PROCEEDINGS OF THE 17th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Iowa State University

April 21-23, 1966

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TABLE OF CONTENTS

PREFACE

The 17th Annual Highway Geology Symposium held at Iowa State University was organized to include a Concrete Durability Field Trip on the first day, a program consisting of papers presented on various aspects of highway geology on the second day, and a panel discussion on the chemical and physical reactions of carbonate aggregates in concrete on the morning of the third day.

Part I consists of ten papers which were presented during the technical sessions of the Symposium. All of these papers are included in these proceedings except Chemical Stabilization of an Active Landslide, by W. W. Williams and R. L. Handy. This paper was published by the American Society of Civil Engineers in volume 37 of "Civil Engineering" in the August 1967 issue on pages 62-65.

Part II is the panel discussion which covered various aspects of carbonate aggregate behavior presented by persons with research experience in this area. The panel discussion represents the first of its kind held for a geologically trained audience. For this reason the editors have transcribed the panel discussion for publication as Part II of the Proceedings. We feel it is a valuable addition to the literature and thank all the panelists for their co-operation in getting the discussion published. Dr. Chalmer J. Roy, Dean of Sciences and Humanities at Iowa State University is especially thanked for the support which made publication of the Proceedings possible.

At the Friday evening banquet Mr. Milo D. Nordyke of the Lawrence Radiation Laboratory in Livermore, California, spoke on the application of nuclear explosives to excavation and aggregate production.

The Co-chairmen wish to express their gratitude to the people and organizations who assisted us in so many ways. Joan Stock, Diana Kneller, Lola Nelson, L. M. Clauson, George C. Christensen, Chalmer J. Roy, H. G. Hershey, A. C. Dodson, T. E. McElherne, James Elwell, David Simon, The W. H. Kessel Company, Geophysical Specialties Company, Soiltest Incorporated, Acker Drill Company, all the authors and panel members for their very fine contributions to the program. Mrs. Lola Nelson, James Elwell and David Simon transcribed the panel discussion. Marvin Gould, and the Staff of the Engineering Extension and the Memorial Union at Iowa State University helped with arrangements for the symposium.

Supporting agencies contributing to the success of the Symposium were the Iowa Limestone Producers Association, the Geological Society of Iowa, Iowa Geological Survey and the Engineering Extension Service at Iowa State University.

The Co-chairmen also wish to express their thanks to those who attended the meeting and have been so very patient in waiting for the publication of these proceedings.

T. L. Welp, Chief Materials Geologist
Iowa State Highway Commission

John Lemish, Professor of Geology
Iowa State University

PART I. TECHNICAL SESSION

ENGINEERING GEOLOGY IN KANSAS HIGHWAY CONSTRUCTION

By

Virgil A. Burgat, Chief Geologist
State Highway Commission of Kansas

First of all I would like to give you a little history of our Geology Section and acquaint you with the staff.

Kansas employed the first Highway Geologist in 1936. Prior to this time bridge foundation and rock information was obtained by driving sounding rods and drilling with hand augers. No correlation of strata was attempted between individual soundings. It soon became apparent, however, that correlation of strata needed to be made between test holes to expand the information of the tests and that this correlation could be done properly only by the Geologist. When highway design was changed from that of terrain grades to the long radius curves and ruling grades, it resulted in marked deepening of road cuts and increased height of the fill sections.

These improved design and construction practices increased the need for Engineering Geologists as the Engineer was becoming more dependent upon the Geologist for many design and construction details. We presently have a sectional staff of 25 geologists. Five geologists are serving in other departments of the Highway, namely, two geologists in Research, 2 geologists in the Photo-Interpretation Section and one geologist in the Materials Department. Nearly all studied Geology in our own State Universities. The Section is administered under the Design Department yet works rather directly with each of the approximately seven other Headquarters Departments and also with the Division Engineers. This organizational structure allows for the efficient use of geologic service throughout the whole of the Highway Department. Within the intricate organizational structures the section is divided into three regions. The regions are supervised by a regional geologist. Close coordination of the three regions is carried out through the office of the Chief Geologist to make the most effective use of both personnel and equipment. Each region is staffed with two surface parties composed of two or more geologists and engineering aides and a core drill party composed of a geologist, a core driller and two or more engineering aides. Surface parties work out the geology for grading plans and administer to the construction problems. The core drill party is used primarily on bridge foundation studies. The duties performed by either party are interchangeable as the need arises.

As you can see this is a rather sizable organization and like any other governmental organization we are required to take inventory occasionally at the demands of those who hold the purse strings to see what is really being bought with the money spent for geologic services. So far we have been lucky enough to show on the ledger sheet that our geologic service is giving the State of Kansas a good bargain. And for more reasons than one we hope the balance remains favorable on our side.

When we consider the present-day Highway Construction program we are aware of the tremendous resources and the combined knowledge and skill of many disciplines to build good highways rapidly. The earthwork and bridge foundation aspects of highway construction are the responsibilities of the Engineering Geologist. Each year's program requires many millions of yards of classified excavation and the construction of several hundred bridges. The traveling public demands highways of high standards and quality and to build highways having high quality in a limited number of working days requires much advanced planning before earthwork is begun and bridges are erected. The plans of how to properly construct the highway are placed on a few blueprints.

I wish to acquaint you with the materials which which we work in Geology.

Our highways are constructed making use of various geologic materials which are under the following categories: Loesses, silts, clays and sands and conglomerates of the Pleistocene and tertiary age; the chalk beds; the alternating shales, limestone and sandstone members of the Cretaceous, Permian, Pennsylvanian and Mississippian Periods.

The Engineering Geologist has the responsibility of identifying the various geologic materials which will support the overall road structure, determine what geologic problems exist and prescribe a solution for these problems.

HIGHWAY ALIGNMENT

The assistance which the geologist gives begins with the highway alignment. The alignment is usually located along a direct route from the point of origin to point of destination. Streams, cross road highways and railroads are the major vertical control points. Unnecessary right of way costs, the movement of historical landmarks, churches, and cemeteries are of course avoided. In general our highway alignment is chosen regardless of the geology except where geological conditions become critical. In very rough terrain the Location Engineer will discuss the geologic conditions and on recommendation from the geologist he will try to minimize, if possible, heavy rock excavation, avoid large areas of landslides and unstable shale strata springs, seeps, and swampy ground and unique water supplies to prevent costly construction and legal involvement. The major alignment problems based on geology are areas where alignments cross unstable shale formations. Several miles of Interstate highway location survey had to be realigned in two locations after it was discovered during the geology survey that the highway, if following the original alignment, would have to be built on several thousand feet of old Graneros Shale slides. The original line would have required costly corrective measures including such measures as removal and wasting of several hundred thousand cubic yards of unstable slide material, construction of benches into shale below the slide plane, backfilling to grade with a stable material and the construction of underdrains to prevent resaturation of the area. Occasionally there are seepy and wet land areas which must be avoided by the alignment if at all possible. Photo-interpretation and reconnaissance field investigation by the geologist often proves to be helpful in the advanced planning of the Highway route. Large savings have resulted by avoiding adverse geological conditions through the proper choice of highway alignment.

GEOLOGY FIELD DATA

With the alignment established, the problems of design which are of geologic nature are resolved in the final plans through the cooperative efforts of the Geologist and the Design Engineer. The field data are obtained by the Geology Field Party. It includes measured stratigraphic sections, description of strata elevation of strata, test hole data and other information. A description of the kinds of material and the conditions in which it will be encountered are presented in a geology report. This report also gives basic information that will enable the engineer to evaluate such factors as: classification of excavation, steepness of backslopes, groundwater problems, unstable subgrades, balance factor of the rock and common excavation and other special specific geologic problems. The report will indicate where the contractor may observe similar geologic units to those on the project in open cut sections, thus giving information to him whereby he can make a more intelligent bid on the excavation materials.

Geologic data must be graphically portrayed to be usable. Detailed Geologic profile and cross sections are developed parallel to and across the roadway throughout the alignment. This portrayal of data on the plans and in the geology report requires the certain detailing procedures and symboling. It requires the use of basic geologic data and nomenclature made available through the publications of the State Geological Survey. These data and nomenclature are incorporated in both the graphic plans presentation and in the written report. Names of geologic strata and their physical characteristics must become common knowledge of both the geologist and the engineer. In some cases this information even becomes common knowledge to the contractor familiar with a certain area. The methods of portraying the geology in the report and on the plans and the engineering usage made of it develops with experience and improved understanding.

CLASSIFICATION OF EXCAVATION

All earthwork is classified as either rock or common excavation on the basis of geologic characteristics. Present specifications are based on geologic terms and definitions of the materials of excavation. Prior specifications which were based upon the methods and type of equipment used to make the excavation rather than the nature of the material led only to many arguments and lawsuits. Specifications now based on geologic characteristics give uniformity of classification regardless of the Contractor's equipment or methods. A successful system of classification can only result by calling the same material the same today, tomorrow and from project to project whenever it is encountered.

Rock excavation includes limestone, sandstone, unweathered sandy and calcareous shale and other materials that have the characteristics of rock, whereas common excavation includes soils, soft weathered shales and other materials which do not geologically have rock characteristics. By classifying excavation as rock or common on the basis of Geology, Kansas has long enjoyed low bid prices by Contractors. The cost of rock excavation per cubic yard is presently only 4 to 5 times, that of common excavation. Common excavation is bid at 15 to 25 cents per cubic yard on normal grading projects. Very few disputes over classification have arisen. The Contractor has been able to plan his work and make better use of his equipment because the geology is plotted accurately on the cross sections showing the boundary lines between the rock and common excavation. This boundary line in excavation is determined by test hole and other pertinent field data to determine classification.

It has become a common practice for Contractors to bid excavation unclassified on classified material. This practice would indicate his trust in the accuracy of the classification. At the same time this creates a saving to the State by the decreased cost of engineering needed to determine the classification for final payment. The Contractor may stand to lose heavily on this unclassified bid if the excavation is not accurately classified.

GRADE LINE

A tentative grade line is plotted showing its relationship to the strata portrayed on the cross section and profile in the cut and fill sections. At this stage the grade line may be adjusted upward or downward to avoid cutting the roadway into undesirable strata requiring a more costly design. Minor realignments by office relocation may be required to avoid potential slide or otherwise unstable areas, unique water supplies and other problems.

After the road designer determines the best desirable vertical and horizontal alignment the work of design proceeds. Templates of the roadway are placed over corresponding geologic sections to show in cross section, the graphic relationship between the roadway template and the strata. The sections will show the depth of the cut and the quantities of soil, limestone, sandstone, or other to rock that will need to be moved. Also they will indicate what is common and what is rock excavation.

BALANCE FACTOR

The adjustment factor for the earthwork balance is the volume expressed in percentage that results when material is broken up in the process of excavation and is moved from a cut to a fill section. The volume of the excavated material may be either greater or less than the original volume. Adjustment factors to compensate for this change must be accurately determined if a satisfactory earthwork balance is to be obtained. Limestone strata taken from the cut section and placed in the embankment will increase in volume from 35 to 50 percent depending upon the physical characteristics of the strata. The volume changes of shales and sandstones vary with their cementation and physical structure.

A clayey soil mantle has a higher shrinkage volume than does a granular or gravelly soil. The proper earthwork balance adjustment factor for the geologic units must be supplied to the road designer, otherwise the volume of fill material will be either too great or too little. An excess is wasteful and a shortage will require additional borrow pits. This can cause a critical problem in construction either in requiring a waste area for excess material or purchasing a borrow pit to obtain the extra material needed.

Problems of overruns are created when the contractor bids unclassified on classified materials and unreliable adjustment factors have been supplied to the road designer. This type of error can be very costly to the State.

To show an example consider a circumstance which would be very favorable to the contractor. A grading project has a million cubic yards of classified excavation. The Contractors bid the job unclassified, his bid being based on the quantities of classified material shown on the plans. The unclassified bid price is basically the average of the rock and common bid price. In this example the total million yards of excavation is divided equally. Thus, $\frac{1}{2}$ million yards of rock at 75 cents and $\frac{1}{2}$ million yards of common at 25 cents would average 50 cents per cubic yard for the unclassified bid price. If the earthwork balance adjustment factors used were 10% too high on the swell of the rock and 10% too low on the shrinkage of the common excavation it would require an additional 100,000 cubic yards of excavation to make up the difference in quantity to complete the grade as shown on the plans. Additional borrow material must be acquired. The contractor receives the unclassified bid price of 50 cents per cubic yard for the 100,000 yards of material needed from the borrow to bring the roadway up to finished grade, thus creating a 50 thousand dollar overrun on the contract. Had the contractor made the classified bids of 25 cents common and 75 cents rock, the additional yardage of common excavation needed, due to the error, would have cost only half as much or a reduction of $25,000.00 on the overrun. Thus, the error in adjustment factor became very favorable to the contractor and vice versa a considerable loss to the State.

On the other hand, if the earthwork balance adjustment factors used were 10% too low on the swell of the rock and 10% too high on the shrinkage of the common the project cost on excavation would not overrun in dollars, but at the same time 100,000 cubic yards of material at 50 cents per yard will be needlessly wasted and again this waste would be no loss to the contractor but likewise another heavy unnecessary loss to the state.

OVERBREAKAGE

Overbreakage refers to the removal of more material than required from the strata which is excavated to complete the highway grade. Rock characteristics such as partings, joints, bedding and weathering have great influence on overbreakage. Reasonably accurate overbreakage quantities must be anticipated and be included in the balance factors to make the earthwork balance complete. The relationship of the plane of excavation for the road grade to that of the strata in both subgrade and backslope, as well as the breakage behavior of the strata, are the controlling factors of overbreakage.

In the present Kansas Highway specifications, the item of overbreakage in the roadbed is allowed as a pay item only if the characteristics of the rock make it unavoidable. There is presently an allowable limit of 5 percent overbreakage in cut slopes. This allowable will be eliminated from the specification where presplitting of the slopes is required.

The expected overbreakage should be clearly shown on the plans to guide the Construction Engineer and the Contractor in planning the earthwork. An unforeseen overbreakage during construction could result in a waste of material whereas had it been expected, the material could have been used in the fill embankment rather than other material taken from the borrow pit.

(Show unit ledge top of which is just above grade near a bridge location on Gage Boulevard)

SUBGRADING

Subgrading refers to the removal of certain materials in the subgrade below the base and surface course of the roadway because it is undesirable or unstable for supporting the road structure. Some shales perform very poorly as road foundation material. Their poor performance is due to volume changes resulting from variations in moisture content. Material of this type must be removed from directly beneath the road surface and replaced with more stable material. At one time it was standard practice to subgrade 12 inches in all rock and shale cuts regardless of the quality of material and backfill with soil from borrow pits. This practice was partially established because there were certain shale strata, which developed into bad actors and also because there was little exercise of control on allowable overbreakage. This practice often resulted in the use of material from available borrow pits which was more undesirable than was the subgraded rock or shale which it had replaced. Often times this practice created a wedge of unstable material between the stable rock subgrade and the surface course. These problems brought about the practice being used at the present time. We presently require no subgrading for substantial limestone strata. Any overbreakage within these limestone strata may be backfilled to within 8 inches of the subgrade with similar material from which the overbreakage occurred and then brought up to subgrade elevation with 8 inches of crushed stone. The crushed stone in this case is a subsidiary item in the contract and thus is a penalty to the Contractor for allowing such overbreakage to occur. Shale strata which have good past performance and exhibit low swell characteristics are not usually subgraded. Shales or certain clay horizons, which by observation and their past performance records show questionable subgrade stability, are tested in the laboratory. Those shales which have unacceptable high shrink and swell characteristics are subgraded 8 inches and backfilled with crushed stone. In areas where crushed stone is at a premium and stable soil are available the subgrading depth is 12 inches and the backfill is approved soil. The bid item for the 8 inches of crushed stone is by the square yard in place. This crushed stone backfill is usually placed across the full width of the roadway. We are presently using the criteria that the questionable shales, which have a volume change of more than 2% to 3%, when the moisture content is varied from 3% below optimum to 3% above optimum, should be considered for subgrading. The amount of personnel and laboratory equipment which would be required to make tests of all individual shale units would be prohibitive. It therefore becomes necessary that much of the decision whether to subgrade or not to subgrade depends upon professional field judgment. This judgment requires careful study of past performance and also careful compliance with previous laboratory studies made on individual shale units.

There are certain shale formations which have a definite history of poor performance. There is no question about the subgrading of these units regarding these areas. Usually these horizons have bentonitic characteristics.

There are also many other shale formations which most often are non-swelling and are desirable subgrade materials. These shale units are more stable in the subgrade and embankment section than are many of the surrounding soils.

The shales which fit the doubtful group between the good and the bad should be analyzed by laboratory study. The shale in this category cause delays in design since the sampling and laboratory testing must be carried out after the grade line has been established if there is to be accurate sampling of the subgrade materials involved in the subgrade.

THE HIGHWAY BACKSLOPE

The backslope is the angle of inclination of a lateral cut made along a roadway. The angle of the cut depends upon the physical characteristics and the weathering quality of the material excavated. The angle of cut is expressed as the ratio of the horizontal distance to the vertical (2:1 slope is 2 units horizontal to 1 unit). A bench in the slope may be needed in some formations to catch rock which would otherwise fall onto the roadway. The angle of the backslopes and width of the benches determine the total width required to make the desired cut. This must be taken into account to determine the volume of earthwork and the right of way requirements. Solid limestones are usually cut to a vertical face. A solid stable shale is cut to a 1:1 slope. Extensively weathered clay shales which are subject to slippage requires at least a 3:1 or flatter slope. Extensively jointed and fractured limestones, high upon the cut slope are subject to the hazards of block falls and will require a bench in the slope to catch the fall. Slope stability factors are evaluated in backslope design for each geologic unit. Consideration should be given to such items: planes of weakness; groundwater conditions; depth of weathering, erosion, frost wedging; slakage and slacking; induced moisture, whether exposed to or shaded from the sunlight; the direction of the roadway in respect to joint pattern.

The factor of economy is not disregarded in cut slope design, however, the major controlling factors are stability, safety, and appearance.

Cut slopes in glacial materials such as outwash, other Pleistocene materials and the Ogallala formation do not generally cause slope stability problems as the result of shear. These materials generally have sandy to silty properties and generally do not slide even on relatively steep slopes. Since occasionally shear failures may develop in these materials they must be studied with a certain amount of respect to prevent passing up shear problems.

As an example a cut slope of some magnitude within the glacial outwash material caused serious trouble along the West bluff of the Kansas River in the Kansas City area. The slide resulted when road cut exposed a soft layer of weak clay in a position low in the cut section. The clay layer was extruded out at the face due to the pressure exerted by the soil column above it. This escape of material allowed the cut slope of silty clay above it to break up in block shear and thus resulting in a major slide. Considerable property damage resulted to a home sitting near the top of the slope.

Shales in cut slopes encountered anywhere in the geologic section across the State can be somewhat troublesome and each unit must be considered on its own merits. Certain shale members may be well known as bad cut slope material whenever and wherever they occur. This behavior is due to their jointing and slick physical characteristics. These shale units present more serious slope problems when they occur above or even below companion limestone or sandstone members which carry water.

Our slope difficulties can be solved if we carefully consider all of these characteristics during the design and construction of the slopes and benches.

Erosion is a real problem on most of the nondurable cut slopes. To reduce erosion, slopes in soil and nondurable rock should be flattened as far as possible to allow the product of weathering to stay in place on the slope to increase the fertility and moisture retention for vegetative cover. The interception of drainage at the top of the slope is of prime importance. Weathered shales sometimes appear barren when erosion which causes a gullied and washed seedbed is really at fault. Vegetative cover is of prime importance to slope stability and careful consideration must be given to the slope to provide those advantages necessary for proper erosion protection.

We are in the process of using new methods of construction to improve slope stability. Techniques in smooth blasting of durable rock layers is giving us better cut slopes. Pre-splitting and angle drilling of shot holes for excavating rock cuts give promise of much better quality slope faces in the future. These methods have reduced the problem connected with overblasting and frost wedging of pre-fractured rock faces. Contractors have discovered that pre-splitting methods have reduced their cost of final shaping and cleanup time. The use of these better drilling and blasting methods are producing a more stable slope with a neater and clean appearance without adding cost to the excavation. These new drilling and blasting methods are given prime importance to keep down blasting vibrations for rock excavation carried on in urban areas where structural damage is of concern.

In design of backslopes the following considerations should be given. Unstable slopes do not tend to become stable with time, therefore, periodic maintenance may be necessary over the entire lifetime of the roadway. Under these circumstances, the cost of maintaining a slope having marginal stability may exceed the cost of right of way and the additional excavation that would have been required to build a flat, stable attractive slope in the first place.

COMPUTER PROGRAM FOR CLASSIFIED EXCAVATION

When the geology is portrayed on the cross section and the line indicating the classification is shown the geologic data can be coded on the computer data sheet. The following items are coded in this program: rock and common classification; whether boulder rock is present; the earthwork adjustment factor expressed in percentage; whether the material is limestone, sandstone or shale; whether a bench is required in the slope on either side or on both sides of the slope and the width required for the benches and indicate if there will be overbreakage. These data are then punched onto the computer cards. Having the information in this form allows variable adjustments of the grade line to more quickly determine a balance in the earthwork quantities.

GROUNDWATER PROBLEMS AND UNDERDRAINS

Groundwater is a major concern in highway construction for two reasons, seepage into the roadbed must be prevented and all domestic supplies must be protected. Groundwater problems which affect the stability of the road must be understood and resolved. Water carried into the roadbed by rock strata will cause damage to the pavement. Underdrains must be installed beneath the pavement to cut off the acquifer and to intercept the water. Underdrains are also installed under fill sections to save springs and to prevent soaking of the fill foundation. There is present and increased future demand for larger water supplies in our roadside parks and rest areas for more conveniences to the traveling public. This demand has brought about increased geologic investigation for water supplies and increased test drilling and pumping. Often times the desirable location for a rest area does not coincide with areas having a plentiful water supply. In such cases much investigation is required to determine water sources. Recommendations are made to Highway Right of Way on problems involving domestic water supplies from wells, springs and spring-fed ponds.

Many thousand feet of interception underdrain and underfill underdrains are installed each construction season. The specifications for these presently allow the use of both perforated and outlet type of drain. Six inch diameter pipe is the standard size used for underdrains.

The interception type underdrain which cuts the acquifer is the type most commonly used. The principle in the type of underdrain is to cut through the acquifer with the underdrain some distance away from its contact with the road surface thus leading the water away to the ditches rather than allowing it to saturate the subgrade.

Under some circumstances such as deep sand horizons and underfill drains it is necessary to install drawdown type of underdrains. This type of underdrain is a means for keeping the water table at what is considered a safe static level beneath the road structure. The blanket type underdrain is used occasionally in special problems which involves a series of layered acquifers. When this blanket underdrain is used directly beneath the road surface there is usually some deviation from the standard design in that the laterals and longitudinal sections are often specialized by deepening to intercept the necessary geologic strata.

