and all of its methods of approach. Fortunately, there was available statewide geological coverage in the form of areal geology maps and written reports published by the West Virginia Geological Survey and also a complete set of county highway maps. This material has been used as a foundation and a starting point for the Project's work.

From this basic material the Project has prepared a surface-geology type map on a scale of one inch equals one mile. As the name implies this type of map shows surface features such as roads, towns, rivers, etc., superimposed on the areal geology. Areal geology, by definition, outlines the formations at or near the surface of the earth which would be exposed if all soil cover and vegetation were removed. For the sake of clarity some cultural details are omitted so that geologic detail would stand out. Aggregate material information is then added to complete the map.

The data sheets, like the maps, were designed by the Project for maximun use. A data sheet is filled out for each sample location. These sheets furnish such information on aggregates as location, ownership, utilization, deadhaul, probable quantities of aggregate and overburden, working conditions, available facilities, geologic information, and test data. They also record the identification number of the USDA aerial photograph and the name of the USGS topographic quadrangle covering the area. In addition, a portion of the sheet is available for a sketch to show the probable outcrop pattern in the area of the sample location. This is derived from aerial photographs by means of a stereocomparagraph.

Data sheet information is compiled from personal observation, a stereoscopic study of aerial photographs, available reference material, and physical sampling and testing. Testing is conducted at the State Road Commission Materials and Tests Branch. Personal Observations are made on field trips into the subject region and samples collected at that time. Each source is assigned a code number consisting of three parts. The numbers represent county, map sheet, and consecutive sample numbers in that order. Each potential site is located by distances from road intersections and by latitude and longitude to the nearest quarter minute. The site is located on the materials map and pinpointed on the proper aerial photo. The rock is identified and its geological age and formation name recorded. This geologic classification is based on or obtained from the West Virginia Geological Survey files and reports. The available quantities of aggregate and overburden are visually measured and estimated on actual exposure. Area coverage can be further estimated by inspecting the outcrop sketch traced from the aerial photo.

In addition to the maps and data sheets a written report explains and elaborates on the information collected. Its aim is to answer such questions as: Is aggregate available? What is it? Is it acceptable? Where is it located? How can it be recognized? How can it best be removed? How much is available? How far is it from the job site? What are the limiting features for a quarry? Are there any commercial aggregate producers in the area or abandoned quarries that would lower the cost of production?

Aerial photographs are used by the Project to facilitate locating aggregate material. Photos are studied stereoptically prior to field trips in order to locate outcrops, quarries, and note other features of an area. The outcrops are checked in the field and all sample locations are plotted on the photos. Then the stereocomparagraph is used to outline the rock strata and show areal distribution. However, this method is only acceptable in areas of relatively flat-lying strata.

The major source of aggregate material in West Virginia will be rock suitable for crushing, since the entire state lies south of the limit of Pleistocene glaciation. With the exception of a few glacial outwash deposits along the Ohio River and belts of alluvium in some stream valleys, there are no natural sand and gravel deposits of commercial consequence. Stream gravels will not be studied until later as it is felt that after all samples have been tested the Project will be in a better position to select streams that flow in territory having acceptable aggregate material. In that way only streams having a high potential of aggregate will be studied. Areas that are scarce in rock material will also be known and emphasis can be placed upon the search for gravel deposits in these areas.

In beginning its study, the Project divided the State into three parts. Basis for this division was knowledge of geological formations most likely to yield aggregate material. This was done with only a very general knowledge of the geology of the State, but it helped to determine where the greatest efforts of the Project would be required to locate suitable aggregate material. It was determined that along the eastern edge of the State acceptable aggregate material would be available in quantity. This region lies east of the Allegheny Front in the Ridge and Valley physiographic province. In the northern part of the State aggregate material would be available in scattered localities. The rest of the State, by far containing the largest area, contains aggregate, but it will require concentrated efforts to locate it. These last two areas are within the Appalachian Plateau physiographic province. As the name implies, this region is a well-dissected plateau whose rock formations are mainly flat-lying interbedded shales and sandstones and contain the coal horizons which are of great economic importance to the State.