All underdrains have thus far been designed to outlet on the surface. The benefits of control of subsurface water through underdrainage is sometimes difficult to determine in actual dollars. There is no question that the use of Geology in designing underdrains has led to the prevention of many surface failures and has eliminated many resulting maintenance problems. Also the actual saving and benefits to the driving public cannot be reduced to actual monetary figures.

Underdrains have been commonly used to prevent destruction of springs and seepage by the road fill and restore this water to the property owners usually water supplies of value are saved if possible by choice of alignment. At times it becomes economically impossible to locate the highway alignment which would probably prevent taking some natural source of water. In most cases some method of underdrainage can be provided which will prevent the destruction of the water supply.

LANDSLIDES

In areas of landslides and potential landslides a solution for stabilizing the slide zone is given. The limits of the material to be removed is designated. A recommendation is made as to whether the material removed should be wasted or used in the embankment. Treatment of the slide will usually require that steps be cut into fresh material below the slide zone and recommends the drainage necessary to intercept any source of water causing the slide. Recommendations made for the corrections of a slide is based on both judgment and laboratory shear tests.

SETTLEMENT

In fill foundations the limits of unstable material which will cause prolonged settlement of the embankment must be identified and shown on the plans. The plans must indicate whether or not the material is to be removed or whether some other treatment should be provided. Settlement studies and shear tests must provide the basis for treatment in deep seated settlement problems.

Settlement due to existing voids are strip pit areas or undermined areas where coal has been removed require that the loose overburden be removed down to stable material and backfilled with compacted approved material up to the road surface.

BRIDGE FOUNDATIONS

Bridge Foundation studies are made on the basis of geology. The strata are correlated within the foundation area of the bridge. Graphic representation of the foundation material for each bridge is portrayed on an engineering geology bridge sheet. This sheet portrays the strata in three dimensions. A Foundation Report which gives recommendations for footing design is furnished to the Bridge Designer. The report discusses the footing material, determines whether this is a pile or spread footing condition and whether or not the pile derives its strength from pure friction or from point bearing on relatively hard strata or from an intermediate condition of both friction and point bearing. The report lists such items as allowable footing pressures on various strata and material, the depth the pile will penetrate into a particular strata for safe bearing. It gives the groundwater conditions, class of excavation, stability in terms of settlement and shear, information for pile test loading, location of test piles, location of possible slides and slipouts, stream bank protection and other pertinent information.

Many of our structures, which are on pile foundation, are either strictly point bearing on solid strata or are both friction and point bearing. The latter case would be in weathered shales in which the tip of the pile penetrates several feet into the shale before reaching bearing. We have deep outwash sediments, where pile obtained their bearing by pure friction. This occurs in our glacial outwash and Rocky Mountain slope wash. Only in isolated instances does deep loessial soil provide the bridge foundation.

The foundation recommendation is based on: the cores of the formation, standard penetration tests, split-spoon samples, air hammer drives and casing refusal data. The casing drive test is used as a friction pile indicator. This test employs $2\frac{1}{2}$ inch casing driven to a final resistance of at least 50 blows per tenth of one foot penetration.

The air hammer drive data involves driving size A drill rod with a foot which is slightly larger in diameter than the driving rod. The machinery for making this test is a small air hammer and an air compressor which furnishes the energy for driving the rod. An air hoist is used to retract the rod when the driving is completed. The foot is detachable and is left at the bottom of the penetration when the drive is completed. The rate of penetration is timed in seconds to penetrate one foot. The drive record is graphically portrayed on the engineering geology bridge sheet. The information is valuable in both point bearing situations and especially valuable in friction pile situations. The air hammer information is used in conjunction with the split spoon samples and cores of the material penetrated. The air hammer penetration test has also provided a saving in time and expense. This information is substituted in part for drill soundings.

During construction the pile is driven to bearing under hammer. The requirements of pile penetration beyond the bearing formula is determined by the geologic condition. The location of the spread footings are given the final determination by inspection when the footings are open.

Construction finals for both the pile penetration and spread footings are recorded and graphically portrayed on a final sheet. This provides a valuable experience record for the various strata and conditions which are consulted by both the Bridge Engineer and the Geologist in making future recommendations.

RIGHT OF WAY PROBLEMS

Recommendations are made to the Highway Right of Way Department in problems involving domestic water supplies from wells, springs, and spring-fed ponds, which may be settled in the right of way agreement. This also involves recommendations for payment for oil wells, shale pits, quarry sites and other problems which are geologic in nature.

LEGAL ADVISEMENT

The geologist serves as an advisor to the Legal Department on matters similar to those involved in right of way procurement. These include evaluation of water supplies, oil wells, shale and gravel pits, quarry sites and other mineral deposits when these items become involved in litigation proceedings. The geologist serves as a witness in court proceedings on geologic matters.

MATERIALS LOCATIONS

This refers to materials which are needed for constructing the base and surface of the highway. The constant demand for materials needed for highway construction has placed a drain on the readily available supplies of sand and gravel, mineral filler and base aggregates. The Materials Engineer has drawn on the geologist to do this work. Previous materials mapping and inventory studies have been made by Geologists. A continual search must be made for available materials.

Presently our Highway Department has a materials mapping and inventory study under way. Photo-geology interpretation is being employed as the key for mapping. This work started in counties where the potential need for highway material is large and the known deposits are small. This current material mapping program will be expanded to nearly full state coverage. The basic work of this mapping program is carried out by geologists.

MEN AND MACHINES

It requires capable men and good equipment to carry out the work of a Highway Geology Program. These men must understand methods and procedures as well as being able to recognize the engineering problems that exist. They must also have an understanding of the engineering significance of the problem which they uncover.

Modern drilling and sampling equipment must be included in the picture of highway geology plan production. Proper tools and equipment have become a necessity. Without them it is difficult to produce a satisfactory geologic picture. The Engineering Geologist who requires cores and samples at various levels below the ground surface to answer his problems owes much to those who will produce good drilling and sampling equipment.

With both capable Engineering Geologists and good equipment the geologic information obtained should be of high quality as well as the engineering application made of it. If we are to keep pace with the ever-rising standards of good construction demanded of our present and future highway programs, the engineer and geologist can look forward to a tremendously challenging future.

INCREASING PROBLEMS

Presently we are faced with many more new developing problems than have confronted us in the past. We are approaching the time that highway locations are being pushed more and more onto unwanted land, thus our problems of foundation stability are rapidly increasing. Where we occasionally had either an abandoned mine problem, a waste strip pit area, a land fill trash area or a sliding hillside alignment we presently are finding all of these problems and more occurring simultaneously on that many individual projects of one program. The program presently being designed or constructed has both a sinking roadbed and a newly constructed bridge caused by subsidence around an abandoned oil well. This problem results from the well either being improperly plugged or from a cavernour condition at depth. This subsidence is brought about by or begun with the repressuring and the increased salt water disposal required in secondary recovery of this oil field. This problem is not one which is expected to occur frequently, neither is it one which is unique in oil field areas. It can be predicted, as time elapses, that this type of problem might become increasingly more common with time and age of development across older fields.

This year a problem of a caving limestone mine in the Metropolitan area of Kansas City occurred in our design work on Interstate Highway 635. This problem developed as the design was being drawn to completion about the time it was concluded that the undermining was substantially supported and was not a stability problem. Something happened on January 11, 1965 when a sizable circular area of the overburden approximately 200 feet in diameter, fell in with an abrupt collapse sounding a warning that not all was well with this mine. Twenty days later another similar area collapsed and by this time re-evaluation studies of this area for highway construction was already under way. With two more collapses six months later and the results of an investigation completed it was all but established that stabilization was necessary if this area were going to be used for the highway. We are still making estimates of costs for acceptable stabilization over the mine area.

In this same area along the proposed route of Interstate Highway 635 it is necessary to remove over a million cubic yards of unstable trash and debris in the interchange area with Interstate Highway 70. This is over a large land fill dump area which has a complex boundary in both surface area and depth. The removal in part will be to a depth exceeding 40' into saturated material. The backfill material to be placed into the saturation zone must be made of either rock or granular material. Although this is the largest of the trash areas to be removed along this route it is not the only one which needs stability attention.

Two areas of old abandoned coal mines separated by more than 200 miles are problems of stability in this program along both of the highway routes. There are open mine voids in various stages of collapse 15 to 20 feet below the surface. Connected with this collapse problem is one equally as troublesome, this is the stabilization of strip pit overburden workings which are 20 feet or more in depth. These workings are a mixture of broken pieces of limestone, shale and clay which is still very loose and partially soaked by the many ponds of standing water along the undulated surface of the workings.

Although no highway routes on the present program pass through the southeast Kansas lead a sink mining country there will be without a doubt new highways planned through this area in the future. A turnpike from Galena to Wichita which is under feasibility study at the present time, will go through the lead and sink mining areas. Although little mining is being done at the present time there remains the prospect of future operations at such time as these remaining resources can be economically produced on the world market. At any rate highway location and construction is difficult in this area. The surface of the ground in this region is nearly a wasteland with many huge piles of chat and mounds of cherty rubble. Below ground are a maze of tunnels, rooms and empty pockets. Only sketchy maps give a clue to the depth, breadth or length of these openings. A safe reasonable highway route alignment that could be placed over solid unmined ground is impossible. The task of determining the underground conditions to safely design a roadway is a complex one in this area since many of the openings are flooded and the abrasive broken cherty limestone make borings slow and expensive. Once the conditions below ground are discovered, detailed plans for filling or bridging must be developed.

Although the problems mentioned here are very normal problems in any area for an engineering geologist we have described them in this manner for the purpose of showing what has strong indications of pointing to the rapid increase in problems of this nature. As Highway locations are forced into more wasteland areas already destroyed or unwanted by man, we can see our job becoming more complicated and we are in the position to see the problems result in a rapid rise in road building costs. As we view this situation we also should be concerned with Man's destruction to the surface of the earth. We can look back and see what problems the lack of control in these many past operations have forced upon us. Yet we look at what is going on in many of these same operations today and wonder if we are not just repeating the same old mistakes. From these observations the work of the Engineering Geologist is becoming more complex right along with other professions and there seems to be nothing in sight which would show a Dead End Road ahead.

SEISMIC AND RESISTIVITY MATURED

by

Robert E. Mitchell, Geologist

Soiltest, Inc.

Evanston, Ill.

SYNOPSIS

Seismic and resistivity instruments are proving themselves to be an accurate, low cost and rapid means of determining the subsurface conditions to depths of 100 feet, an area of great interest to the highway construction field. Using these modern, electronic instruments, the number of borings can be reduced significantly.

A method of complementary usage has been developed in recent years which utilizes the capabilities of both seismic and resistivity techniques. The complementary usage technique results in extremely accurate interpretations of the subsurface conditions. Some specific applications of these techniques include accurately determining bedrock depth, identifying refusals, determining bedrock rippability and determining directional trends of joints in bedrock.

A new breed of professional men--engineering geologists--is developing. They will be skilled not only in analyzing soil and bedrock conditions but in correlating past and present geological activities to the engineering capabilities of the materials.

At long last the methods of testing for shallow depth subsurface conditions by seismic shock and electrical resistance have matured. In retrospect, it is fortunate that the two methods have lingered in adolescence during the research and development stages. For today the agriculture, soil conservation, engineering and geological professions, hard put to keep pace with expanding economics around the world, need and are demanding proven and electronically refined engineering geology exploration tools to conduct more accurately, economically and effectively shallow depth, subsurface investigations.

Geological Surveys

The information for the initial geological surveys and maps for bedrock, sand and gravel, ground water, and soils deposits was assembled by men who, for the most part, relied on their interpretation of ground features only. In some areas the geological interpretations of subsurface conditions could be correlated to additional evidence such as nearby exposures and local wells or borings. In many areas they could not be. Hence, the information assembled, while basically true, was in error because of the lack of substantiating evidence. Today the seismic and resistivity instruments can supply substantiating the evidence a geologist requires, when making his interpretation of shallow depth subsurface conditions.

Engineering Surveys

Borings will always be required on engineering surveys because of the need to determine bearing capacities and other engineering characteristics of soil and bedrock deposits. However, for most projects the number of borings can be reduced significantly if seismic and resistivity tests are used to supplement borings. Seismic and resistivity tests can also be used to:

1. Indicate what type of drilling equipment is most suitable for use on the job.
2. Indicate where borings should be located for most effective test results.
3. Extend boring and probing information over a larger area.

Common Sense Use of Seismic and Resistivity

Although there are many exotic uses for the seismic and resistivity methods, it is best in general field use to think of the equipment as representing practical exploration instruments, such as a drill rig or back hoe, designed to do an effective job of determining subsurface conditions.

The Seismic Refraction Method

The seismic refraction method is based on the principle that subsurface seismic or shock waves travel through materials of different density at varying speeds; the more dense and hard the material, the higher the seismic velocity. There are engineering seismographs, such as the Soiltest R-150 Terra-Scout, that enable first wave arrivals to be timed by visually displaying the seismic wave on a cathode ray tube. Others, such as the R-117 series Seismic Timer, use a series of digital readout lights.

The seismic waves are initiated by striking the ground surface with a tamper or hammer at increasing intervals from a detection device called a geophone. The first wave to arrive at the geophone is timed for study purposed. As the hammer is moved away from the geophone, each first arrival of seismic wave is timed in milliseconds. These arrival times are plotted on a graph at increasing horizontal hammering distances from the geophone. The plotted lines are studied for possible trends and then joined by a series of sloping lines. The slope of each line is read as seismic velocity in feet per second (ft/sec.); the flatter the slope the higher the seismic velocity.

According to refraction laws the depth of a seismic investigation is approximately equal to one-third of the horizontal hammering distance from the geophone. The actual depth is determined by formula. Depth as well as the velocity of subsurface materials can be determined by a seismic refraction sounding.

Seismic velocities also can be correlated to the classification of materials which, in turn, is indicative of the type of material as well as the hardness or density. Removal or rippability characteristic of the soils and rock also can be correlated to the velocity.

Visual Display of the Seismic Wave

In seismic studies the seismic wave is not only "timed" but also analyzed for wave shape and appearance and disappearance of refracted distortions along the side slopes of the wave. In seismic refraction work, the visual display figure of the seismic wave has only recently been introduced in the portable refraction seismographs.

The methods of recording this seismic wave differ from mechanically plotting the wave shape on a roll of moving paper to displaying the wave on the screen of a cathode ray tube. Because the mechanical stylus plot of the seismic wave is a roll of moving paper is not as sensitive to minute changes in wave shape as the cathode ray tube, it is best used for very deep exploratory studies involving the use of dynamite and truck mounted instruments.

Shallow depth refraction studies down to 100 foot depths are probably best performed with a hand portable refraction seismographs. One instrument, called The Terra-Scout features a visual display of the seismic wave on the screen of a cathode ray tube. The shape of the wave is "retained" on the viewing screen for about 90 milliseconds. This is ample time for "timing" the wave and observing wave characteristics. If a permanent record of the wave is desired, a camera attachment can be used.

The importance of the visual display of the seismic wave best can be noted by observing the development and disappearance of refracted distortions along the side slope of the wave. On one project, a pictorial record was made of the seismic wave at 2 foot hammering intervals from the geophone. The subsurface soils consisted of terrace sands with a 12 inch thick silt and clay layer at a depth of 10 feet. The pictorial record of the seismic wave is shown in figure 1.

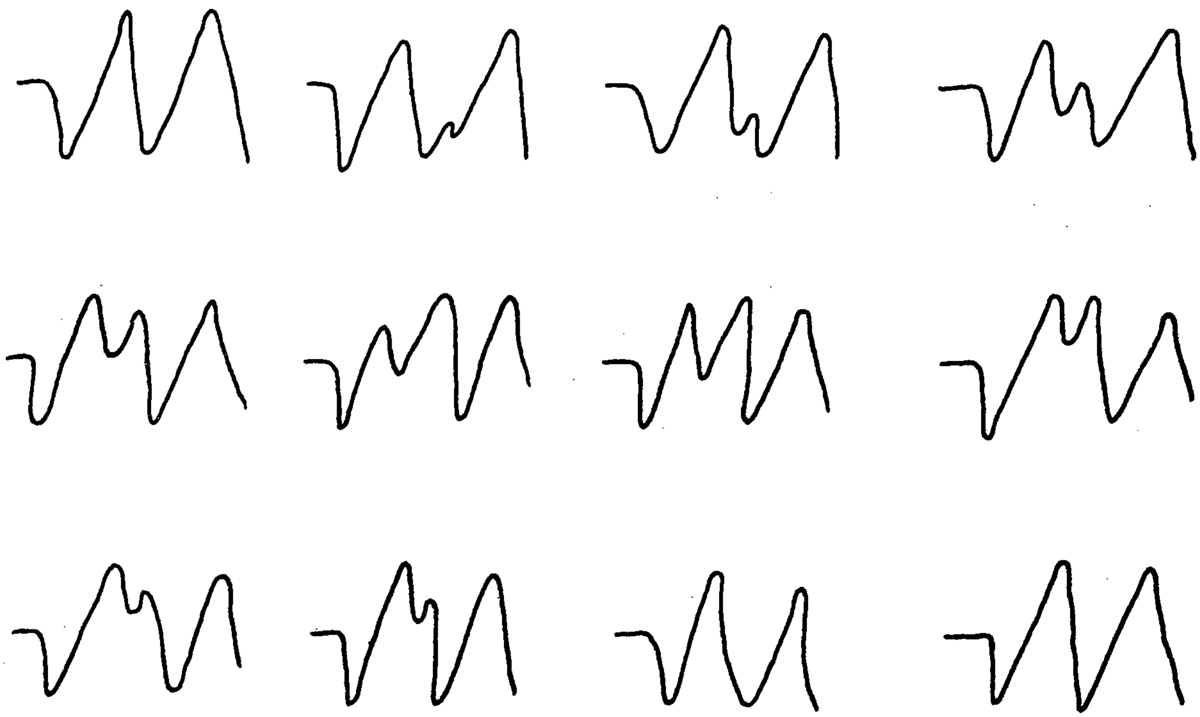


FIGURE 1

Note the systematic development and disappearance of the refracted distortion along the side slope of the wave. Remembering that the shape of the seismic wave reflects the movement of a sensing device called a geophone, it can be seen that the geophone was being acted upon by a secondary shock wave that refracted along the silt and clay layer. The refracted distortions disappeared when a more dense terrace material was detected below the silt and clay layer.

The Resistivity Method

Another engineering geology technique is the resistivity method. This method is used to determine the resistance of subsurface materials to electrical current; the more dense and dry the material the higher the resistance. The depth of a resistivity investigation is a function of the spacing between electrodes placed on the ground surface.

Many different types of electrode configurations may be used; but at the present time most of the portable resistivity units (such as the Soiltest R-30 Michimho) are designed for maximum investigation depths of from 75 to 100 feet using the Wenner 4 electrode configuration. The inner two serve as the receiving or potential electrodes. As voltage is applied, a theoritical equipotential bowl of electrical activity is induced around each current electrode. At each depth investigated an electrical bridge, calibrated to read resistance or conductance, is used to measure the variation in potential between the inner electrodes. For final interpretation, the readings can be converted into ohm/feet or ohms/cubic centimeter resistance at layer intervals beneath the ground surface.

Complementary Usage

The seismic and resistivity methods have never enjoyed the prominence they do today. This can be attributed in large part to an early eagerness to over-sell the merits of the two methods while not recognizing the inherent limitations of both seismic and resistivity testing. Limitations, however, can be reduced when using the two methods together.

Because the acceptance of "Complementary Usage" dates back only a few years, the earlier proponents of either seismic or resistivity studies were often led into blind alleys.

For example, to make realistic interpretations of seismic and resistivity testing without the benefit of drilling logs or observation of local exposures, the operator must recognize that the seismic method used alone cannot tell the difference between a medium velocity bedrock formation or ground water deposit. He must recognize too, that the resistivity method used alon cannot differentiate between a high resistance bedrock formation or sand and gravel deposit.

With the acceptance of "Complementary Usage," however, realistic interpretations can be made of the soil, ground water sand and gravel and bedrock deposits under almost all types of conditions. And these interpretations can be made with out the benefit of drill logs or nearby exposures in most cases. But more important, the number of test borings can be reduced greatly and those borings performed can be pin pointed for most effective results.

An example of the merits of complementary usage: a seismic survey indicates a high velocity, 6200 ft/sec. material at a 20 feet depth. Question: Is the material a limestone bedrock or terrace sand or glacial till deposit with ground water, all known to exist in the area?

In reality, there is no way of making a positive interpretation using the seismic data alone, because both the bedrock and ground water can have 6200 ft/sec. velocities. (Velocity of water $4750\pm$ft/sec.)

Another example: a resistivity sounding has detected a high resistance 200,000 ohm/cu cm material at a $42\pm$ feet depth. Question: is the material igneous bedrock or sand and gravel, both known to exist in the area. Again, there is no sure way of making a realistic interpretation using the resistivity data only, because both the bedrock and sand and gravel deposits can have similar high resistance values.

The only way realistic interpretations can be made of the preceeding hypothetical subsurface situations is to use both methods in the same area. This means performing both a seismic and resistivity sounding over the same test area. Then correlative data, established in this country and around the world, could be used to identify the various subsurface conditions, as follows:

Bedrock: high velocity, high resistance (except some shales) and weathered or fractured formations.

Ground Water: high velocity, low to very low resistance.

Dry Sand and Gravel: low velocity, high resistance.

Cohesive Soils: low velocity, low resistance.

Use During Plan and Design Stage

Before and during the planning stage for a structure or a road, it is usually desirable to know something about the general subsurface conditions, such as—at what depth is bedrock or ground water, are the subsurface soils cohesive or granular, are there available sources of aggregate close to the job site? All of this information can be compiled quickly by a seismic and resistivity survey.

Presenting Seismic and Resistivity Data

The presentation of seismic and resistivity data on a plan and cross section print can be similar to that used for a boring or probing.

The location, direction, and length of a seismic and resistivity sounding should be presented on the plan view as shown in Figure 2.

FIGURE 2

SM # 1

=Reverse Sounding; first pound direction, left to right.

SM # 2

=Single Spread Sounding; symbol placed midway between geophone and first critical distance.

RY # 1

=Resistivity Sounding; Arrows at widest spacing of current electrodes.

The number, stationing and elevation of the seismic or resistivi-sounding and test results can be presented on the cross section view, as shown in Figure 3.

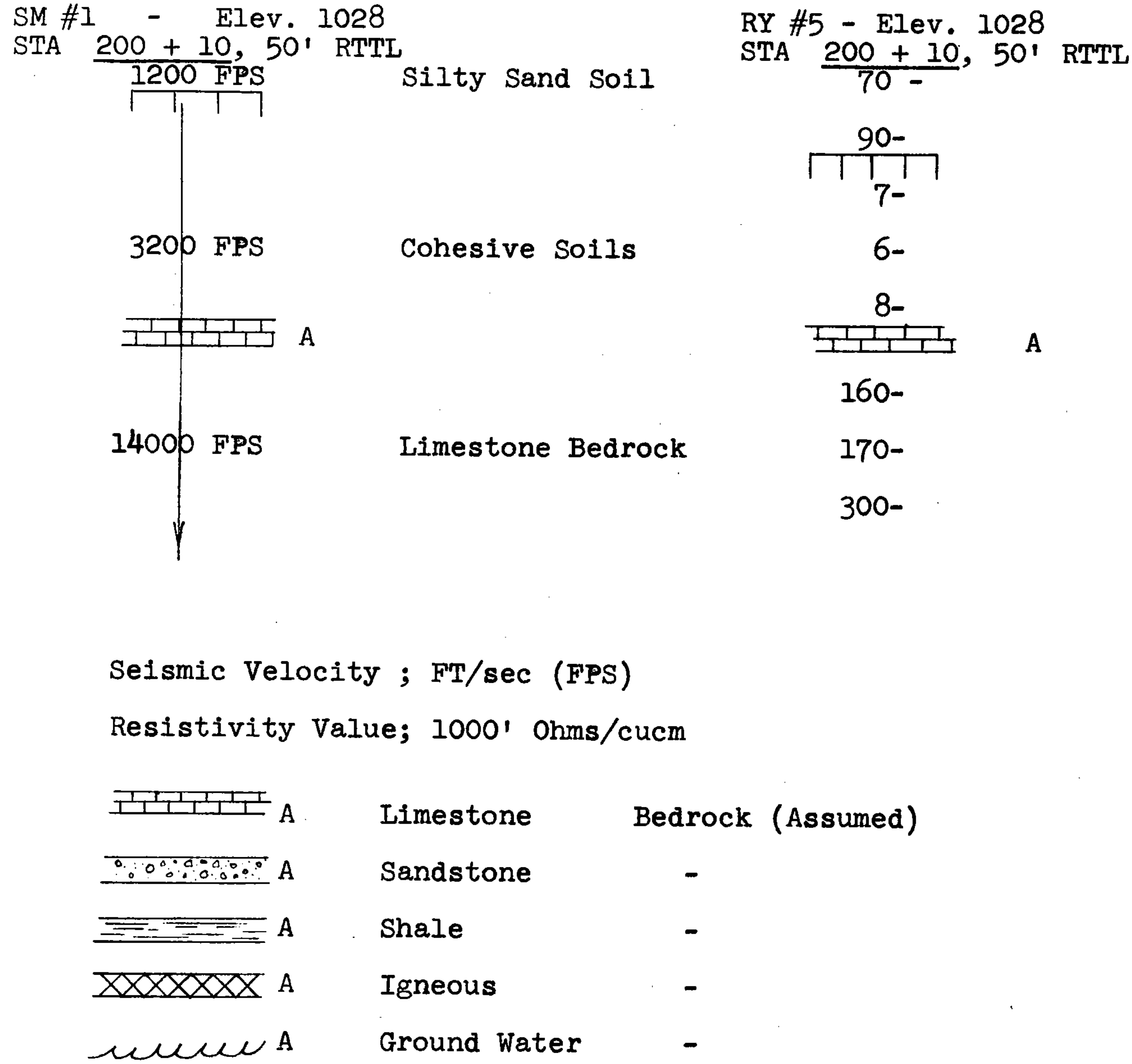


FIGURE 3

Simplest Use of Seismic and Resistivity

An often overlooked aspect of seismic and resistivity surveys is the use to discover the absence of bedrock, ground water or sand and gravel deposits above a certain depth. For example, if a medium or high velocity material is not encountered out to a 150 foot hammering distance from the geophone, this would indicate the absence of bedrock or ground water down to about a 50 foot depth. And, if a high resistance material was not encountered out to an electrode spacing of 35 feet, this would indicate the absence of a clean granular deposit such as sand and/or gravel down to about a 35 foot depth.

Identify Refusals

The accepted manner of identifying refusal as bedrock or a floating boulder is to take a rock core. If rock is cored through an acceptable distance, the refusal is generally identified as bedrock. However in many glaciated areas and on slopes adjacent to exposed bedrock formations, the erratics and floating rock fragments can be much larger than 5 feet in diameter.

A very effective and economical way of identifying a boring or probe refusal is to conduct a seismic sounding over the refusal and correlate the seismic velocity to either soil or bedrock. If a low velocity material is detected below refusal elevation, the refusal can be identified as an erratic or floating rock fragment. If a high velocity material is detected at about refusal elevation, the refusal can be identified as bedrock.

Pin Pointing Bedrock Depth

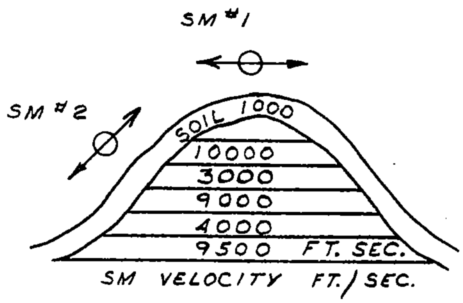
The seismic and resistivity method cannot be used to pin point the depth to bedrock if the surface of the rock is erratic. Only a boring can pin point the depth to bedrock. However, 12 feet from the boring, the bedrock depth might be much deeper.

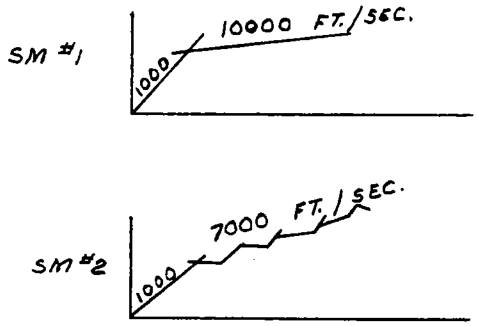
The seismic and resistivity method would have given an average depth to the bedrock, over the area covered by the seismic or resistivity survey.

Bedrock Rippability

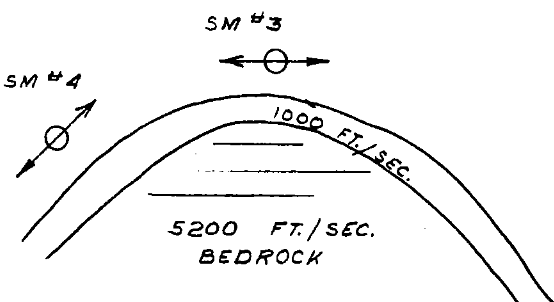
The rippability of a bedrock formation depends not only upon hardness but upon the formational and cementing characteristics of the bedrock. A thinly bedded rock, consisting of alternating hard and soft layers, is more easily ripped than a massively bedded rock consisting of medium hard layers throughout. In areas where side slopes are present, the formational characteristics of a bedrock can be determined by a proper technique of sounding layout shown in Figure 4.

FIGURE 4

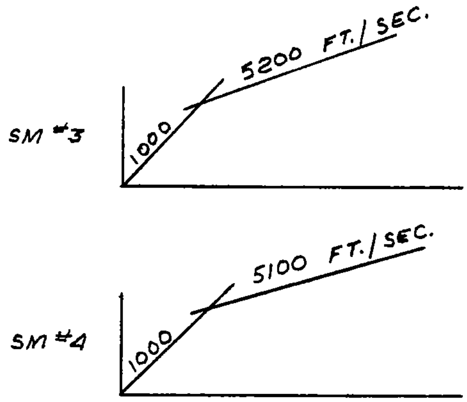




THINLY BEDDED BEDROCK



UNIFORMILY HARD BEDROCK



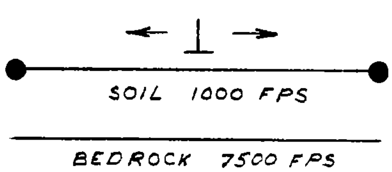
In areas where side slopes are not present, the seismic velocity across the bedding layers can be determined by up-hole seismic study. In this method either the geophone or shock source are placed at the bottom of a boring and the seismic velocity measured directly up towards the ground surface.

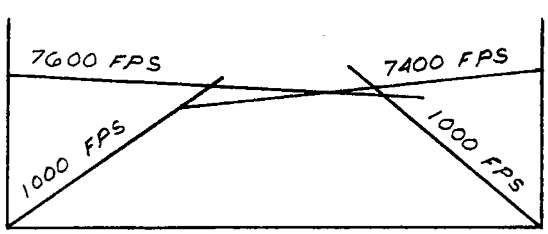
In areas of dipping bedrock the soundings should be positioned parallel and at right angles to the bedding layers.

Dipping Bedrock

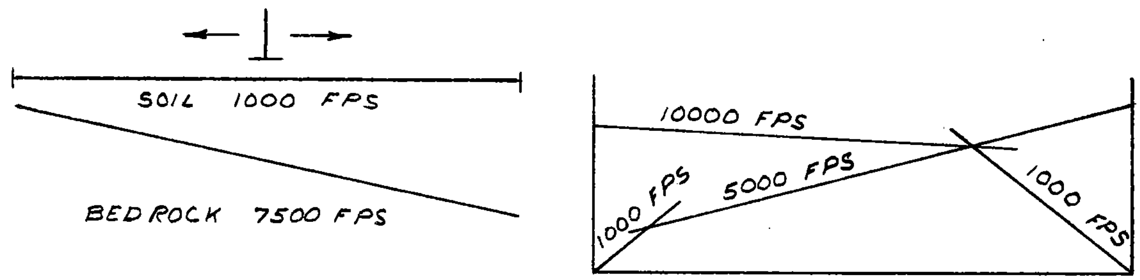
Seismic test results will indicate the actual hardness of bedrock and also the flatness of the bedrock surface, dipping characteristics and bedrock pinnacles or depressions. These relationships are shown in Figure 5.

FLAT BEDROCK

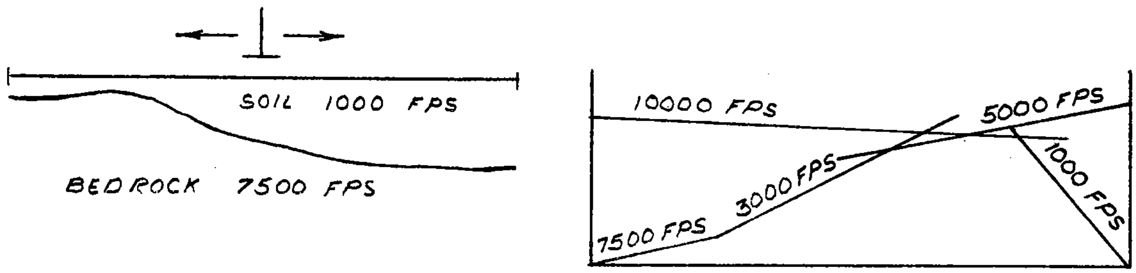




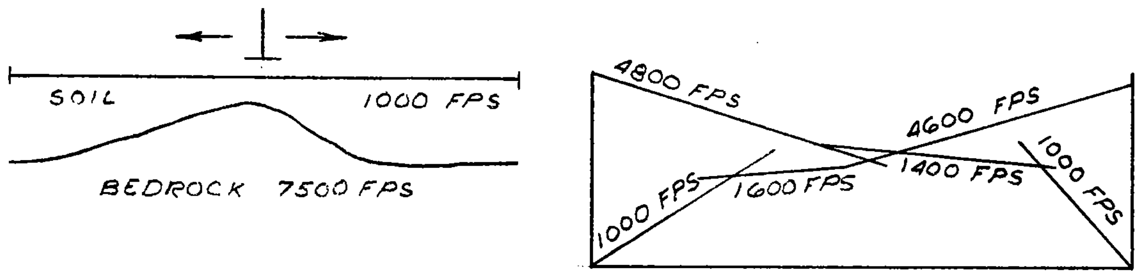
DIPPING BEDROCK



FLAT THEN DIPPING BEDROCK



BEDROCK PINNACLE



BEDROCK DEPRESSION

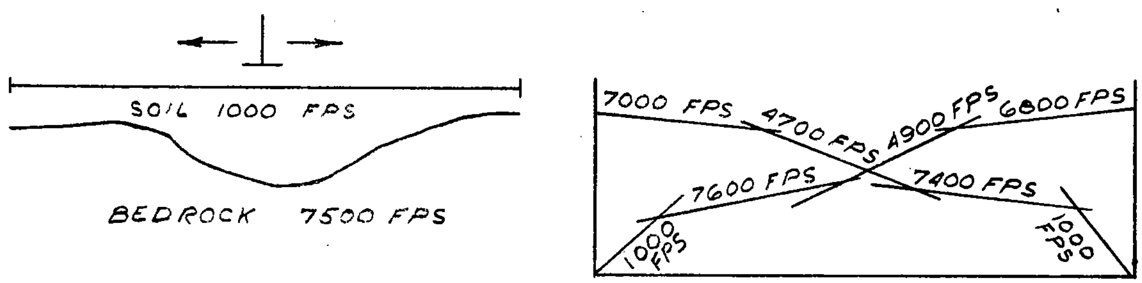


FIGURE 5

Determine Directional Trends of Joints in Bedrock

The directional trends of depositional planes on joints in bedrock can be determined by laying out seismic soundings in a **radial pattern** on the ground surface. The seismic shock will travel more rapidly parallel to the depositional planes and joints in the bedrock than across them -- see Figure 6.

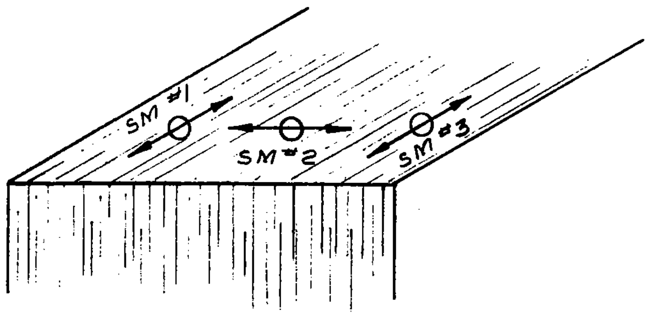


FIGURE 6

A knowledge of the directional trend of depositional planes or joints in bedrock can be of value on many engineering projects, for example:

1. Bedrock generally can be ripped more easily across the depositional planes or joints than parallel to them.
2. Ground water will flow more readily along the depositional planes or joints.
3. A blast hole pattern planned along a rock controlled cut should take into consideration both the direction of the depositional planes or joints and the direction in which the material is to be moved. This will indicate the most effective and economical type of spacing for blast holes.

Contract Drilling

Until recently, contracts let for site investigation studies or searches for aggregate have generally been performed by hand or machine auger borings. In some areas back-hoes or large shovels have been used, especially in searches for bedrock or sand and gravel deposits.

Today, however, there is a growing trend in the contract drilling field to use seismic and resistivity testing in conjunction with hand or machine borings. The organizations that use seismic and resistivity instruments are able to offer a complete range of subsurface investigation studies, including:

A. Drilling and Sampling.
B. Sampled borings supplemented by seismic and/or resistivity testing.
C. Seismic and resitivity testing only.

If site investigation studies can be performed using seismic and resistivity tools with a limited number of borings, it is probable that this economical type of survey will appeal to organizations which formerly omitted or restricted subsurface surveys because of the high cost.

A New Breed of Geologist and Engineers

Today, many universities recognizing the need for experts who can interpret and analyze shallow depth conditions, are educating engineering geologist. These men will be skilled not only in analyzing soil and bedrock conditions but in correlating past and present geological activities to the engineering capabilities of the materials.

With the world on the move it is imperative that the structures and roadways of tomorrow, for reasons of economy and safety, be planned, designed and constructed with a complete knowledge of the engineering and geological characteristics of the subsurface materials. It is imperative too, that the shallow depth resources of the earth are utilized. Resources which although representing one of the greatest treasures endowed by nature are not yet fully utilized.

It is to these ends that the engineering geologist, schooled in the field of shallow depth subsurface exploration, will serve mankind.

TECHNIQUES OF ENGINEERING GEOLOGY IN EVALUATION OF ROCK CORES FOR CONSTRUCTION MATERIALS SOURCES

By Robert D. Michael
Geologist, Iowa State Highway Commission

ABSTRACT

Cores drilled by commercial materials producers in their exploratory programs may be submitted to the Iowa State Highway Commission for evaluation.

Most of these sedimentary rock cores are logged by suites using an area generalized geologic column. Beds are separated into workable "ledges," and samples for physical and chemical tests are taken accordingly. Cross sections are drawn using core locations, relative elevations, and overburden thicknesses. Test data is superimposed on the cross sections to give an overall picture of the formation's potential in regard to construction materials.

After prospects are opened into quarries, follow-up correlations are made between core evaluations and actual ledge conditions and production data.

INTRODUCTION

A recent Bureau of Public Roads publication (4) states, "of the various materials used in highway construction, aggregates constitute one of the major elements of cost, amounting to between 28 and 33 percent of the cost of all materials and supplies and between 12 and 16 percent of the total construction cost (excluding costs of right-of-way and engineering.)" These figures would, of course, be even higher if the cost of rock related products such as cement, lime, rip-rap, cut stone, brick, etc. were included.

This paper is presented with the hope of helping to lower, or at least level off, these costs and still maintain a high, or attain a higher quality of product.

Defining The Problems

One of the primary aims of the engineering geologist is to identify and evaluate the various rock beds exposed on or near surface in a given area. This in itself may be quite a problem, but the real difficulty arises when these correlations must be recorded on paper for the use of other geologists, engineers, inspectors, quarry operators, or even for his own use at some future time. Identifying features of a rock exposure may be obvious to some people and completely hidden to others. Lapses of time between visits to a quarry or exposure also seem to aid in masking once obvious features. It is the duty then of the geologist to use points of identification which can be easily recognized by the trained and untrained observer and to record these observations in some manner so that they can be fitted into the large jigsaw puzzle of regional geology. Mitchell (3) suggested the existence of these problems in the duties of the engineering geologist related to highway construction.

Recording Geologic Data

It has been found that a simple description of the general geologic sequence supplemented with a columnar drawing is the most effective method for recording. Several other methods have been tried including the use of colored photographs with superimposed bed numbers, but none have proved nearly as effective. All columnar drawings are scaled to a 1"=5' basis so that they may be laid out side by side and have a direct bed thickness relationship. The drawings have become very significant in tieing in regional geology because of the ease in comparing a large number of sections in a relatively short time, such as Lahee (2) has indicated in the correlation of graphic oil well logs.

The geologic descriptions are standardized to include rock type, color, grain and texture, hardness, bedding characteristics if significant, fossils, special features, and thickness all in that order. It is important here to stress a degree of simplification. Lumping together of beds when possible usually will serve to make easier identification. When a new source of rock is being sampled, on a bed to bed basis for quality, the geologic description is keyed directly to the engineering characteristics of the rock. Emphasis is not placed on formal stratigraphy in this description. It is used only as another aid in further identification of the beds.

Actually, we have found in the midwest that the physical characteristics of the rock are very closely alligned with the established stratigraphy. For example, a ledge of high quality rock, at the base of the Coralville Member of the Cedar Valley Formation, can be traced using physical tests across four counties in Iowa, a distance of over 100 miles.

Tests

Evaluation of high quality sources is primarily based on two tests: 1. freeze and thaw "Method A" which measures the breakdown of the rock after 16 cycles of freezing and thawing in a water-alcohol solution and 2. the Los Angeles abrasion test. Service record is also used if available and chemical composition is considered. For Portland cement coarse aggregate each ledge of rock must separately meet specifications. The Iowa State Highway Commission's Book of Specifications (1) elaborates further on these requirements.

Use of Cores

In Iowa, cores may be submitted to the Highway Commission for testing and evaluation. These may have been drilled by commercial producers, construction companies, county engineers, mineral developers, or private land owners. The cores have varied in size from AX 1 1/8", to BX 1 5/8", to NX 2 1/8", and up to 4" in diameter. The usual type submitted is the NX size.

The role of the engineering geologist, in the handling of cores, is to scientifically predict the quality of the rock deposit in question if it were opened into a quarry or mine for the production of construction materials.

Many variations on the procedure have and can be used. The following would be a more or less ideal approach to the problem, broken down into 10 major steps as follows: 1. field checking of coring site, 2. drilling, 3. assembling cores for logging by suites, 4. marking major breaks, 5. marking minor breaks, 6. describing general geologic section, 7. logging bed thicknesses and recovery, 8. dividing cores into quality "ledges" for sampling, 9. evaluating test data, and 10. comparing core data with opened quarry.

Welp (5) mentions some problems in sampling of cores useing these methods in a paper for the Highway Syposium in 1963.

Step 1 - The core site should be visited to establish any relationship between possible rock exposures or quarries and the proposed cores. A geologic description is made of the exposed beds. The use of aerial photos at this time will aid in spotting core sites.

Step 2 - During drilling the geologist should visit the site several times, depending on the number of cores taken, to advise the driller as to the depth to drill on each core. At least one core should be long enough to tie in the stratigraphy with known regional data. Each additional core is then correlated with the long core and need only be deep enough to penetrate through the quality ledge.

There is a tendency for many properties to be underdrilled with cores being too short to tie in with a known stratigraphic level. On the other hand, there can be a waste of money in drilling cores a fixed depth, when they may penetrate or bottom out 15 to 20 feet in a shale, siltstone, or other material which may have little commercial potential and which is of no value to test.

Referring to Figure 1, Core 1 has been drilled long to pick up lithologic contacts which will establish the stratigraphy. It will also be of value to assess potential future reserves. Core 2 is too short to tie it in with definite stratigraphic check points and has probably not penetrated the full extent of the high quality ledge of Bed 4. Core 3 has been unnecessarily drilled over 15 feet too long. The full extent of the high quality limestone has been determined and it would have been more economical to stop the drilling after penetrating through Bed 5 or even at the top of Bed 5. In an actual situation Beds 6 and 7 of this core would be discarded after logging. No testing would be done on Bed 6 because it would be available from Core 1, and the amount of core in Bed 7 is too little to make up a sample. Core 3 would be considered of adequate depth, having established the full extent of Bed 4 and has picked up the top of the siltstone of Bed 6 for a stratigraphic contact.

There may be a problem in establishing upper contacts rather than lower because of either: 1. an irregular bed rock surface caused by erosion or 2. structural features which are reflected in dipping beds or faults, or 3. a combination of both.