In practically every phase of a material survey there is some application of geology. It is a well-known fact that an essential prerequisite for any material survey is a knowledge of the local geology. Rocks exposed within the State of West Virginia range in age from basal Permian down through Cambrian plus a few isolated areas of Pre-Cambrian volcanics. The bulk of the State contains Carboniferous sediments deposited in the Appalachian geosyncline whose axis is located in the western part of the State and roughly parallels the Ohio River Valley.

The age of the various rock formations exposed in West Virginia and their utilization for highway aggregate material can be broadly classified somewhat as follows. The limestones, sandstones and even shales of the periods from the Lower Devonian and below are hard and slightly metamorphosed and are found to be acceptable as aggregate material unless greatly weathered. The Middle and Upper Devonian formations consisting chiefly of sandstones, sandy shales, and chert are not generally suitable as an aggregate source except locally, where a few sandstone beds yield acceptable material. The basal Mississippian consists of sandstones, shales, and conglomerates with the sandstones being acceptable in some localities. The Mid-Mississippian age contains the Greenbrier limestone which is unquestionably and consistently satisfactory for use in highway construction. It is one of the most commercially exploited rock formations used for construction purposes within the State. Upper Mississippian formations are mainly sandstones, shales, and conglomerates and acceptable aggregate material may be found in some localities.

The Pottsville formations, representing the Lower Pennsylvanian, contains sandstone, conglomerate and coal and generally the sandstone is acceptable. It has been found that Allegheny formations in the south central part of West Virginia consisting chiefly of coarse sandstone and coal are not acceptable for aggregate. Conemaugh strata consists of many channel deposits of shale, sandstone, thin limestone and thin coal deposits and consequently are quite variable as to content and location. The Monongahela system is generally good in the northwestern part of the State and contains several limestones in quantities that are sufficient for local requirements of highway construction. Elsewhere it consists chiefly of shale and coal. The uppermost system exposed in West Virginia is the Permian consisting of sandstone, shale a few thin limestones and thin coal formations. Wherever weathering has not taken place formations of this period yield quite acceptable aggregate material.

Since sandstones and shales of the Permian, Pennsylvanian and Mississippian periods cover almost three-fourths of West Virginia it would be well to consider the adaptability of these sandstones for highway construction. Because of the variable nature of sandstone its use as a highway construction material has been limited. Consequently, other aggregates have been imported into a region even when shipping costs have exceeded the initial cost of the same aggregate at its source. However, through investigation, it has been found that even though sandstones possess widely varying properties they are still usable for highway construction. When a suitable sandstone has been found at a potential quarry site its apparent abundance must be investigated. Because of variations in thickness or "pinch-out" there is no assurance of high quality sandstone for any great distance, either horizontally or vertically. Among the Pennsylvanian sandstones in West Virginia lateral "pinching-out" and vertical changes in lithology are the rule rather than the exception. In spite of these shortcomings, the Project has thus far been able to locate acceptable material within its five-mile radius haulage goal.

The Highway Aggregate Research Project in West Virginia now has under study a field check that will rapidly predict Los Angeles Abrasion Test results. Briefly, this test is a measurement of a rock's ability to absorb a liquid, in this case alcohol. It has been determined that there is a direct relationship between the alcohol absorption capacity of a sandstone and per cent wear by the Los Angeles Abrasion Test. This modified porosity test is now performed with 85% accuracy on non-siliceous sandstones. Further studies are being conducted in expectation of increasing the accuracy of the test and to adapt it to field use.

The Project is also preparing graphs showing the Los Angeles Abrasion Test results for each sample of any one geological formations. Those samples having an L. A. of 65% and higher are colored red while those below 65% are colored green. In West Virginia an L. A. of 65% is the dividing line of acceptability for base course material. Therefore, with these graphs one can visually estimate the percentage of acceptability of any geological formation. The acceptable samples of one formation are also plotted on an outline map of the State. This will indicate geographical distribution of an acceptable formation and this information can be used for later geological and highway construction studies.

Many studies can be made from information gethered in the course of conducting a meterial survey. Both practical and academic problems exist that can be assisted in their solution by such data. Take for example, landslides; a problem that exists in West Virginia and other mountainous states. In a short time anyone associated with a material survey recognizes rock types and structures that, under certain conditions, favor landslides. Think of the potential value such information would have on planning and routing of future highways. A material survey can provide a basis for study of testing of highway materials, soil types, bearing loads, frost action, and probably many other problems. Gathering and disseminating information relative to sources of aggregate materials should become a regular function of all highway organization.

GEOLOGY IN FOUNDATION ENGINEERING

By G. A. Fletcher

Raymond Concrete Pile Company

Geology is an old and recognized science. Leonardo DaVinci, that brilliant and versatile genius of the 15th Century is considered, by some, to be its founder. Today, the mining and petroleum industries, probably more than any other groups, have teamed up with the geologist and made full use of that science in exploration and development work.

In civil engineering, the recognition of the part which geology can and should play has been too long delayed. While it is true that in civil engineering collaboration with the geologist is accepted practice for large and important projects, such as dams, tunnels and other engineering works of great magnitude, the engineering profession has been lax in calling upon the geologist for the important information he can provide in connection with run of the mill mundane projects.

The development of the engineering geologist is a much needed and welcome effort to bridge the gap between geology and engineering. No doubt the time will come when every engineering organization, specializing in foundations will include on its staff an engineering geologist.

The relatively recent growth of soil mechanics, while of inestimable value in the solution of foundation problems is not calculated to supply or interpret the geological data which the foundation engineer should analyze in connection with virtually every project which comes before him. Nevertheless, engineers who have been trained in the science of soil mechanics, are fully aware of the importarce of reviewing the geology associated with any project. In nearly every case where engineering reports on foundation design are prepared by specialists in soil mechanics, a section of that report will be devoted to the geological environment of the project.

Test borings, essential as they are, to the intelligent design of foundations, should always be reviewed and studied in the light of the geological environment in which a project is situated. Failure to follow this basic principle has resulted in many unforseen foundation problems. On the other hand, where the geological data have been carefully reviewed in advance, the opportunity to simplify foundation design or to achieve economies in design have been realized.

It is my purpose to review a limited number of both such cases.

The city of Milwaukee, Wisconsin, situated on the west shore of Lake Michigan is underlain by two general types of soils. The eastern portion of the city, closest to the lake, is located on a clayey silt bluff which in olden times sloped steeply to the west to meet the alluvial deposits formed at the confluence of three rivers. The downtown section of the city is located on the fill which was brought down from the bluff and spread over the organic silt which was formed over the alluvial deposits. As a result, the principle structures have been supported on piles and our company have been making borings and driving foundation piling in Milwaukee for over fifty years. A few years ago, an addition to the City Hall was planned and borings were made. Careful attention was given to the location of the borings since the original City Hall lying just west of the addition was on piles. The borings did, in fact, show that organic silt and fill extended somewhat into the western area of the new addition and on the balance of the site. Although no organic silt was encountered, the general density of the clayey silt deposits in the first 40 feet of depths indicated the need of a pile foundation.

In endeavoring to drive the cast-in-place concrete piles, it was discovered at the very beginning that the soil surrounding the piles heaved an amount exceeding the volume of the driven pile, but even more important, the resistance to which the piles were driven, was largely dissipated and redriving the piles only made matters worse. It became apparent that an entirely new approach to the installation of the piles on this site must be developed. The solution to the problem was to pre-excavate by wet rotary methods through the upper 40 feet so that the upper layer was not disturbed by pile driving and to drive longer piles to obtain their resistance in the more granular material below the 40 foot layer of clayey silt.

A review of the geology of the site provided the explanation for the difficulties. First, the bluff had been cut down by close to 40 feet and the soil removed had been used for fill in the lower parts of the city. Second, the bluff had been subjected to glacial ice loading of from 6 to 9 tons per square foot. As a result, the clayey silt formation had been highly surcharged and over consolidated so that the water contents were as low as 11 to 15% by dry weight. When such a dense soil is disturbed, it cannot be further consolidated, but must expand, thus changing from a dense to a materially softer state.