TYPICAL CROSS SECTION OF CORE DATA

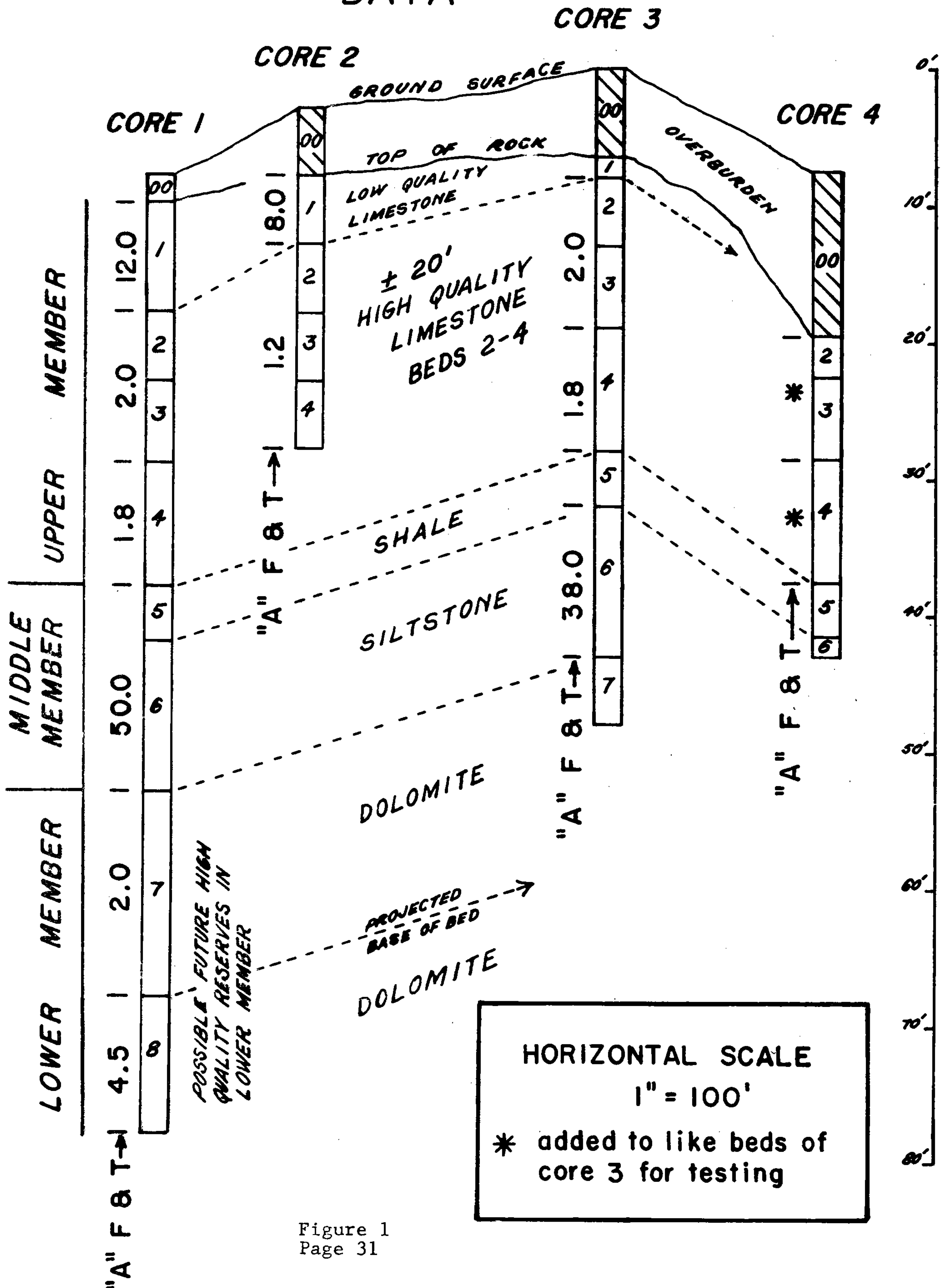


Figure 1

It is advantageous to plot a cross section during drilling using relative location and elevation of core sites, overburden thicknesses and depths. Identification features of the various cores can be plotted here to indicate the presence of structures and also to program depths to drill in additional cores.

Step 3 - Formal lodging of the cores is done in the laboratory where all of the cores from one area or property are assembled. In the average situation, where sedimentary formations are concerned, the cores will have definite similarities. Each core is then logged as part of a "suite" instead of on an individual basis. Anomalies may, of course, be present and these are noted for the particular case.

Step 4 - Most cores are quite clean when received. If not, they should be washed free of drilling mud, rock flour, and soil. Painting the surfaces with water also will bring out the true color and texture. On the initial study of the suite obvious common breaks in lithology or points of identification (change in color or rock type, grain, concentrations of fossil species or chert, etc.) are marked on each core. These points are also plotted on the field cross section if they have not previously been noted during the drilling stage.

Step 5 - Additional study of the cores will reveal secondary bedding breaks or common points of identification which are not as easily recognized but which may be significant (shale sequences or bands, weathering features, textural differences, etc.). Selected sections of cores are split to check the "fizz" reaction with acid and to study grain size, crystallization features, fossils, etc.

When all beds in all cores are identified and correlated, then each similar bed in each core is numbered with chalk, starting with Bed 1 at the top of the core.

Step 6 - A geologic section is described from the total number of beds represented in the cores. If a field geologic section was made of natural exposures or quarry rock in the vicinity, this is also compiled to make one general geologic section of exposed rock and core data. Here, again, the beds are numbered with Bed 1 at the top of the section. This numbering method allows for beds to be added to the section if further quarrying in the floor or deeper coring reveals undescribed strata.

Step 7 - A numerical log is then compiled, as in Figure 2, of each core listing the bed number, thickness of bed, footage recovered of bed, and total depth drilled to base of each bed. Anomalies in beds may be noted on this data sheet.

Step 8 - The cores are now ready for sampling. Each core is divided into workable "ledges" as would be done on an actual quarry face. The thickness of the "ledges" sampled depends on the economics of rock production in the particular area of the cores. If a "ledge" appears very similar in several closely adjacent cores, (see Figure 1), a combination may be made to make up one representative sample.

It should be noted here that beds which have less than 100 percent recovery will tend to show distortion on the test results. In most cases the recovery has an inverse relationship with the quality indicated by the physical tests; the lower the recovery, the higher the quality appears.

The Method "A" Freeze and Thaw and the Los Angeles abrasion are the two main tests performed. Chip samples for chemical analysis may be taken and representative sections of each bed may be saved as a reference for the particular geologic member or formation.

Step 9 - When test data is received from the lab, sample results are plotted on the cross section to provide a full picture of the quality of the beds. Combinations of ledge values can be computed to simulate results of a selective quarrying process in quarry production. The geologist must also decide if there has been any distortion of test results due to coring loss or testing error.

Step 10 - When the deposit in question is opened, the geologist should compare the geologic description made from the cores with that of the actual exposed face. Many times the same description can be used for quarry control. However, it is not uncommon for new beds or variations in thickness to show up in quarrying.

Bed No.	Core 1			Core 2			Core 3			Core 4		
	Drill	Rec	Total	Drill	Rec	Total	Drill	Rec	Total	Drill	Rec	Total
00	2.0	-	2.0	5.0	-	5.0	6.5	-	6.5	12.0	-	12.0
1	8.0	5.0	10.0	5.0		10.0	1.5	1.0	8.0	-		
2	5.0		15.0	5.0		15.0	5.0		13.0	3.0		15.0
3	6.0		21.0	5.0		20.0	6.0		19.0	6.0		21.0
4	9.0		30.0	5.0		25.0	9.0		28.0	9.0		30.0
5	4.0	3.0	34.0				4.0	2.5	32.0	4.0		34.0
6	11.0	10.5	45.0				11.0	10.0	43.0	1.5		35.5
7	15.0		60.0				5.0		48.0			
8	10.0		70.0									
Total			70.0			25.0			48.0			35.5
	Loss in Bed 1 at base in silty zone.						Bed 1 silty throughout. Bed 5 calcareous			Bed 4 highly calcareous.		
100% Recovery Unless Indicated												

Figure 2. Numerical Log of Cores as Described in Step 7 of Use of Cores.

The presence of shale sinks, collapse zones, or weathering features can often be missed in coring. Steps can be taken to cut down this possibility by using a closer coring pattern during the drilling operation.

Production tests should be compared with those computed from the core data. If the cores have been patterned correctly and the recovery has been good, there may be up to 100 percent accuracy in prediction.

ACKNOWLEDGEMENTS

The methods of handling cores described in this paper were developed jointly during the past ten years by the following members, including the author, of the Geology Section of the Iowa State Highway Commission: Theodore Welp, Chief Materials Geologist, Kermit Dirks, Materials Geologist, and James Myers, Materials Geologist.

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THE EFFECTS OF PEAT DEPOSITS ON HIGHWAY DESIGN IN IOWA

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INTRODUCTION -

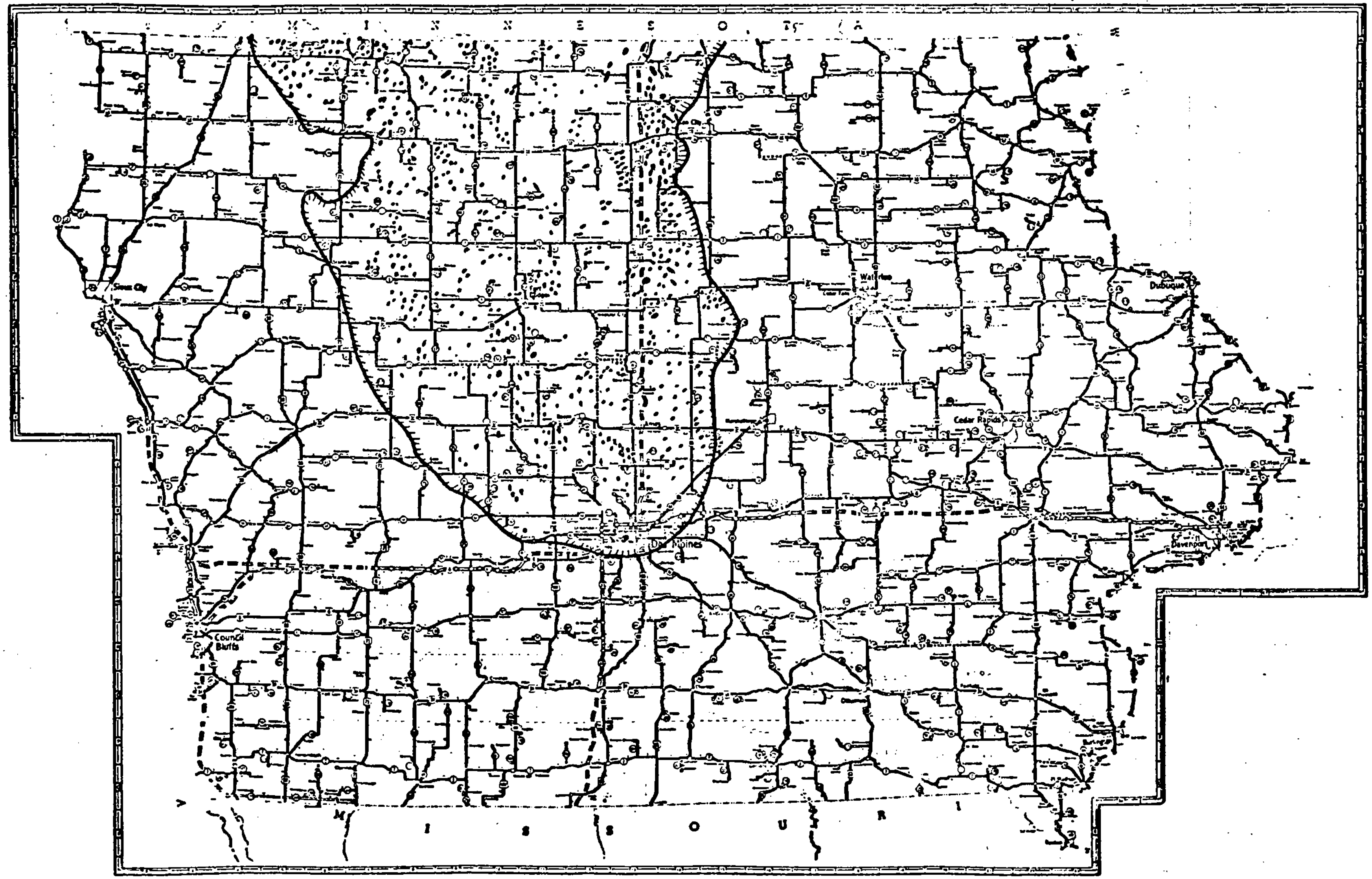
This report is prepared to encompass an understanding of the occurrence of peat deposits in Iowa, and the effects they have on highway design. The need for highways with Interstate Standards in Northern Iowa has made it necessary for engineers to locate highways over adverse soil conditions. One of the most costly, although interesting, of these conditions is that of peat. Many problems have developed, and some of these will be discussed here.

Origin of Peat -

Iowa's peat region involves the north central one-fifth of the State, on the Des Moines Lobe of the Cary Drift. Figure 1 shows the region described as the Cary Drift. The peat deposits shown here are exaggerated in size and include those deposits which we would consider as surficial muck deposits. The average depth of peat sediment is approximately ten feet, below which is the Cary till. Many of the deposits yield only a surficial black muck less than five feet in depth. The maximum depth of peat sediment that we have encountered is twenty-five feet.

It is acknowledged that there are other areas within the state in which organic sediments occur. However, they cannot be included as peat deposits and will not be considered at this time. Examples of these organic sediments would include slackwater deposits on the flood plains of many of our streams, and channel-fill deposits in cut-off meanders.

In order to grasp an understanding of the origin of peat, some consideration must be given to the Pleistocene history of the State. With the advent of radiocarbon dating, much detailed information has been published for dating of the Wisconsin stage.



DesMoines Lobe of Cary Drift
with Locations of Peat Deposits

Figure 1

Table 1 is taken from data published by Walker (1) and serves to place the relative ages of the Wisconsin substages in perspective.

Table 1

Stage	Substage	Date (B.P.)	
		0	Post Cary Eros. Possible cycles at 6600 BP Post 2000 BP
Recent	Post Cary		
		11,790± 250	
	Cary	12,970± 250	
		13,300± 900	
	Tazewell	14,700± 400	
		17,000	
	Farmdale	24,500 + 800	
	Iowan	29,000	
		37,000	
	Pre-Iowan	37,000	

From the above time-table, the youthfulness of the Cary Drift can be seen. The retreat of the glacial ice constructed a topography of swells and swales, many of which resulted in closed depressions. It is in these closed depressions that the vast majority of Iowa's peat deposits have formed.

Stratigraphically, the peat deposits begin with fine-textured mineral sediment washed from the adjacent slopes. These sediments are a result of instability shortly after the release of the glacial load. Walker (op.cit.,1),describes subsequent periods of instability during which an upper mineral silt sediment developed. His findings are supported by radiocarbon dates.

Associated with the mineral sediments of the closed depressions are accumulations of fibrous and non-fibrous organic debris.

In attempting to make a visual classification of the organic debris within Iowa's peat depressions, one notices that by far the most predominant vegetation is that of grasses and prairie growth.

In the investigations made by our soils personnel, no woody fibers have been identified and certainly no tree remains have been found. The results of our investigations would indicate that the organic debris found in depressions on the Cary Drift are accumulations from prairie environment. It is acknowledged that visual studies alone are not sufficient to substantiate this opinion. One method of study used to identify organic debris is that of pollen analysis. Lane (2) published a preliminary pollen analysis, and identifies the lower sediments as containing coniferous and deciduous pollen, with the upper sediments being dominated by pollen of prairie vegetation. Pollen, of course, would be expected to be transported by winds and ice. Therefor, even a high pollen count could be misleading as to the intensity of forest cover.

<u>International Distribution</u> -

In comparing Iowa's peat region with the overall geographical distribution of peat, a review of published reports is necessary since one cannot possibly be personally familiar with world-wide distribution. Conditions favoring peat accumulation are due to a combination of several factors, among which are - a cool, moist climate with abundant precipitation throughout the year, slow percolation and ponding conditions. A growing season must produce luxuriant plant growth to produce the organic residue needed for peat accumulation.

These conditions are met in northern Europe, Asia, and North America. Glaciated regions of these continents are described by Davis (3) as having peat deposits so large that single deposits encompass tens of square miles. Germany alone has peat deposits large enough to sustain such industries as those producing fuel, dyes, fertilizers and disinfectants, only to mention a few.

According to Davis (op.cit.3), Canada has more peat distributed across its land surface than any other country of equal size. Peat comprises approximately twelve per cent of Canada's terrain, most of it located in the northern regions. These deposits are so broad in extent that a considerable amount of research has been conducted by the Canadian Government. According to Ivan C. MacFarlane (4), current research in Canada is almost entirely of an engineering nature. Canadian research involves aerial photographic interpretation to permit route planning, field and laboratory studies to determine engineering characteristics and techniques of applying soil mechanics.

The Radforth classification of peat deposits is a broad and all-inclusive classification. It is based on a paleobotanical investigation and utilizes surface, subsurface and topographic features of the terrain. This classification apparently presents a nearly complete picture of existing conditions for Canadian peat deposits.

Classification -

A classification of peat deposits in Iowa, being used by the Iowa Highway Commission involves primarily subsurface classification. The soil survey crews of our Soils Department have investigated nearly seventy-five peat deposits in recent years. All of these sites have been classified as being located in closed depressions on the Cary Drift. The surface vegetation includes prairie grasses and herbaceous plants, unless cultivated. Where the peat areas are under cultivation, they have been tiled and are planted in corn, soya beans, potatoes or onions.

Many of the peat deposits are found within the recessional moraines that are identified within the boundary of the Cary Drift. The topographic relief in these areas will approach fifty feet or more from the bog center to the top of the peripheral slopes. The terrain is rolling with moderately steep slopes. The peat deposits found in this terrain are usually anticipated to be deep, up to twenty feet, and numerous. Associates with the morainal belts and occasional glacial knobs of sand and gravel. These sand and gravel deposits are investigated for use as backfill when peat excavation is necessary.

On the gently undulating till plain, other than the morainal belts, relief is less severe. Gentle swells will normally form the peripheral uplands enclosing the bogs described here. Many of the bogs on the till plain will have only an accumulation of black muck less than five feet in depth over glacial clay.

It is not difficult to locate numerous depressions on the Cary Drift, within or without moraines, where complete enclosure is absent, thus permitting water and sediment to find an outlet. Where this condition occurs, peat will be absent and only a highly organic topsoil will have developed.

Figure 2 is an aerial photograph depicting typical topographic feature in the morainal zone of Cerro Gordo County, Iowa. Typical peat and muck deposits are shown as closed depressions with the lighter colored glacial knobs associated with them. Minor shifts in the proposed lines of Interstate 35 and US-18 were suggested as means to assure a more desirable crossing of the peat deposits. Maximum depth of peat excavation of the major deposits will be approximately 8 feet.

In studying peat from its subsurface characteristics, the results are not as repetitious as those concerning topography and vegetative cover. The stratigraphy of the peat bogs is quite variable in depths and sequence of accumulation.

Following is the classification system used by the soil surveyors for the Iowa Highway Commission when conducting a peat investigation.

<u>Table 2</u>

Classification of Peat Sediments
in Iowa


4-20-65
17 50
US18

1. Fibrous Peat:

Fine-fibrous non-woody peat containing abundant partially decayed plant remains held in a fine fibrous framework; color will range from brown to gray, depending on degree of decay.

Engineering Properties

AASHO Classification A-7-5. Atterberg limits not normally run. Moisture content - variable (200% to 1000%).

2. Sedimentary Peat:

Fine fibrous to amorphous; Fibers, when present, are held in an amorphous matrix by organic residue; Mollusk shells may be present in abundance; minor amounts of mineral silt and clay may be present; color will be shades of green, yellow, gray and black; occasionally called humified muck.

Engineering Properties

AASHO Classification, A-7-5
Liquid Limit, 100
Moisture Content, 100% to 500%

3. Muck:

Black structureless ooze; containing no identifiable plant remains; all constituents are in advanced state of decay.

Engineering Properties

AASHO Classification, A-7-5
Liquid Limit, 100
Moisture Content, 100% to 500%

4. Semi-Organic Silt:

Amorphous mineral and organic sediment; the organic content is in advanced state of decay; often found near bottom of bogs; abundant mollusk shells may be present; color usually shades of gray, green, or yellow, may be white when calcareous (as Marl).

Engineering Properties:

AASHO Classification, A-7-6
Liquid Limit, $50\pm$
Moisture Content, 100

5. Sedimentary Silt:
Amorphous granular, primarily mineral silt and clay with variable low organic content; associated with erosion processes.

Engineering Properties

AASHO Classification	A-6
Liquid Limit	35$\pm$
Moisture Content	35

Notes: Units 1, 2, 3 above are unsuitable for embankments under nearly all conditions and usually require excavation or displacement.

Unit 4 above may be stabilized, depending on embankment height, consolidation time rate, underlying material, organic content, and should be studied from Shelby core data.

Unit 5 above will normally consolidate under loading, but when very fine textured and/or confined may require a drainage face.

Utilizing the above classification system, we can assure a close understanding between our office staff and field staff. Four conditions of peat accumulation have been prepared for this paper, which will demonstrate the sequence of deposits in Iowa's peat bogs.

Figure 3 illustrates a condition of deposition in which an early interval of instability caused erosion and subsequent deposition of mineral soil. A later, more stable interval was favorable for an equal depth of organic accumulation. Probable maximum excavation necessary in this case would be five feet, thus permitting the underlying semi-organic sediment to be consolidated by methods prescribed by the designers.

Figure 4 suggests two intervals of severe erosion permitting the deposition of the sand sediment. It is apparent that near quiescence was sufficiently long enough to permit semi-organic silts to be deposited alternately with the sand.