Had the geological environment of this site been studied, prior to the design of the foundations, I have no doubt but that the problems and potential difficulties would have been foreseen and time saved and costs reduced.

As another example, borings were made upon a site for an industrial project near Childersburgh, Alabama. The borings revealed alluvial deposits over the rock which were loose and unconsolidated for the most part. Some stiff clays and dense granular deposits were also found and the borings also disclosed that the surface of bedrock was most irregular. In endeavoring to drive cast-in-place concrete piles under these conditions, many of the piles were driven through the unconsolidated over-burden and great difficulty was experienced in bringing them up on the rock surface because it was so steeply sloped that the points of the piles tended to drive down the slope, thus bending the piles and the mandrels. It was as though bedrock had been covered with stalagmites and that we were endeavoring to drive piles on to the edges of pinnacles. Under conditions encountered on this site, there are no simple or cheap solutions. Open end pipe piles driven to rock and cleaned out before concreting constitute a practical solution, but they are expensive and where rock may contain cavities or pockets filled with soft materials, which may be located below the points of the piles, it is not sound engineering to design such piles for high loads. The same objection applies to installing caissons to rock. Every caisson would become an experimental project since it would be necessary to prove by diamond drilling that a sufficient thickness of sound hard rock existed below each caisson to carry the column loads safely.

However, if the geology of the site had been investigated before the foundations were designed, it would have been discovered that this site was located on a zone of badly fractured rock which extends from Alabama, northeast into Virginia. The rock surface is broken into a saw-tooth type surface with V-shaped troughs and tall pinnacles. The bedding of the rock results in slopes of from 45° up to 90°. Frequently, the softest material will be found in the troughs or cavities. The rock formation consists primarily of limestone and dolomite.

When it is know in advance that a rock formation of this nature exists, the difficulties of driving piles can be foreseen and the work can be planned, taking into account that some piles will be so bent as to be useless resulting in driving additional piles and requiring redesign of the pile caps. While these expenses will result in relatively high costs, the overall expense will be far less than going ahead blindly and spending time and money in experimentation. For that is doing the job the hard way.

Where the foundation engineer does call on geology for its valuable assistance, economies and simplification, as well as the avoidance of construction problems have resulted. For example:

An engineering organization in New York responsible for the design of foundations for a considerable number of widely scattered projects, discovered from a series of soil investigation that a large portion of New York City, more specifically, the eastern part of Manhattan, had a surprisingly uniform overburden. Reference to the geology of the area disclosed that the eastern half of Manhattan Island, for a great part of its length, from the Battery to the Harlem River, had been the bottom of an ancient glacial lake. The soils of the lake were composed of varved silts and clays which in more recent times, had been covered in most areas by relatively loose sand; in other areas, by peat and organic silt. Man-made fill has brought the ground surface to its present general contours. From extensive laboratory tests on undisturbed samples, it was determined that the varved clays and silts of the glacial lake deposits had been preconsolidated to loads of from 6 to 10 tons per square foot in addition to the existing over-burden pressures. From this it was concluded that the lake bottom must have been ice loaded at some time in the past. The significance of this research was to open up the possibility of much more economical foundations for buildings up to 12 to 15 stories in height. Previous practice had been to drive long piles through the varved silts and sands to the hardpan overlying rock. The alternate foundations made practical by the foregoing study include short friction piles driven into loose sands of relatively recent depositions or to employ spread footings or mats.

Each proposed structure, however, requires a soil investigation since tidal estuaries, deep ponds, and old streams have cut into the varved clays and silts and deep beds of organic silt and peat exist from place to place. At such locations, foundations on piles to hardpan or rock, are still required.

For my last illustration, I would like to describe a project where we turned to geology for counsel and advice in planning the work only to discover that the deductions of the geologists were wide of the mark.

In 1954, it became necessary to carry out a soil investigation out in the Atlantic Ocean more than 100 miles east of Cape Cod. The purpose of the investigation was to determine the soil conditions on George's Bank, one of the fishing banks off the New England Coast. At that time, the construction of the George's Bank Air Station was being studied by the engineers for feasibility and foundation design. This was the first Texas Tower planned by the Air Force.

The drilling project was a joint venture composed of the Raymond Concrete Pile Company and the DeLong Corporation and we sought the advice of geologists associated with the Woods Hole Oceanographic Institution who know more about the area than anyone else. Based on the fact that the location and the contours of George's Bank as well as other fishing banks have been shown on mariner's charts for many, many years with no appreciable changes in spite of the hurricanes and resulting high waves in the area, the geologists deduced that the soil must be extremely dense and probably a glacial till, composed of clay, gravel and boulders. Only so dense a formation, they felt, could withstand the action of the sea. This information was vital to us as we had to complete the project successfully prior to August 10th of that year which is the beginning of the hurricane season. We had to plan our drilling equipment and our drilling techniques to meet the worst conditions which might be anticipated.

Fortunately, for all concerned, we encountered dense sand to depths of 70 feet, then a layer of medium stiff clay about 12 feet thick, and then underneath the clay the dense sand occurred again. Such a formation, of course, can be drilled successfully and rapidly. Also, it permitted an economical design of the foundation of the permanent caissons for the Texas Tower since the caissons could be dredged down the necessary distance in the sand and then filled with concrete. It may be of interest to know that we finished the drilling program on August 8th, but that the following year, when we were erecting the tower on George's Bank, Hurricane Connie came up the coast exactly on August 10, 1955.

It has been my purpose in this brief paper to emphasize the responsibility which rests with the foundation engineer to consult with geologists and study the geology of the area in which a project is located. Every one stands to gain by making such consultation a matter of accepted practice, including the geologist, the consulting engineer, the construction industry, and the owner.

APPLIED

GEOMORPHOLOGY

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This word when broken down into its roots means "an earth form discourse". The science of geomorphology as it is conventionally used is a study of the configuration of the earth. Geomorphology as a classical subject is far too general for the specific requirements of engineering; hence, the term "applied geomorphology". The broad and general nature of geomorphology is best illustrated by the categories of geomorphic units or landforms that it recognizes.

Von Engeln[(1)] lists first order geomorphic units as consisting of continents and oceans; his second order includes plains and plateaus, the third and final order of landforms are classified as valleys, ridges, and cliffs.

Valleys, ridges and cliffs are far too general for a highway engineer, hydrologist, or a contractor engaged in the construction of a public structure. Since a long section of highway can be constructed within one valley, one can recognize that a considerable amount of subdivision or refinement must be added before geomorphology can be considered useful to the field of engineering. It is necessary to work with smaller units or landforms that have some practical relationship to the design and construction of an engineering project. Such a unit must have a definite bearing on the design or ultimate performance of a road. It should represent a set of conditions that are closely associated with a specific landform and those conditions should differ materially from conditions that are associated with some dissimilar landform. The conditions are basically related to topography, soil texture, the depth of soil overlying bedrock,

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(1) O. D. von Engeln, Geomorphology

moisture conditions and ground-water movement, the type of rock that may exist beneath the surface and any special problems particularly associated with landforms, such as landslides or solution cavities.

To accomplish this refinement of geomorphology, it is possible to add a fourth category of landforms and these are recognized by many geologists as well-defined units. A listing of these would number approximately 36 and will include such familiar forms as sand dunes, beachlines, clay shales, alluvial fans, eskers, and others like these that are representative of landforms found in various parts of this and other countries.

In applying geomorphology to highway engineering we find that each of these units has a special significance in design and construction because it embraces a more or less unique set of conditions. Each landform is a package and each package includes well-defined limits of topographic characteristics, a typical range of soil texture and often, some degree of sorting. The ground water will probably fluctuate between predictable limits throughout the year and the soil moisture conditions will reach average values during various seasons.