As in the first case, stability permits the site to be more recently filled in by organic sediment. The overlying black muck will require excavation, and it is anticipated that the sand

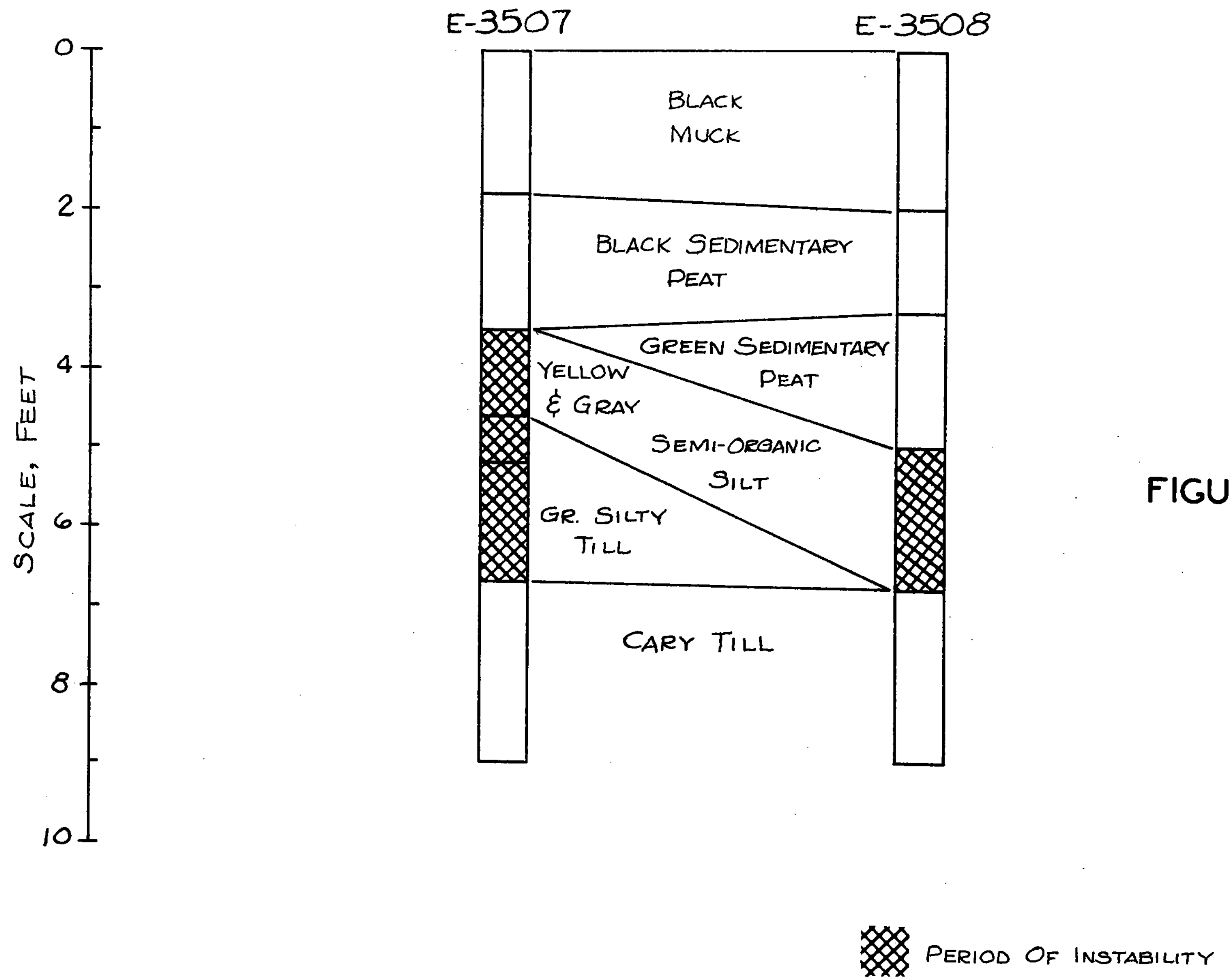


FIGURE 3

County: Hancock, Section 2 Twp: 94N Range: 23W
Stratigraphic Sequence Of Peat Bogs

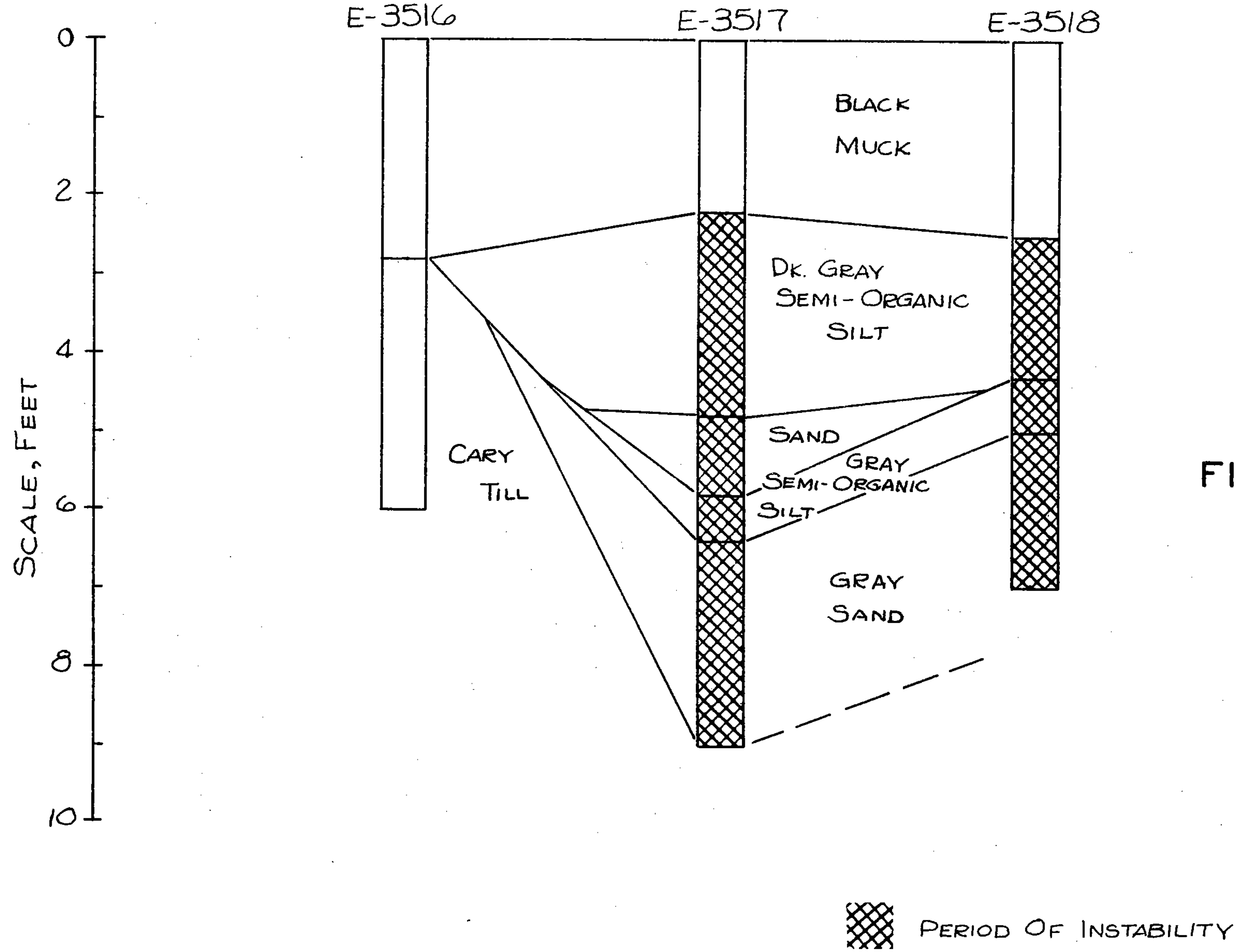


FIGURE 4

COUNTY: WORTH, SECTION: 17 TWP.: 99N RANGE: 21 W
STRATIGRAPHIC SEQUENCE OF PEAT BOGS

layers will serve as drainage faces, thus assuring consolidation of the semi-organic silts with an embankment load.

Figure 5 is typical in showing a condition favorable for abundant mineral sedimentation with a minor stable interval early in the cycles depicted by the fibrous peat layer. Again, present stability creates the surficial muck accumulation.

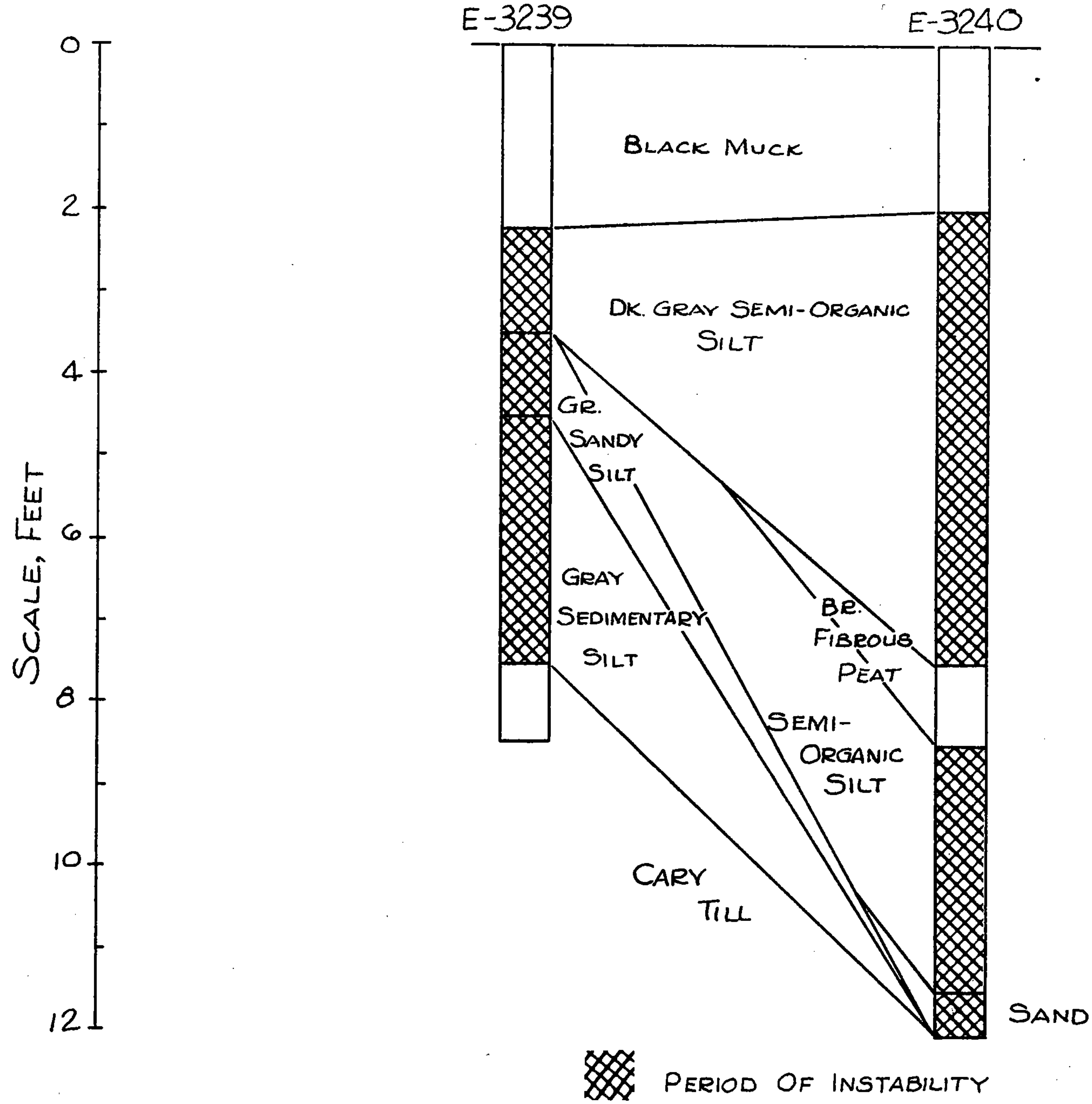
This condition may instill major headache to the Soils Engineer in his analysis for a sufficient foundation design. The black muck can economically be excavated. The semi-organic sediment can probably be consolidated by the embankment and proper controls, especially with the underlying sand as a drainage face. The problem lies with the fibrous peat accumulation. This will be difficult to consolidate within a reasonable time period, and its excavation will be costly. This would be magnified if the fibrous peat attains any real thickness.

Figure 6 represents conditions favorable for a considerable amount of fibrous peat accumulation, especially in the bog center. The upper erosion cycle was apparently short lived or not severe, and depositing only a veneer of mineral sediment at the bog center. This condition would require at least twelve feet of excavation or displacement if an embankment were placed over it. Since excavation would be below water table granular backfill would be necessary. The backfill would serve as a drainage face to assist the consolidation of the underlying semi-organic silt. This consolidation should be considered after studying the Shelby core results.

<u>Field Investigation-</u>

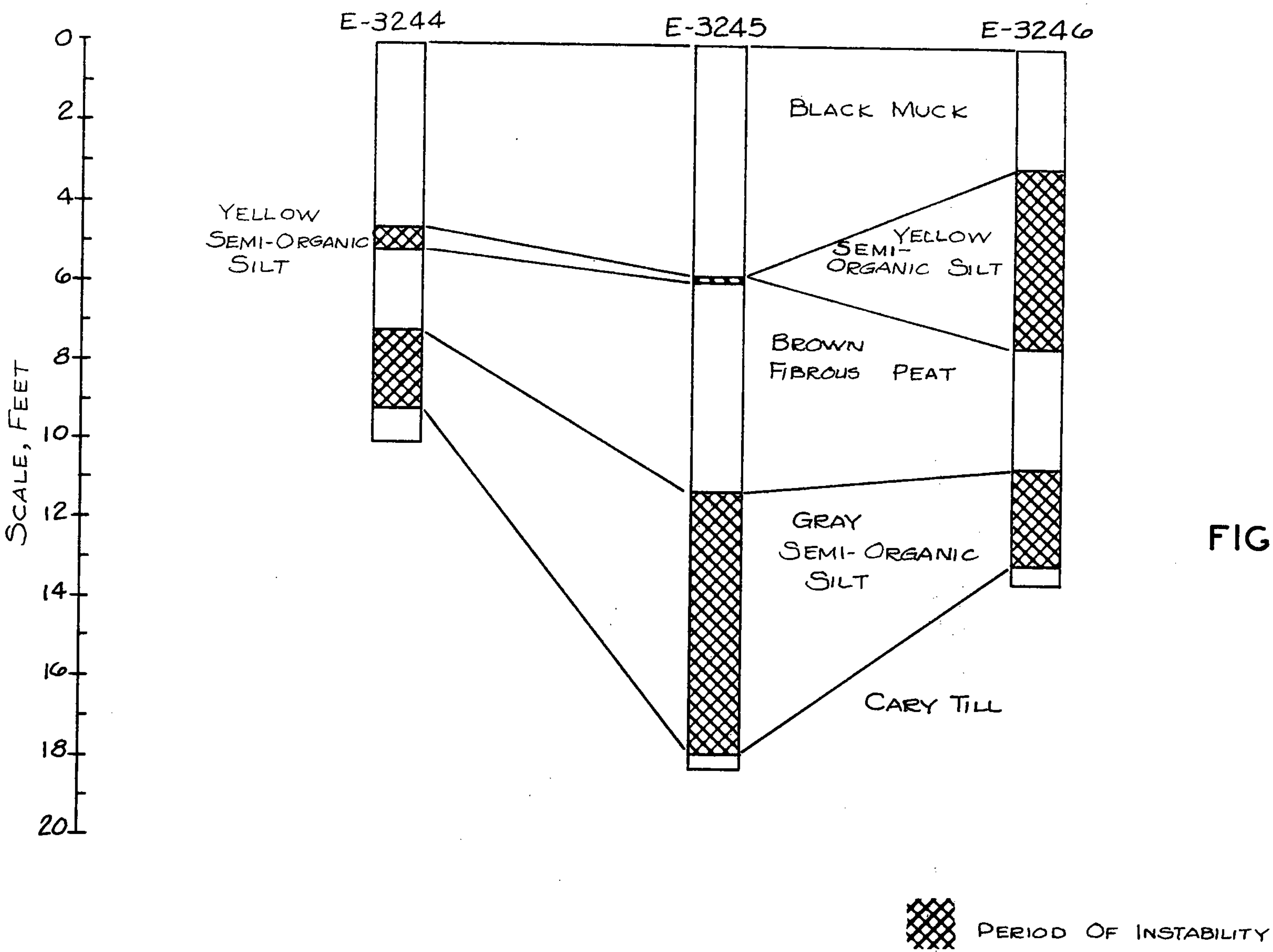
Any field investigation for soil and foundation conditions begins with an evaluation of anticipated problems while in the office. This is especially true when working with problem areas such as peat deposits. The first endeavor of our staff is to consult the county soil survey maps, which have been prepared for nearly every county in Iowa by the Soil Conservation Service. This permits the use of information that has already been gathered by others. In addition to the county maps, we have in our files, stereoscopic aerial photographs of our proposed Interstate Routes and the majority of our Primary Routes. With the interpretations gathered from these tools, we are capable of preparing for, and conducting, preliminary field exploration.

FIGURE 5



COUNTY: WRIGHT, SECTION: 16 TWP.: 90N RANGE: 23W
STRATIGRAPHIC SEQUENCE OF PEAT BOGS

FIGURE 6



County: Wright, Section: 16 Twp.: T-92N Range: R-23W
Stratigraphic Sequence Of Peat Bogs

After having gained preliminary field data, the Soils Engineer can then consider his design requirements.

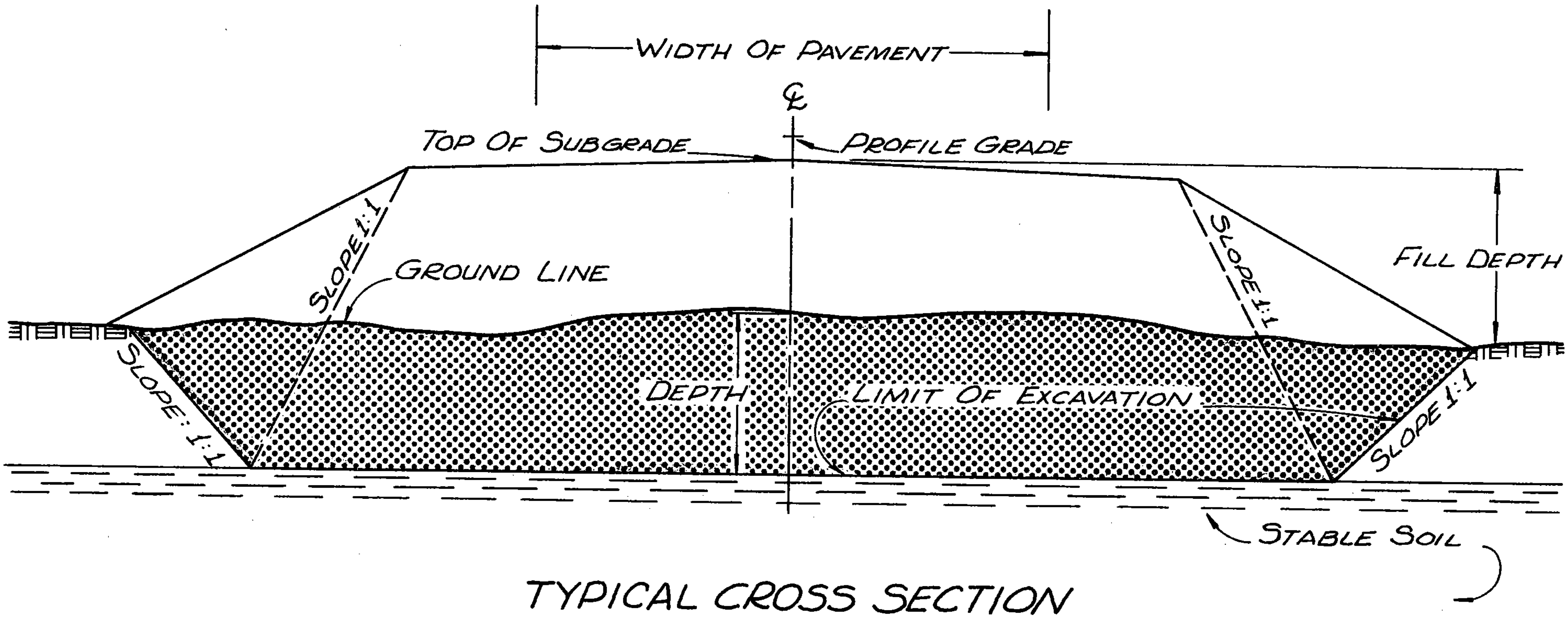
Preliminary investigation of peat deposits will be made by using a Davis Peat Sampler. This is a retractable plug samples with which samples can be taken at any depth. With experience, the operator can tell when a change of material occurs by the feel of the sampler as it is pushed into the various sediments. Visual classification is sufficient for preliminary studies. When the soil survey party is in the field making preliminary borings, a geologist will be present to assist in the classification. If a soils party chief is familiar with the classification, and has experience with the properties of peat, his judgment alone will be reliable.

After preliminary investigations have been conducted and final highway alignment has been established, a detailed field study will be performed. At this time, Shelby cores may be necessary for final design and sources of granular backfill will be investigated.

Design:

The first approach a Soils Engineer takes toward a peat deposit is one of avoidance. This is understandable when considering the cost involved in necessary treatment plus a delay in construction that may be necessary. The peat deposits in Iowa are not so extensive in area that many of them cannot be avoided by line shift. Often, line shifts may eliminate a deep peat deposit but locate us in a shallower one. When excavation is required, a waste area must be secured and granular backfill must be selected. The excavations that have been necessary by the Iowa Highway Commission have been handled by the Design Standard, Figure 7, that has been in use for several years.

The condition which can cause the most anguish is one involving the semi-organic silt. This sediment may be consolidated by several means. It may consolidate under embankment load alone, or with an overload that could be removed before paving. A sand blanket may be used to induce consolidation in a short period of time. Sand pile drains can be used in conjunction with a sand blanket to assure consolidation. Displacement by end-loading methods may be used if the sediment is very soft. All of these methods will work satisfactorily under individual conditions.



TYPICAL CROSS SECTION
EXCAVATION OF WASTE MATERIAL

WASTE MATERIAL INCLUDES PEAT AND MUCK OR OTHER MATERIAL WHICH MUST BE WASTED AND IN NO CASE USED IN CONSTRUCTION OF EMBANKMENT.

IOWA HIGHWAY COMMISSION
STANDARD ROAD PLAN
DETAILS OF EMBANKMENTS
(DISPOSAL OF UNSUITABLE MATERIAL)

FIGURE 7

The Iowa Highway Commission experienced a very definite problem in designing for peat excavation on US-69 in Wright County. Our field boring logs and test data indicated the presence of fifteen ft. of muck and peat. This material was excavated and replaced with heavy glacial till. A semi-organic silt was left in place below the peat. This semi-organic silt was eight ft. thick and would undergo consolidation.

The semi-organic silt was unloaded for seven days during excavation, and then loaded with new backfill and embankment for seventy-five days before paving.

Shortly after the pavement was in place, deformation of the pavement was noticed. It was suspected that the eight ft. of semi-organic silt was undergoing continued settlement. We obtained additional Shelby cores of this material and conducted further consolidation tests. Analysis indicated the swell during excavation to be about three inches due to unloading. This would be re-consolidated plus an additional ten inches of settlement would occur with the new load for a total of thirteen inches.

During construction, a settlement of three inches had been realized, which would leave ten inches to occur in the next eighteen months. All of the predicted settlement has taken place at the present time since more than the eighteen months has elapsed.