We can rely with a high degree of assurance on the fact that these conditions recur in similar landforms regardless of their geographic location. Consequently, if certain cut slope designs are satisfactory in flat lying, interbedded shale and sandstone formations in one place, they will be found satisfactory elsewhere provided that we make sure that we are dealing with a similar landform. It is for this fundamental reason that one looks for sand or gravel in beachlines; that we expect seepage in highway cuts in a glacial moraine; that we design vertical cuts in loessial soil areas; and that outwash plains provide stable foundations as well as serve as a source of granular material. So we propose that landforms of this order of magnitude be considered as fourth order landforms to be added to the classification list.(2)

(2) See attached list.

These characterisitc conditions of landforms have been understood and appreciated for a number of years, but they do not yet embrace the full potential of applied geomorphology because experience has shown that there still remains predictable details that are of importance to a civil engineer and to a contractor. These details represent minor variations within the fourth order landforms just reviewed.

These minor variations are the fifth order forms of the configuration of the earth's surface. They are really micro-forms or micro-features and they are so small that they do not appear or are not accurately portrayed on a topographic map. This is one reason that the earlier geomorphologists failed to see them. Only since the advent of aerial photography has there been an opportunity to observe them and relate them to their origin and to evaluate their significance.

Rather than become involved in an involved subject, it may be best to explain the genesis of micro-features. Usually, they represent some flaw or weakness inherent in the origin of a deposit or a bedrock that is more susceptible to attack by weathering.

In its early stage of development it is a scar or a slight depression that was originally considered as micro-relief, but as weathering proceeds, this micro-feature develops more soil than nearby ground, it has a higher clay content, it holds more than average moisture, plants usually grow more lushly and as a result, this small area has a higher organic content in the soil. All of this may photograph as a small dark area of distinct outline in humid areas or perhaps as a white spot in arid areas.

These fifth order forms fall into this geological Bertillon system like nature's finger prints that help to identify various soils and rocks. Is this carrying applied geomorphology too far? Is it a matter of learning more and more about less and less? Obviously not, because we can now reach a level at which applied geomorphology can help an engineer.

Today more than ever, construction materials are critical materials. Selected borrow having various characteristics, as well as granular materials of the gravel and sand classification, are more urgently needed now than ever before. The demand is for larger quantities of materials per mile or road, and yet these materials are more difficult to obtain because in the past we have exhausted the most obvious deposits and now must look farther away or deeper in the ground for the necessary quality and quantity demanded. There is a need for special combinations of materials which in larger quantities can be substituted for smaller amounts of preferred materials. So in each region of the United States major emphasis is being placed on the location of new sources of these materials. We believe that applied geomorphology is the primary step in these surveys.

The materials situation in some areas of California is so critical that we were retained by the California Department of Public Works to search an area of approximately 5,000 square miles in the San Joaquin Valley. Not only was this for the purpose of locating materials, but it indirectly located the 200 mile West Side Freeway because this work preceded the actual location of the highway. The final location was based upon the shifting of the line that was established by the quantities of materials discovered in various places and ton-miles of haul that would be required to provide the fill.

In the same category of work we may come closer to home by citing work in South Carolina. A large part of South Carolina is included in the coastal plains which falls into von Engelns' second order of landforms of the classical geomorphologist. Within this broad classification, we find third and fourth order landforms as well. The fourth order landforms include upper and lower coastal plains, terraces, beachlines, tidal flats and a variety of swamps, etc.

Although these fourth order landforms are useful in planning and other general engineering considerations, we still need to proceed to the fifth order before we can discover buried deposits of sand-clay or the more granular materials. We were engaged by the Department of Highways to locate sand-clay deposits that would be available for use on a by-pass around Sumpter. Combining a very careful analysis of the aerial photographs provided by the Department and some preliminary ground inspection to clarify the meaning of some of the micro-forms noted on the photographs, we were able to lay out a drilling program that was conducted by the Highway Department forces and the samples tested in the Highway Department Laboratories. The results of this exploration provided satisfactory sand-clay sources meeting a very rigid gradation and cohesion specification set forth by the Department. The discovery of these buried deposits was based upon common-sense geomorphology assisted by the indications of these materials that can be found in the aerial photographs.