Conclusion:

In conclusion, it should be remembered that peat does not have to be exceptionally deep to cause concern to highway designers. Detailed studies of the individual peat deposits are necessary for highway location, and a reasonable classification should be adopted by the individuals involved. Economics will play an important part in the designers decisions for possible alternates.

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NEBRASKA GEOLOGY AND HIGHWAY ENGINEERING PROCEDURES

By

Duane A. Eversoll[1] and Ray Burchett[2]

A study of the general geology in Nebraska is necessary for use in highway engineering problems. This paper describes in general terms the soils of Recent Age and also the older bedrock as encountered either in the surface or subsurface.

Problems dealing with these different geological formations are examined in their relation to highway engineering design.

Procedures, techniques and equipment used by the Nebraska Highway Department in obtaining samples and information is discussed.

GENERAL GEOLOGY OF NEBRASKA

The surface and subsurface deposits of Nebraska are composed of thin to massive unconsolidated sediments of Recent and Pleistocene age, which are under-lain by the older sedimentary bedrock of Tertiary, Cretaceous, Permian and Pennsylvanian age. (Figure 1.)

RECENT AND PLEISTOCENE SYSTEMS

The Recent and Pleistocene Systems in Nebraska provide many problems to Highway Engineering Geologists because of the various lithologies represented and the massive layers of loess, glacial tills, sands and gravels. (Figures 3 & 4.)

Alluvium of Recent age consists of mixed river sediments of clay, silt, sand and gravel. The moisture content, irregular thicknesses, low strength and drainage create excessive settlements in the construction of roadways, embankments and bridge approaches over these deposits. For these reasons they are generally sur-charged to shorten the time of consolidation, replaced with an acceptable soil, or berms are occasionally used to achieve a satisfactory factor of safety for embankment stability.

The Pleistocene System mantles most of the State of Nebraska, (Figures 2, 4 & 5) and is divided into loess or glacial till deposits. Generally the loess is differentiated into the younger Peorian Loess consisting of yellowish-tan silt-clays overlying the older Loveland Loess consisting of dark reddish-brown silt-clays. The main problems concerned with loess in building roads in Nebraska is the differential moisture content in the transitional zone between the soil horizons. This transitional zone usually exists between the Peorian Loess and the Loveland Loess, and between the Loveland Loess and the Glacial Till. Generally it is the Peorian Loess just above the transitional zone that is most affected by the increase in moisture content. When the Peorian is subjected to a higher moisture content than its original environment it loses much of its shearing strength. It may go as low as 150 psf or less (unconfined shear strength.)

In the design of roadways, embankments, or footings of any type the possible saturation of the transitional zones must be considered. Frost boils, one of the biggest highway problems in Nebraska, sometimes occurs in these transitional zones. (Figure 6 & 7.) These frost boils affect flexible and rigid pavements as well as

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gravel roads, with damage sometimes running into the thousands of dollars each year. The Geologist with his knowledge of these formations, their locations and characteristics, provides the means for identifying and understanding them and their relationship to one another. He thus makes a major contribution to the ultimate solution of the problem, Sub-surface drainage of several types and the stabilizing of the upper more permeable zone by the addition of hydrated Lime is used to alleviate this condition. Severity of frost boil problems is mainly dependent upon weather conditions. A rainy Fall season tends to saturate the transitional zone and if this is followed in the Spring season by a prolonged freeze and thaw cycle, major damage to highways occurs.

In foundation studies for a proposed State building, where there was a fairly thin (10 to 12 feet) layer of Peorian Loess overlying the Loveland Loess, the footings had to be founded in the Loveland Loess. The Peorian at and in the transitional zone had a low shearing strength and was generally too wet to safely support the building loads economically. Whereas six to twelve inches into the Loveland Loess sufficient shearing strength to carry the proposed loads was obtained. Incidentally it was very easy during construction to get the footing to the proposed depth because the color differences between the yellowish -tan Peorian Loess and the reddish-brown of the Loveland Loess were readily recognized by construction personnel.

The Peorian Loess has the widest variation in strength characteristics of any soil encountered in Nebraska. Shearing strengths usually vary between 500 psf and 2000 psf. but samples have been tested from 50 psf (alluvial redeposited Peorian) to 5000 psf. When footings for buildings, retaining walls ... etc. are to be founded in the Peorian Loess in Nebraska it is economically advisable to take undisturbed samples at the site and to use the resulting shearing strengths instead of using the average values for Peorian Loess.

Shearing strength values for Loveland Loess also vary but through a narrower range of values than Peorian Loess. As mentioned previously the Loveland Loess is also affected by the higher moisture content in the transitional zone, but to a somewhat lesser extent than the Peorian Loess.

The Peorian Loess is also quite susceptible to water erosion on exposed slopes, and in many cases special measures are required, such as the construction of ditch checks and the placement of selected topsoil.

During the construction of fill sections in Nebraska particular care is taken in constructing the upper three feet of the embankment with a homogeneous material.

Bridge pilings are usually driven through the Peorian Loess and the Loveland Loess and tip out in the underlying materials; however, where the Loessial soils occur in massive thicknesses displacement type or friction type piling are used and do tip out in the Loess. In certain areas of Nebraska there are Loessial type soil deposits that are up to 200 feet in thickness.

Clays, silts, sands, gravels and boulders were deposited as the result of glacial advance and retreat. The extent of these glacial advances are shown in Figure 4.

It is generally known that Nebraska was affected by four major times of continental glaciations. (Figure 5.) Starting with the earliest there were the Nebraskan, Kansan, Illinoian, and the Wisconsinan. The Nebraskan invaded the area twice, resulting in a double Nebraskan. The Kansan which was probably the most extensive glaciation in the area, retreated at least partly out of the area and then readvanced over the area again. In many areas the present topography does not reflect the advance and retreat of some of the Kansan glaciations. The third glaciation, the Illinoian was probably a three fold advance and retreat of the ice. Representatives of two Illinoian glaciations are found in northeastern Nebraska; however, these only extend part way across the state. The fourth glaciation, the Wisconsinan is the youngest continental glaciation represented in Nebraska. Some of the Wisconsinan ice came into extreme northeastern Nebraska.

The Nebraska Highway Engineers and Geologists ordinarily do not try to identify these glacial tills by name. Major emphasis is put on identifying glacial tills from other types of deposits and physical engineering properties of the tills.

Glacial soils in Nebraska are predominantly high in silt-clay materials and low in granular materials. They have high cohesive and expansive characteristics. Non-uniformity prevents any selective handling in the construction of embankments as irregular lenses of very compacted silts, sands, gravels, and boulders occur in nearly all deposits of till in Nebraska. For the same reason bridge piling lengths may vary considerably from one pile group to another and sometimes even within the same pile group. Many bridge pilings in Eastern Nebraska have their tips founded in the very dense glacial tills. Displacement type piling are best suited for glacial tills although, end bearing piling are sometimes used as friction type piling due to the economics of cutting off and adding on where variations in the glacial till is anticipated.

The unconfined shearing strength of glacial tills ranges from 2000 psf to over 7000 psf with the average value approximately 3200 psf.

Pleistocene dune sands, commonly known in Nebraska as the sand hill region, cover approximately one-third of the central part of Nebraska. (See Figures 2 & 4.) Highway Engineering problems here are at a minimum due to the general stability and excellent drainage the dune sands offer. Scattered deposits of Volcanic Ash in these regions occasionally provides a source of mineral filler in bituminous construction of many sand hill highways.

One of Nebraskas biggest geological assets is the readily available Pleistocene sands and gravels found thru-out the state. The whole Platte River Valley from the eastern border to the western border is filled with sands and gravels. In some areas the thickness of these sands and gravels is over 250 feet.

The new Interstate Route 80 parallels the Platte River a total distance of over 250 miles. The sands and gravels afforded one of the best natural foundations available, and effected a lower cost per mile of roadway built. These sands and gravels also provided excellent bearing materials for bridge pilings. Probable scour depths were computed from the United States Geological Survey records and other information available to the Nebraska Department of Roads. Pilings were then designed to be driven the necessary footage below the scour depth to safely carry the bridge loads.

TERTIARY SYSTEM

The lithology of the Tertiary System consists of clays, silts, sands, gravels, and limestone. Induration of these lithologies vary greatly from one place to another creating problems of lateral extent and stripping. Most of the roadway and embankment problems in the Tertiary are easily overcome by the availability of suitable granular materials. Moderate to low cost surfacings are usually adequate in the Tertiary areas.

Bridges are founded on spread footings in the Tertiary bedrock areas whenever possible. Piling are then used where scouring or other problems will occur. We find areas where the Tertiary bedrock will be slightly cemented for five feet, well cemented for six inches or one foot, then uncemented for a foot or two, and so on for maybe 50 feet or more. It is usually found that these layers from one abutment or pier site to another are not correlative with each other. In fact the well cemented layers may extend horizontally only five to ten feet. Designing piling lengths in these areas is usually difficult. Bridge soundings are usually made at every abutment, pier or bent site where this type of condition is found to exist.

CRETACEOUS SYSTEM

Sandstones, shales, and Limestones dominate the lithology of the Cretaceous deposits. These deposits are present at depth over three-fourths of the State of Nebraska and outcrop in a thin band in easter Nebraska and along the Northern border. (Figure 1.) The shaly members of any of the Cretaceous deposits are highly detrimental in road-way construction. The alignment is carefully selected through these areas to eliminate as many engineering problems as possible. These problems include slide areas, and heavy embankments. The upper portion of these shale embankments are constructed of selected granular materials. High strength surfacing is mandatory in these areas. Whenever possible bridges in areas of Cretaceous bedrock are founded on spread footings, where spread footings are not feasible end-bearing piling are used.

PERMIAN AND PENNSYLVANIAN SYSTEMS

Permian and Pennsylvanian deposits of Limestones, shales and Sandstones outcrop in southeastern Nebraska only, but are present in the subsurface over most of Nebraska (Figure 1.) Permian Limestones are utilized for road metal but are not desirable for aggregates in concrete. Concrete aggregates are produced from the Cass Limestone Formation of Pennsylvanian age in the Ashland, Nebraska area and from either the Ervine Creek or Plattsmouth Limestones of Pennsylvanian age in the Weeping Water, Nebraska area. Most highway problems in the Permian and Pennsylvanian are limited to the Bridge sub-surface investigations, which include verification of depth to, and continuity of the bedrock layers.

ADMINISTRATION AND EQUIPMENT

Under the Department of Roads Administration Organization the Soils and Materials Survey Unit and the Embankment Foundation Unit are assigned to the Materials and Tests Division. The Bridge Sounding Unit is assigned to the Bridge Design Section which is a sub-division of the Design Division.

Soil data for the design, construction and maintenance of highways is obtained and furnished through the coordinated work of the Soils and Materials Survey Unit, the Soils Laboratory Unit and the Embankment Foundation Unit.

The Soils and Materials Survey Unit is responsible for making the field borings and taking soil samples from the excavation areas and places where high fills will be constructed along the alignment of proposed highway projects. Material pits suitable for roadway surfacing are located and sampled as near to each project as possible.

Highway locations where high fills will be constructed on questionable foundation soil and conditions are further explored by the Embankment Foundation Unit. This investigation and interpretation of the test results allows the development of information on the factor of safety for embankment stability and the amount of time of fill settlement.

The Embankment Foundation Unit runs two primary testing programs. Soil consolidation testing and tri-axial shear testing. In the soil consolidation tests, incremental loads are applied to a soil sample in a one dimensional consolidometer, periodic percentage readings are taken over a 24 hour period. Based on this data estimates of total settlements and time required in the field for these total settlements are made. In tri-axial shear testing, tests are made on representative soil samples from various sub-surface soil layers. Based on the shear strength of these samples a safety factor for embankment stability is computed by use of I.B.M. computors. The Embankment Foundation Unit runs all of the unconfined and confined shear tests on Shelby samples that are sent in by the Bridge Sounding Unit.

The Soils Laboratory Unit performs routine physical tests on samples submitted from preliminary soil and subgrade surveys, for the purpose of classifying the soils and determining their suitability for use in highway construction.

After receiving the test results, the Soils and Materials Survey Unit prepares a Soil and Situation Report with recommendations for the construction and compaction of the highway embankments. These reports are submitted to the Design and Construction Divisions. Sketches of the more desirable material pits are drawn for the plans and furnished to the Right of Way Division for optioning.

The Archeological and Paleontological salvage program which is a cooperative effort by the Historical Society, University of Nebraska, and the Soils and Materials Unit of the Nebraska Department of Roads, has salvaged many items of historical and fossil value.

The Bridge Sounding Unit makes subsurface explorations for bridges, retaining walls, some of the larger culverts, Department of Roads buildings, and shelters and restrooms at the new rest area sites thru-out Nebraska. A preliminary drilling program to help determine the availability of water at the proposed rest areas is also made by this unit. Standard ASTM penetration tests, thin walled or Shelby tube tests, and visual examinations of augered soil samples are all utilized by the Bridge Soundings Unit in their testing program. A cooperation program between the Bridge Soundings Unit and the Nebraska Geological Survey is maintained. Records and samples are exchanged for useful information such as maps, reports, and personal communications. A strengthening of both programs and the savings of much time and money is the end result of this exchange.

The Soils and Materials Unit makes an extensive use of the agricultural Soil maps which are published by the Conservation and Survey Division of the University of Nebraska and the U. S. Department of Agriculture. It has often been said that this group of maps is one of the Departments most valuable aids for soil survey and materials prospecting work. A tremendous saving in time and money has been effected in the past 25 years through the daily use of these maps.

The most common uses of the agricultural soil maps are: Estimating costs of flexible pavements for programming purposes, locating slope stability problems, Estimating the required thickness of flexible pavements, Estimating the granular foundation course requirements for rigid pavements, Estimating the need for a clay blanket on gravel surfaced roads, Planning and conducting the soil survey, Locating deposits of materials for highway construction, and Estimating drainage and runoff characteristics for drainage structure design. These maps in Nebraska must not be treated as being infallible but rather used as a guide for the Highway Geologist in his work in soils.

The embankment foundation unit uses a B-52 Mobile Drill for their subsurface investigations. This drill is operated by two to four men with a Senior Engineer in charge over all work. Undisturbed sampling with 2½ inch O.D. Shelby tubes are used in conjunction with 2-3/4 inch I.D. by 7 inch O.D. hollow-stem-augers. Six inch augers are also used but mainly for exploratory information.

The Soils and Materials Survey Unit has seven B-40 Mobile Drills that are used for their soil and sub-grade surveys and materials investigations. Each B-40 is manned by a soil survey Party Chief and his helper. Their work is controlled by a senior geologist and his two supervisors, one an agronomist the other a civil engineer. The B-40's are equipped with six inch augers. Most samples obtained in the field are either bagged or sealed in separate containers for shipment back to the soils laboratory for further tests.

The Bridge Soundings Unit has two B-52 Mobile Drills and a Ka-Mo electric drill that are used for making bridge soundings and foundation investigations for miscellaneous state buildings. Each B-52 is manned by a Drilling Foreman, an operator and two helpers. The Ka-Mo drill is used by either of the two B-52 crews when needed. A geologist supervises over both crews. The B-52's are equipped with six inch augers and the 2-3/4 inch I.D. by 7 inch O.D. hollow-stem-augers. The six inch augers are used mainly for exploratory information as the first hole drilled on the site, or at sites where hollow-stem-augers are not suitable. Hollow-stem-augers are used in approximately 75 percent of all holes drilled. The penetration tests make up the bulk of information obtained by the Bridge Soundings Unit. The undisturbed Shelby tests are also used as a supplement to the penetration tests at bridge sites. In building and retaining wall foundation studies undisturbed samples are then preferred over the penetration tests.

CONCLUSION

It is becoming increasingly more important for the soils Geologist to know and fully understand the geological problems, properties and environments of the soils that he uses in the design and construction of highways. With the continuing high cost of right-of-way it is evident that highways in the near future will be aligned on less desirable soils than in the past, therefore the soils geologist is going to have to keep pace with the increasing problems of these less desirable soils, in their relationship to highway engineering.

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ACKNOWLEDGEMENTS

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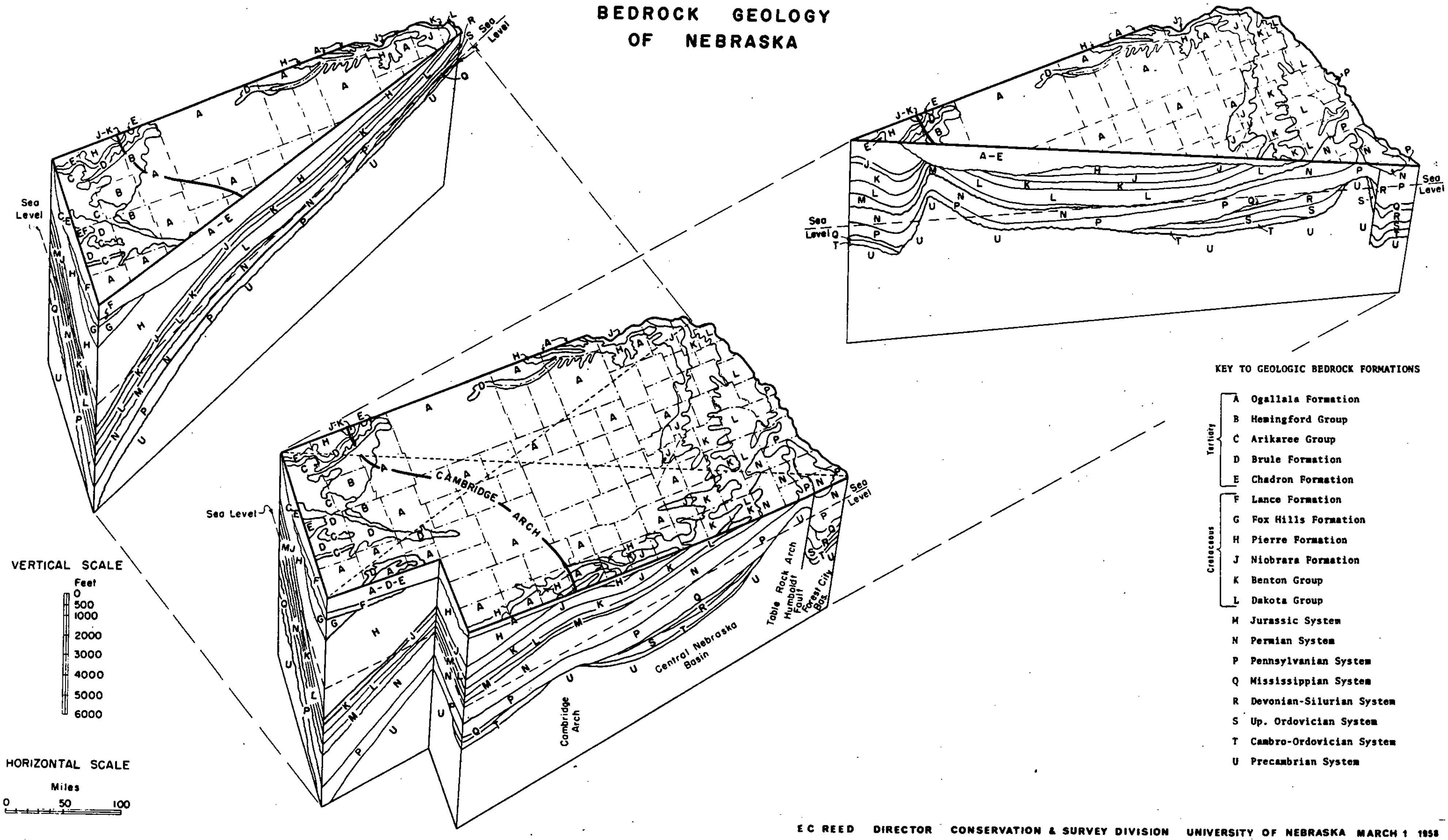
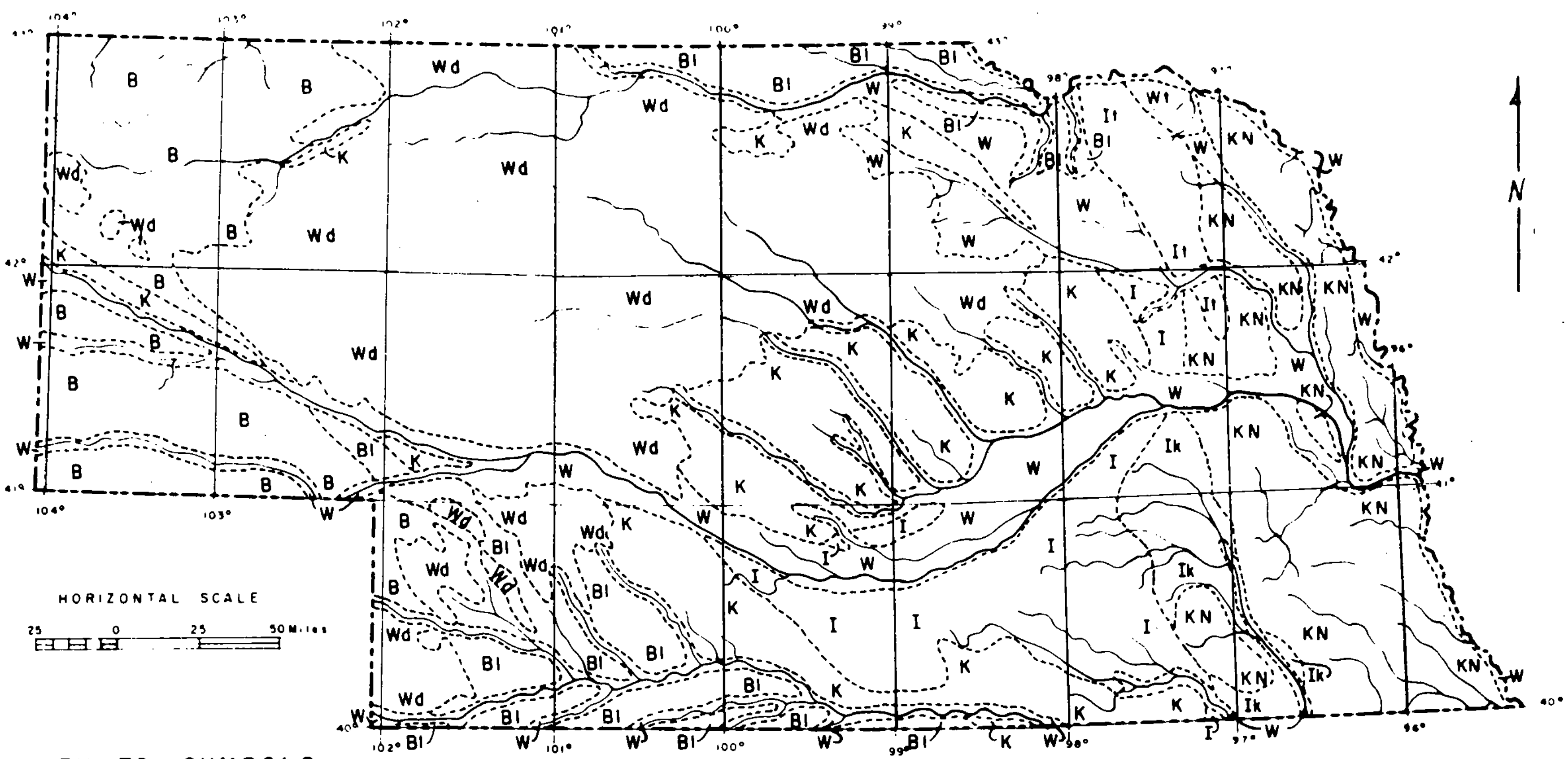

BEDROCK GEOLOGY
OF NEBRASKA
KEY TO GEOLOGIC BEDROCK FORMATIONS
Tertiary
A Ogallala Formation
B Hemingford Group
C Arikaree Group
D Brule Formation
E Chadron Formation
Cretaceous
F Lance Formation
G Fox Hills Formation
H Pierre Formation
J Niobrara Formation
K Benton Group
L Dakota Group
M Jurassic System
N Permian System
P Pennsylvanian System
Q Mississippian System
R Devonian-Silurian System
S Up. Ordovician System
T Cambro-Ordovician System
U Precambrian System
VERTICAL SCALE
Feet
0
500
1000
2000
3000
4000
5000
6000
HORIZONTAL SCALE
Miles
0
50
100
Sea Level
CAMBRIDGE ARCH
Cambridge Arch
Central Nebraska Basin
Table Rock Arch
Humboldt Fault
Forest City Bas.
E C REED DIRECTOR CONSERVATION & SURVEY DIVISION UNIVERSITY OF NEBRASKA MARCH 1 1958

Figure 1.



KEY TO SYMBOLS

W - Wisconsinan terrace deposits (Bignell and Peoria Formations), loess-mantled; also includes Recent alluvium. Wd - Wisconsinan dunesand (Bignell and Peoria Dunesand). Wt - Medial Wisconsinan till (Hartington Till), generally mantled with Late Wisconsinan loess and terrace deposits (Bignell). I - Illinoian terrace deposits (Loveland, Beaver Creek and Grafton Formations), loess-mantled; overlies remnants of Kansan terrace deposits. Ik - Illinoian and Kansan terrace deposits (loess-mantled) above remnants of Early Kansan Till (Nickerson). It - Illinoian Till (Santee and Clarkson Tills), loess-mantled; overlies remnants of older till. KN - Kansan till (Cedar Bluffs and Nickerson Tills), loess-mantled; overlies remnants of Nebraskan tills (Iowa Point and Elk Creek Tills) locally. K - Kansan (Sappa, Walnut Creek and Red Cloud Formations) and Nebraskan (Fullerton and Seward Formations) terrace deposits, loess-mantled. Bl - Loess-mantled bedrock; terrace deposits in channels. B - Bedrock; some thin loess mantle.

Figure 2. Generalized Pleistocene map of Nebraska.

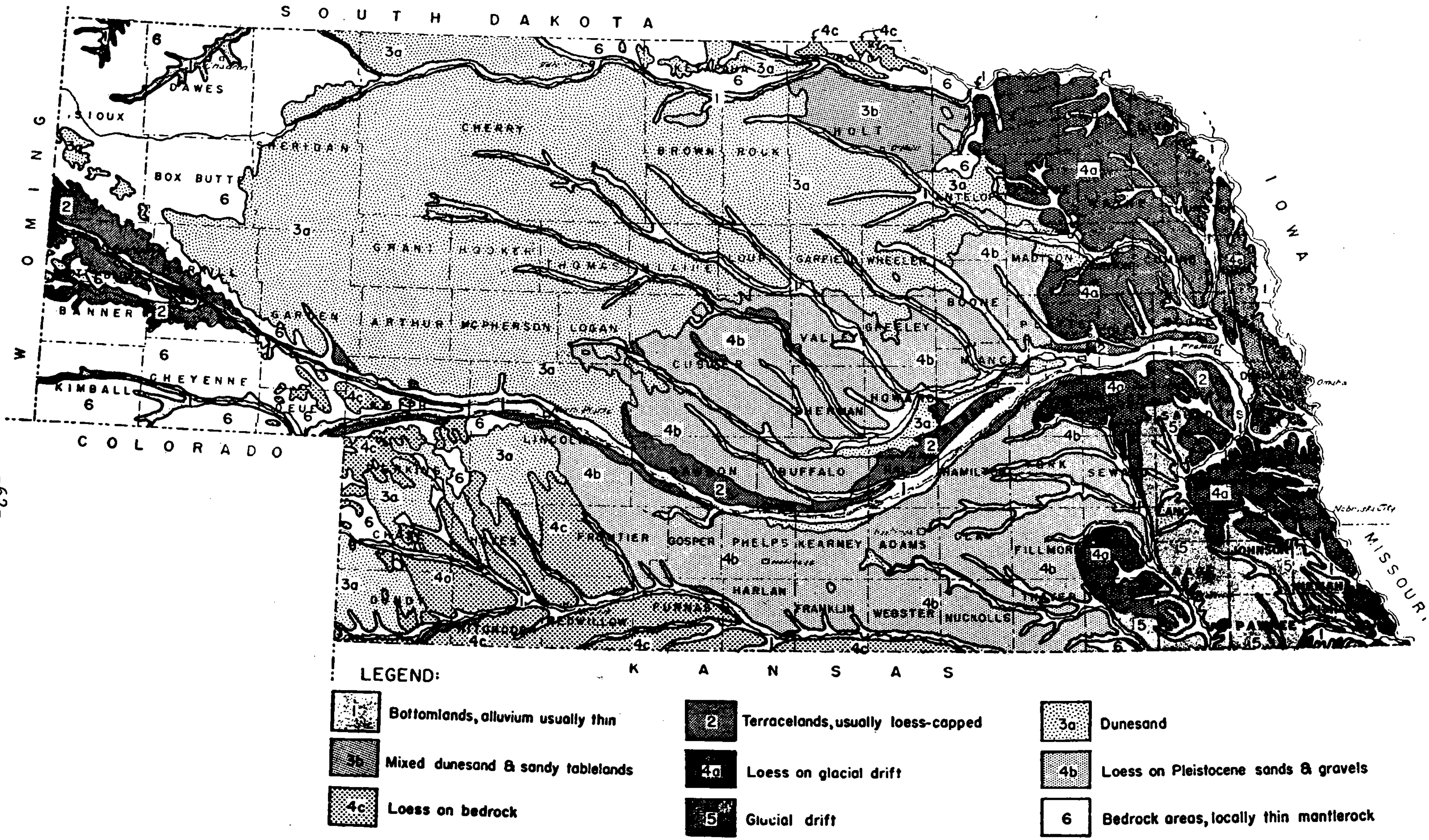


Figure 3.—Generalized Areal Mantlerock Map of Nebraska. Note that the Loess-on-glacial-drift areas include some inliers of glacial drift and the glacial drift ares includes small outliers of Loess in the higher topography. The scale of the map does not permit us to show narrow belts of terracelands or bedrock along some valley sides.

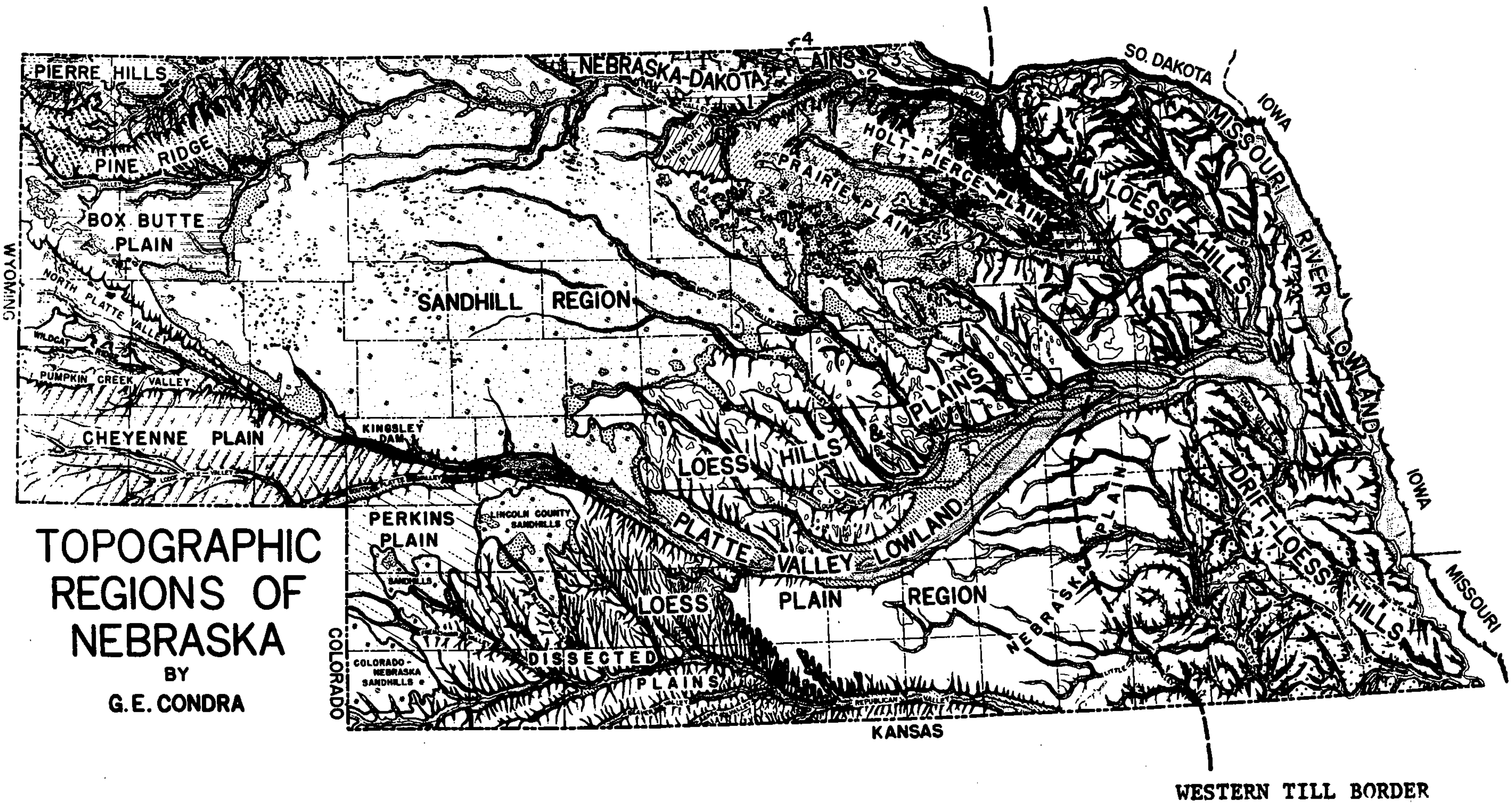

TOPOGRAPHIC REGIONS OF NEBRASKA
BY
G.E. CONDRA
PIERRE HILLS
PINE RIDGE
BOX BUTTE PLAIN
NORTH PLATTE VALLEY
WILDCAT
PUMPKIN CREEK VALLEY
CHEYENNE PLAIN
KINGSLEY DAM
SANDHILL REGION
NEBRASKA-DAKOTA PLAINS
AINSWORTH PLAIN
HOLT-PIERCE PLAIN
PRAIRIE PLAINS
LOESS HILLS
LOESS HILLS & PLAINS
PLATTE VALLEY LOWLAND
PERKINS PLAIN
LINCOLN COUNTY SANDHILLS
LOESS PLAIN REGION
DISSECTED PLAINS
COLORADO-NEBRASKA SANDHILLS
NEBRASKA PLAIN
DRIFT-LOESS HILLS
MISSOURI RIVER LOWLAND
SO. DAKOTA
IOWA
MISSOURI
KANSAS
COLORADO
WYOMING
WESTERN TILL BORDER

Figure 4

Time-Stratigraphic Units			Rock Stratigraphic Units	
		12	Glacial Deposits	Periglacial Deposits
	Late		No till	Bignell loess and alluvium
		11		
Wiscon-sinan	Medial		Hartington till	Peoria loess, silt and dunesand Todd Valley sand and dunesand
		10		
	Early		No till	Gilman Canyon loess and alluvium
		9		
	Late		No till	Loveland loess and silt Crete sand and gravel
		8		
Illinoian	Medial		No till	Beaver Creek loess and alluvium
		7		
	Early		Santee till pro-Santee sand & Gravel	Grafton loess and alluvium
		6		
	Late		Clarkson till	Sappa loess & silt (Pearlette volc. ash) Grand Island sand and gravel
		5		
Kansan	Medial		Cedar Bluffs till	Walnut Creek loess and alluvium
		4		
	Early		Nickerson till Atchison sand and Gravel	Red Cloud alluvium
		3		
	Late		Iowa Point till	Fullerton loess and silt Holdrege sand and gravel
Nebraskan		2		
	Early		Elk Creek till David City sand & gravel	Seward silt and loess David City sand and gravel
		1		

Significant Events:

1. Development of Tertiary climax soils; major pro-glacial erosion of the Pennsylvanian, Permian, Cretaceous and Tertiary Bedrock.
2. Development of interstadial soil; erosion.
3. Development of Afton interglacial soil; major pro-glacial erosion.
4. Development of Fontanelle interstadial soil; minor erosion.
5. Development of interstadial soil; minor erosion.
6. Development of Yarmouth interglacial soil; major pro-glacial erosion.
7. Development of interstadial soil, minor erosion.
8. Development of interstadial soil; minor erosion
9. Development of Sangamon interglacial soil; major pro-glacial erosion.
10. Development ofFarmdale soil; minor erosion.
11. Development of Brady soil; minor erosion.
12. Development of Soil; Recent erosion and deposition

E.C.Reed
V.H.Dresszen
Nebraska Geological Survey 1965

Figure 5 Classification of Pleistocene Deposits in Nebraska

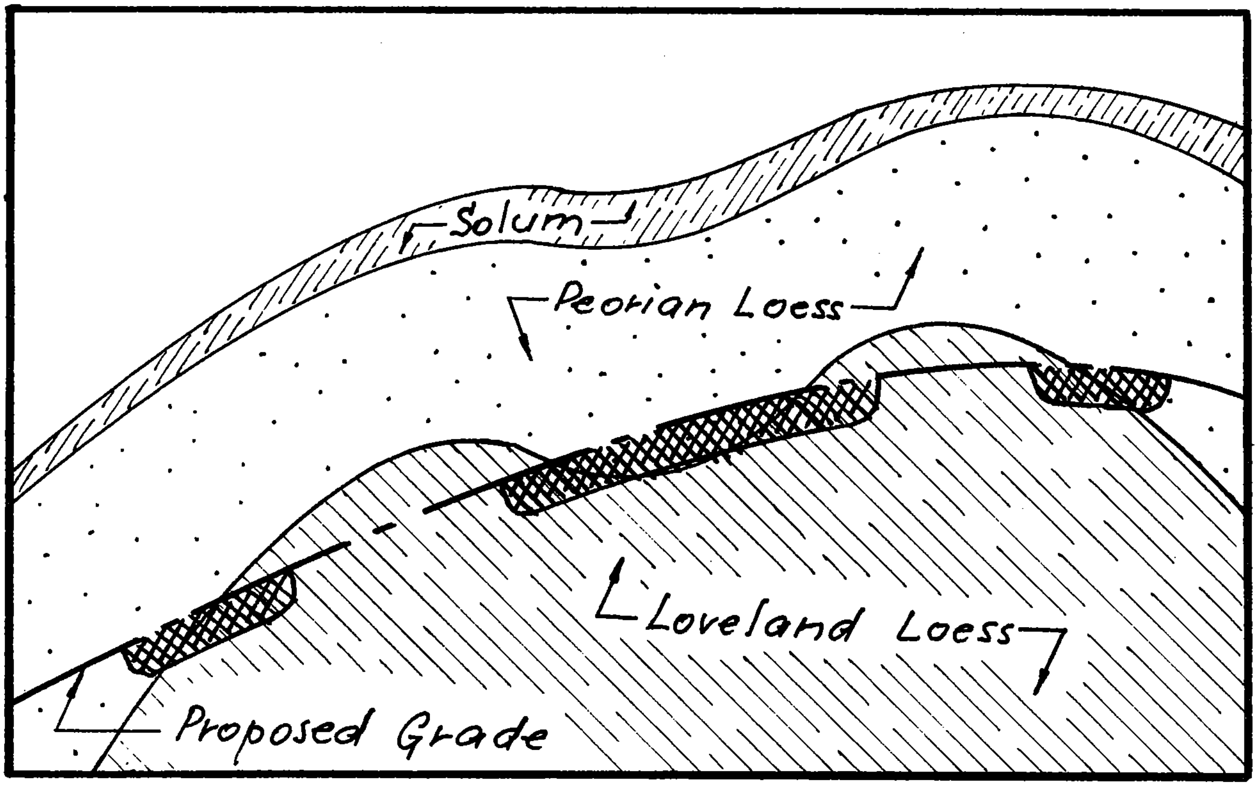


Figure 6: Potential areas for frost boils in a Peorian Loess and Loveland Loess highway cut.

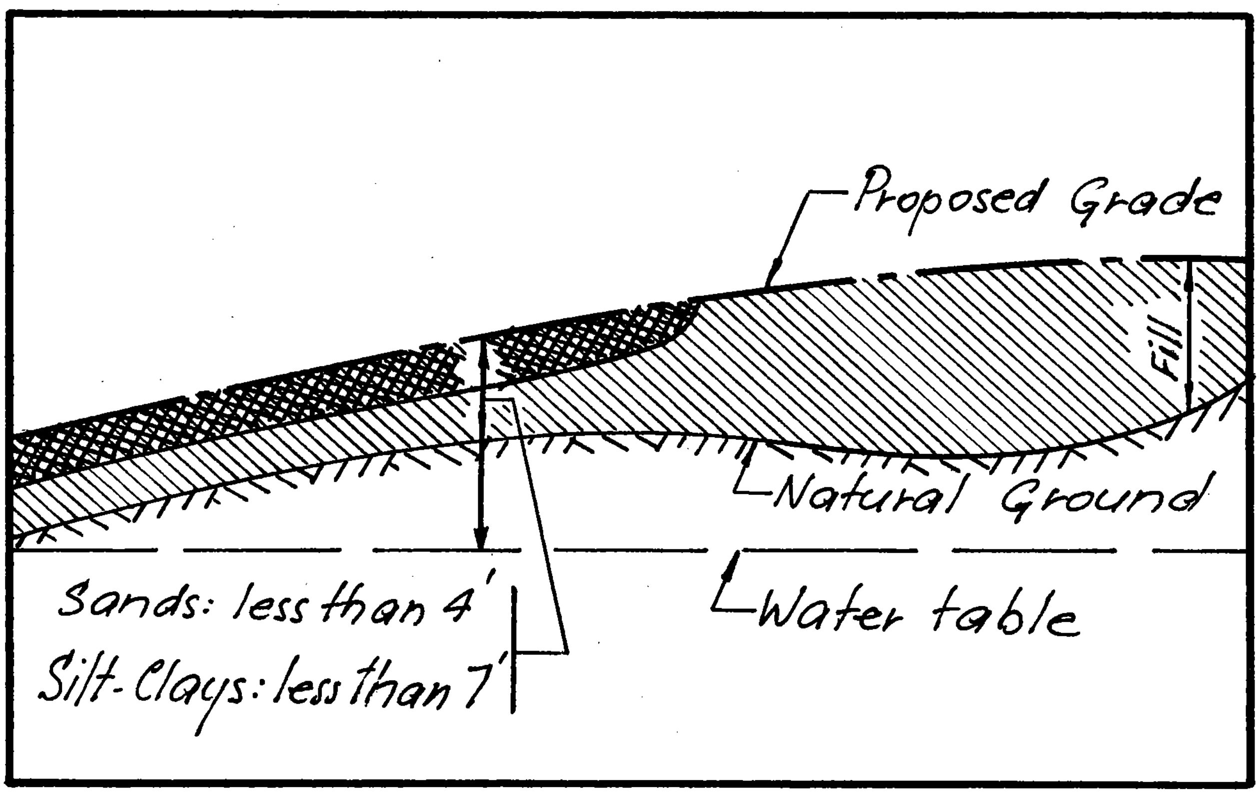


Figure 7: Another potential area for frost boils, involving a high water table.

LANDSLIDES IN THE PIERRE SHALE OF SOUTH DAKOTA

by
Richard L. Bruce
South Dakota State Geological Survey

INTRODUCTION

South Dakota, in the past, has been considered an area of minor landslide activity (Baker and Chieruzzi, 1959, p. 10-11). New highway construction in the State has illustrated that, although the landslides are not of catastrophic types, they create a major constructional and thereby a financial problem.

Recognizing the increasing problem created by landslides and the economic effect they have on maintenance and construction, the South Dakota Geological Survey, through funds provided by the South Dakota Department of Highways and the U. S. Bureau of Public Roads, initiated a program to study landslide activity in the State.

The purpose of this study is to identify those rock formations in South Dakota which are most susceptible to landslides, and to prepare a manual for the Highway Department that discusses those formations and points out the conditions under which they will slide. The manual is to include a guide to the recognition of potentially unstable areas, and suggests some methods for the prevention and correction of landslides.

The majority of landslide activity in South Dakota occurs in the Pierre Shale. Because of the increase in construction in the outcrop area of the Pierre, it has received the major emphasis in this study.

Landslides in the Pierre Shale can be attributed to a combination of many factors acting through time. Varnes (1958, p. 42-45) presented a summary of the factors which may lead to landslides either by increasing the shearing stresses acting on a slope or by decreasing shearing resistance of the slope-forming material. Many of these same factors are now acting on slopes in the Pierre Shale, and are known to effect slope stability. The only factor, however, that is common to all slopes developed in the Pierre Shale, is the character of the material.

CHARACTER OF THE PIERRE SHALE

The late Cretaceous Pierre Shale crops out in an area of 20,000 square miles in South Dakota (fig. 1). Exposures of 200-300 feet are not uncommon along the valley walls of streams entrenched into its outcrop area.

To the untrained eye, the Pierre Shale is a relatively homogeneous formation. It has been divided, however, into eight units based on exposures in the Missouri River trench in central South Dakota (Crandell, 1950). Erskine (1965) has shown that landslides occur in all units of the Pierre, and depending upon various environmental factors, all units may become unstable.