Using the same techniques that can be partially illustrated in the next slide, we are now conducting a survey of approximately 9,000 square miles in one of the coastal plain States for the purpose of locating deposits of acceptable granular materials.

Applying geomorphology to a somewhat different problem our staff has assisted in the location of various highways throughout the country. To briefly illustrate the various applications, consider first the proposed northern extension of the Massachusetts Turnpike. This was a survey of 70 miles of proposed right-of-way to achieve the most economical location of the road as it passed through very complex country. In making these studies, the refinements of geomorphology were applied in order that the construction would encounter a minimum of bedrock. This was done on the basis of determining the depth of glacial drift covering rock and the presence of boulders. It also concerned the determination of depths of swamps so that a swamp area could be utilized provided that the bottom material was gravel and that the organic material was not excessively deep. Drainage was also a significant consideration and in many types of terrain, there is no better way of locating the lines of water movement, both surface and subsurface, than by applying practical geomorphology through the use of aerial photographs.

In the location of a new road in Virginia, paralleling U.S. No. 1 south of Washington, geomorphology was applied in the same manner to the complex conditions of the area. In the Massachusetts Turnpike work it was a matter of working in an area where glacial drift was covering igneous and metamorphic bedrock. In Virginia the road traverses an area where the coastal plain materials ranging from clay to gravel cover the underlying bedrock in varying degrees. In this area stratification of the coastal plain materials creates a situation in which seepage occurs almost exclusively on the east side of the hills. Found in conjunction with unstable materials, this seepage induces landslides in highway cuts that over-steepen the east facing slopes of these hills. In this rough terrain that has a heavy brush and forest cover, applied geomorphology was used to guide the inspection, by photo analysis, of every hill within range of the project. By so doing, it was possible to recommend one specific alignment out of many possibilities so that the road would avoid unexpected rock excavation, largely eliminate seepage and by-pass local areas that showed a tendency toward sliding and slumping. Areas of gravel were mapped directly and on individual hillsides the outcrops of various strata were identified and described.

If we direct attention to another landform area we find that in Pennsylvania, applied geomorphology has made an interesting contribution to the location and design of a portion of the interstate system. This project is in an area of flat lying sedimentary rock consisting principally of six to eight different shales grading from clay to sand and some sandstone with coal measures included. The objectives here were not only the conventional ones that include seepage zones, slope design data, landslide appraisal and construction materials, but it also included the item of secrecy. The requirement placed upon this work was that no ground work was to be performed in the interest of avoiding land speculation. Although this procedure is not practical in every area as it was in Pennsylvania, the results of this work have been eminently satisfactory and it is of special interest to see the preliminary design carried through based upon this "remote" means of photogrammetry and the determination of ground conditions.

In closing, it must be emphasized that work of this nature is performed by the use of applied geomorphology made practical through the medium of experienced analysis of aerial photographs and carried through to the ultimate drilling and testing by carefully planned preliminary field inspection of the fifth order of geomorphic units or micro-features that have been described.

LANDFORMS

Sedimentary Rocks

Sandstone

Shale

Limestone

Limestone (tropical-karst)

Coral

Interbedded Sedimentary Rocks
(flat-lying)

Interbedded Sedimentary Rocks

(tilted)

Igneous Rocks

Intrusive Rocks

(Granite and Related Rocks)

Extrusive Rocks

Basaltic Lava

Fragmental and Related Interbedded Flows

Miscellaneous Features

Metamorphic Rocks

Gneiss

Schist

Slate

Serpentine

Waterlaid Materials

Flood Plains

Terraces

Filled Valley

Continental Alluvium

Alluvial Fans

Deltas

Lake Beds

Beach Ridges

Coastal Plains

Tidal Flats

Marshes and Swamps

Glacial Materials

STRATIFIED

Eskers

Kames

Outwash Plains

Terraces

Lake Beds

UNSTRATIFIED

Till Plains

Moraines

Drumlins

COMPLEX

Complex Glacial Deposits

Windlaid Materials

Sand Dunes

Loess