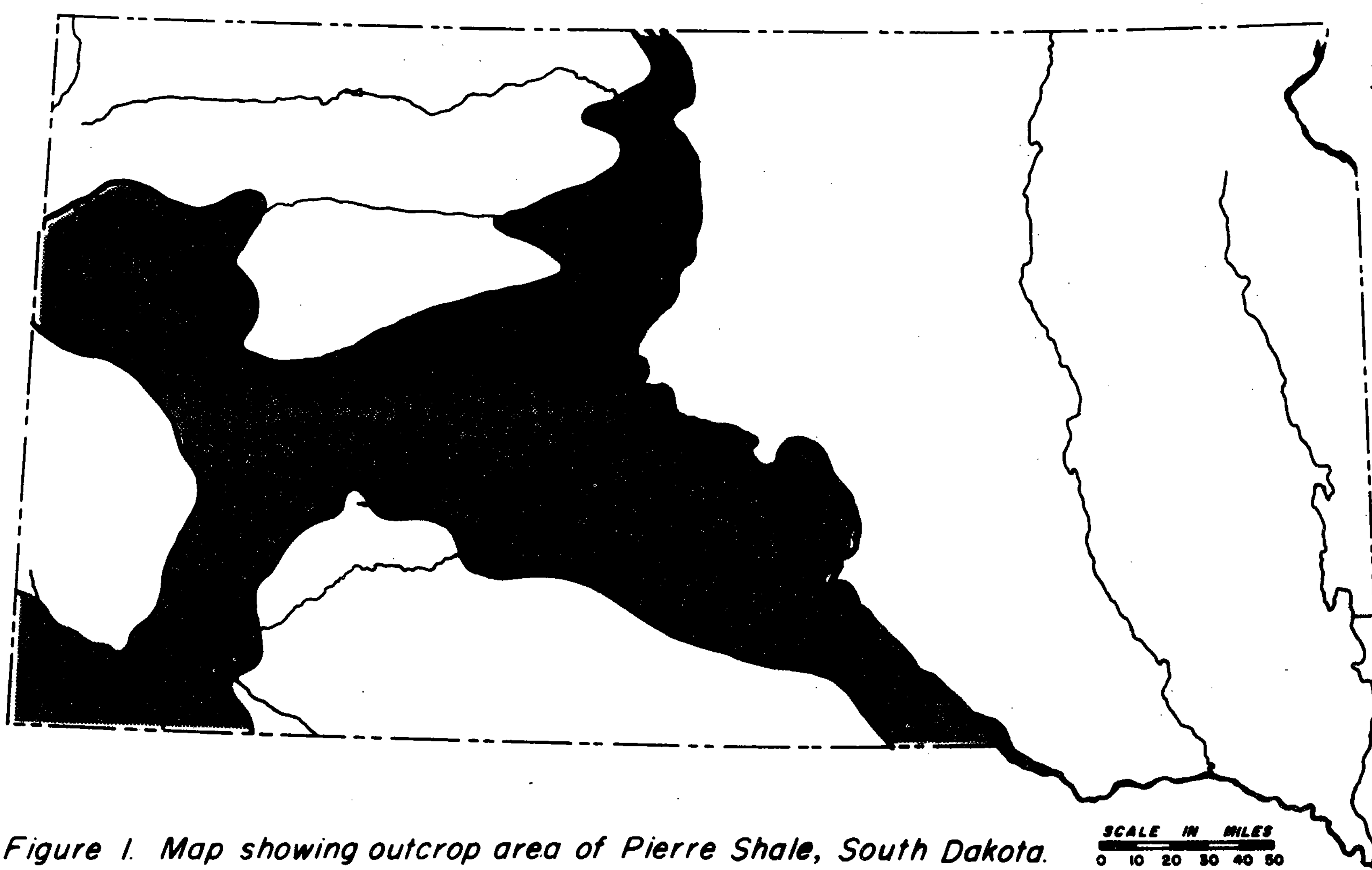


Figure 1. Map showing outcrop area of Pierre Shale, South Dakota.

Lithology

The Pierre Shale is a black-to-gray clay, clay shale, and shale which contains altered volcanic ash (bentonite) disseminated throughout the formation and/or concentrated into one-to-six inch beds. Layers of carbonate concretions are scattered throughout the formation. Samples of this shale collected by Tourtelot (1962) from exposures along the Missouri River were found to contain an average of 70 percent material finer than 4 microns and over 50 percent finer than 1 micron. Clay minerals comprise 70 to 80 percent of the shale, with montmorillonite and mixed-layer illite-montmorillonite comprising up to 80 percent of this fraction.

Undisturbed and unweathered shale contains brittle zones which usually contain cracks (fractures, joints) that occur normal to the shale layers. With the addition of water and with exposure to weathering, this brittle material expands and becomes plastic, forming what is locally referred to as "gumbo."

The fissility of the Pierre Shale varies. For example, certain zones in the shale are highly fissile, while others lack fissility and exhibit a decided "block structure."

The Pierre Shale, therefore, reveals three-dimensional defects and these not only control permeability and weathering, but also exert great influence on the stress distribution in a slope. These defects are herein called discontinuities. These discontinuities may have either a primary or secondary origin.

Primary Discontinuities

Primary discontinuities characterize the shale and are contemporaneous with the deposition and diagenesis of the Pierre sediments. These are:

Bentonite beds
Concretion zones
Brittle zones
Plains of fissility
Lithologic changes

Figure 2 is a diagram showing the origin of the Pierre Shale and of the primary discontinuities in the shale. Sediments that make up the Pierre were derived from several sources. Deposition and diagenesis of these sediments occurred in a changing physio-chemical environment. In this variable environment, changes in sediment composition took place as the original clays were mixed with volcanic ash. Subsequent compaction aided in the formation of bentonite beds, concretion zones, brittle zones, and fissility. The final result of these depositional and diagenetic processes is the Pierre Shale.

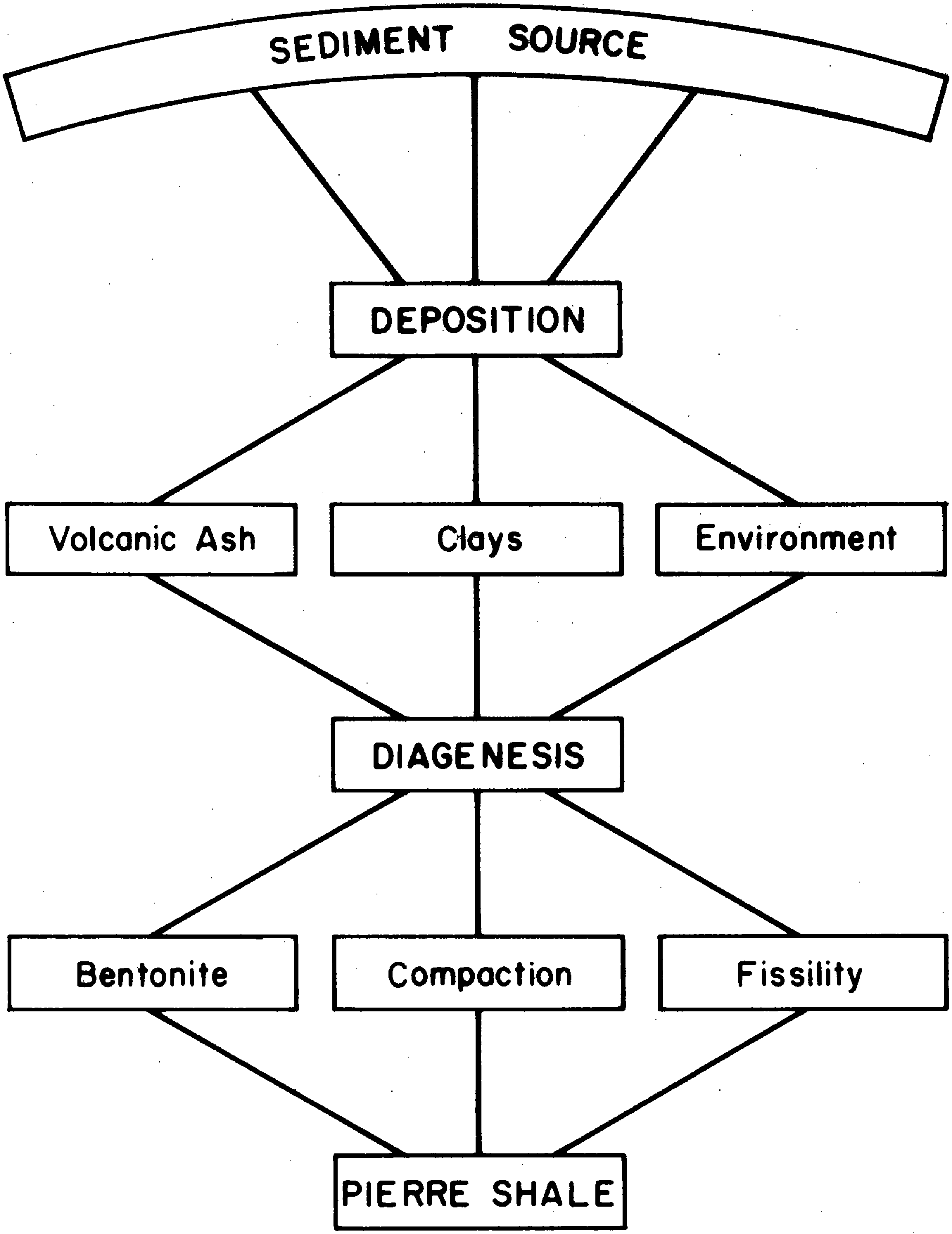


Figure 2. Diagram showing origin of Pierre Shale and primary discontinuities.

Secondary Discontinuities

Secondary discontinuities are formed insitu in the shale by the action of outside forces. These discontinuities are cracks which may be fractures or joints.

Figure 3 is a flowchart showing the formation in nature of secondary discontinuities in the Pierre. The shale, on exposure, is subjected to the degradational processes of weathering, mass-wasting, and erosion. These processes are contemporaneous and continuous and relieve confining pressures on the shale by unloading. This unloading causes differential rebound and continuous creep to take place. Because the more brittle zones in the shale are unable to adjust to these movements, cracks form. These cracks act as permeable zones in an otherwise impermeable shale. Any water movement through these cracks causes a progressive deterioration or alteration of the structure of the shale.

Excavation of a slope during construction could also act as the unloading process; the result, although on a smaller scale, would be identical.

ROLE OF DISCONTINUITIES

Primary and secondary discontinuities in the Pierre Shale control water migration. Differential rebound and creep, caused by unloading, form cracks in the more brittle zones in the shale. The amount of cracking is dependent on the rate and amount of unloading. These cracks act as zones of downward percolation of water. Weak zones, such as bentonite layers, plains of fissility, and concretion zones, control lateral movement of water through the shale.

When these discontinuities become interconnected, they form the avenues of water migration. Ground water, therefore, moves through narrow and restricted zones in the shale. Reaction of this water with the shale causes expansion of the clay minerals and progressively decreases the shearing resistance of the material. Hydraulic pressures, that are built up by this invading water, increase shearing stresses acting on the slope. Some of these stresses are released when the discontinuities intersect the surface of the slope, and seeps develop. When this release is not present, these stresses can become critical and ultimate slope failure occurs. Slope failure creates an outlet for the confined water, and the slope soon reaches equilibrium. If the hydraulic pressures again build up faster than they can be relieved by seepage, the slope soon becomes unstable again.

Current studies of four slopes in the Pierre Shale seem to support these conclusions. Continuous cores from these slopes are being subjected to exhaustive laboratory tests to provide detailed data on the characteristics of the shale. Field observations using Slope Indicator wells, porous tube piezometers, and water observation wells are providing data on slope movements, and water pressures and movement.

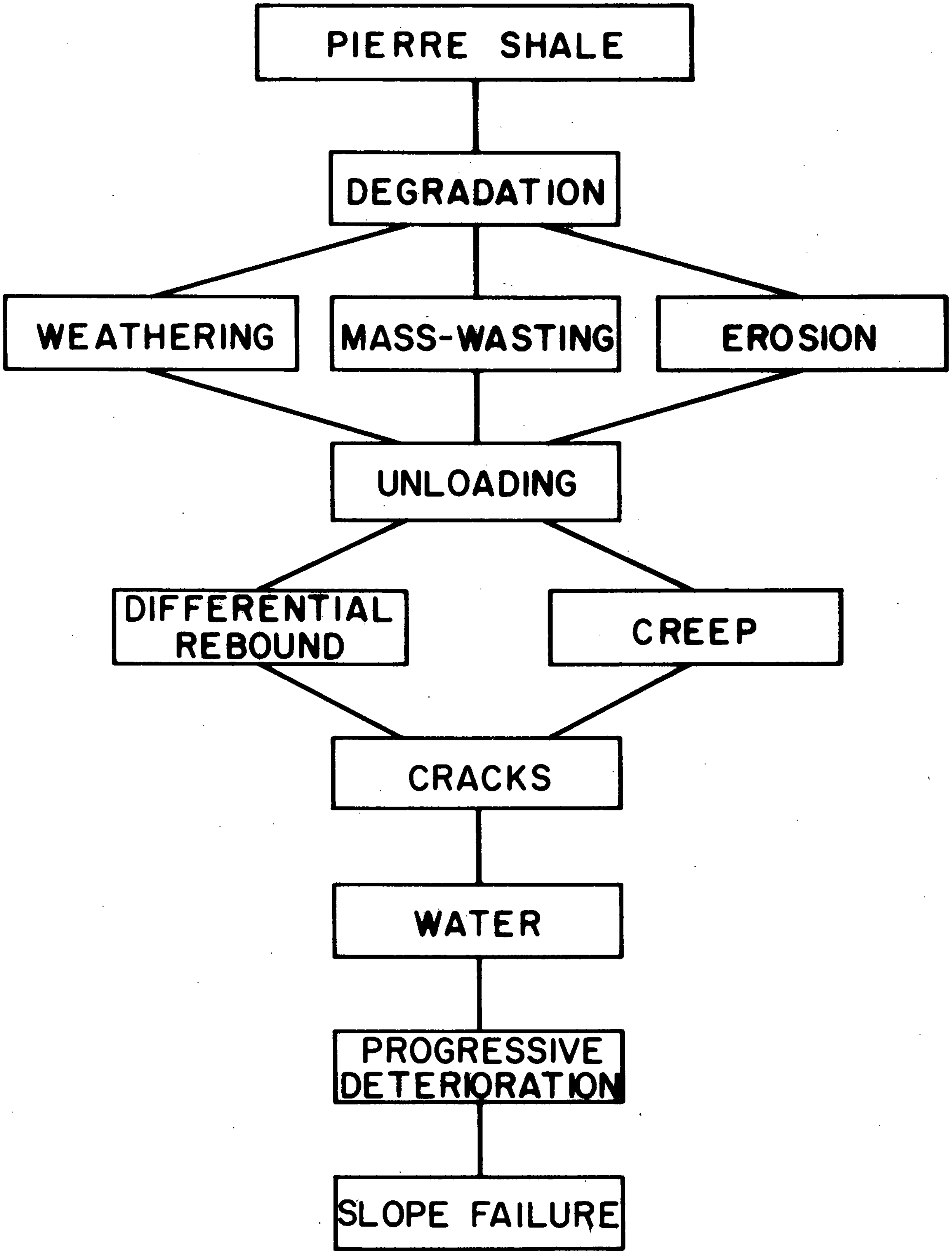


Figure 3. Diagram showing relationship of cracks to slope failure.

Two induced recharge wells are presently being constructed in an active landslide in the Pierre Shale. Chemical tracers will outline the zones of water migration. This slide will then be excavated and these zones will be exposed, providing firsthand observation of the role of discontinuities in water migration.

CONCLUSIONS AND APPLICATIONS

Landslides are not controlled by geologic boundaries in the Pierre Shale, but are controlled by the topographic environment of the slope and the concentration of discontinuities in the slope. These discontinuities are avenues of water migration and contribute to the buildup of hydraulic pressures and the progressive deterioration of the structure of the shale material.

If the hydraulic pressures in a slope can be reduced, the slope can be held in equilibrium. In nature, the formation of seeps in the slopes releases the pressure. Pressure reduction can be undertaken artificially by the construction of release wells (horizontal or vertical drains) in the slope. The proper location of these wells in the narrow zone of water movement in the slope is critical. The major problem facing us in our study is to devise an accurate method of locating these zones of water movement so that efficient installation of wells can be accomplished.

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A STUDY OF CHERT AGGREGATE REACTIVITY BASED ON OBSERVATIONS OF CHERT MORPHOLOGIES USING ELECTRON OPTICAL TECHNIQUES

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Abstract

Porous, spongy, intermediate, polygonal and crystallite morphologies of chert were observed by electron optical techniques and were compared with routine engineering laboratory tests. The chert morphologies represent possible stages of devitrification and polymerization of fine-grained silica. A change of surface area and crystallinity occurs with grain growth and influences the physical and chemical behavior of the chert.

The degree of devitrification and polymerization is a function of the following geological factors: (1) time; (2) depth of burial; (3) tectonics; and (4) diagensis. If the service record of a chert aggregate is to be predetermined, it is therefore necessary to study in detail the geologic occurrence of the chert.

Porous, spongy, and crystalline cherts exhibited (1) the least freeze-thaw-durability; (2) the highest solubility; and (3) the greatest expansion due to alkali-chemical reactivity. The intermediate and polygonal chert morphologies showed the lowest total porosity and smaller pore densities thereby exhibiting better cryothermal durability and lower chemical reactivity. Polygonal cherts are most susceptible to failure by thermal shock. These stresses are related to microfractures that inordinately expand due to changes in temperatures.

INTRODUCTION

Many sources of concrete aggregates have been viewed with suspicion because they contain considerable amounts of chert that possesses undesirable thermal and chemical properties. It is the scope of this study to relate some of these harmful properties to the electron optical surface morphologies of chert.

Twenty-two varieties of chert from ten (10) different localities in Ohio representing four (4) depositional realms were investigated. These realms include: (1) glaciofluvial gravels; (2) recent river gravels; (3) middle-Devonian marine carbonates and (4) several dominantly clastic Pennsylvanian sections.

TECHNIQUES

Electron Microscopy

Freshly fractured chert surfaces were replicated by a two-stage process utilizing a biodin film technique. The peal was fixed by a vaporizing carbon coating at 1 x 10^{-5} torr vacuum to stabilize the specimen under the electron beam. In addition an 80% Pt and 20% pd alloy was vaporized onto the peal at a coating angle of 45° - 60° degrees. Simultaneously, Chromium was deposited opposed to the Platinum-Palladium at a coating angle of 15° - 20° degrees creating a backshadow and increasing the three dimensional effect on the replicated surface. The grid mounted specimens were then cleaned in a soxhlet extractor after the technique of Beals and Bigelow, (1962). Several hundred grids were prepared in this manner, and over 500 electron micrographs were taken in order to define what surface morphologies are present as well as what surface morphologies are most characteristic of any given specimen.

Porosimetry

A 15,000 psi mercury penetration-type porosimeter manufactured by the American Instrument Co. of Silver Springs, Maryland, was employed in determining porosity and pore-size distributions. Pore-size distribution measurements utilized the techniques of Drake (1949) and Drake and Ritter (1945).

Solubility Determinations

The solubility determinations used the D. U. Spectrophotometer and the Bunting (1944) molybdenum blue method for determining the concentrations of silica. Two hundred and fifty milliliters of NaOH solution with varying pH ranging from 8 to 12 were prepared for each of the twenty-two samples investigated. Twenty-five grams of 100-mesh chert were then added to each of the solutions. The solutions were then continuously agitated on rollers at 24 rpm for over eleven months.

Routine Engineering Laboratory Tests

Mortar bar expansion tests: The ASTM C-227-58T mortar bar expansion test was modified by autoclaving chert-bearing 12"x 1"x 1" mortar bars under slowly rising temperature to 390°F at about 200 psi. The temperature and pressure were maintained for a period of 5 hours, after which the power was shut off and the sealed autoclave was permitted to cool for 15 hours. The chamber was then opened and bars were stored at a constant temperature (73°F) and constant relative humidity (53%) for 9 hours. The bars were then measured and weighed again. This modification of ASTM C-227-58T was deemed necessary in order to shorten the normal 3 to 6 month test period.

Freeze-Thaw-tests: Quick-freeze-thaw tests followed ASTM C-290 except for a reduced prism size (3"x 4"x 4"). These tests were performed with both low and high alkali cements and with aggregate contents of 1%, 10% and 20% chert. Blast furnace slag was used as the inert aggregate. Figure 1 illustrates an example of the freeze-thaw failure caused by the presence of chert aggregate in concrete.

Mineralogy

The term chert is a misnomer that has been applied to fine-grained silica. A considerable range of compositions exists in rocks that have been described as chert, jasper, or flint. Their composition may be entirely opaline, chalcedonic, quartzitic or an admixture of each. Likewise the texture and degree of crystallinity can also occur in any combination. Impure cherts commonly grade into siliceous limestone. This gradation adds to the confusion concerning what can be classified as a chert.

TYPES OF FAILURE

Certain cherts whenever incorporated in concrete are known to possess non-desirable thermal and chemical properties. The inability of chert to withstand thermal shock above the freezing temperature of water is an example of such a property. Cryothermal reactions in chert may cause disintegration of concrete due to repeated cycles of freeze and thaw. Thermal properties are significant because small temperature changes well within the heat of hydration of the cement, or within the diurnal variation of pavement temperatures, may cause inordinate expansion with resultant fissuring of concrete. Freezing of entrained water in chert may cause excessive physical failure in concrete. Some cherts react chemically with alkalies that are liberated during the hydration of the cement. This reaction is known as alkali-aggregate reactivity, the end result of which is the cracking and deterioration of concrete presumably due to osmotic pressures produced by the formation and hydration of silica gels formed by the interaction between a susceptible chert and the alkalies liberated by cement during hydration.

RELATION OF MINERALOGY & MORPHOLOGIES TO FAILURE

INTRODUCTION

Porous, spongy, intermediate, polygonal, and crystallitic surface morphologies of chert were observed by electron optical techniques and were compared with laboratory tests. It is proposed that these morphologies represent different stages of devitrification or polymerization of fine-grained silica, ranging from the gel state to the cryptocrystalline state. It is further assumed that with this grain growth sequence, the surface area and crystallinity vary and influence the physical and chemical behavior of the chert. Taylor (1959, 1960) lends support to this concept by his discussion of the effect of crystal imperfection and the degree of crystallinity on the rates of chemical reactivity.

The degree of devitrification or polymerization of a substance is a function of time, temperature and pressure, therefore it is necessary in order to cogently evaluate the desirability of any chert aggregate to study the following geological factors involved in the origina of chert: (1) geological time; (2) depth of burial; (3) tectonics; and (4) diagenesis.

The following figures and data exhibit the relation of chert mineralogy and surface morphology to the physical and chemical reactivities that cause failure in concrete.

Fig. 1. Chert particles in 3 x 4 x 8 inch concrete prism causing failure after 60-90 cycles of rapid freeze and thaw.

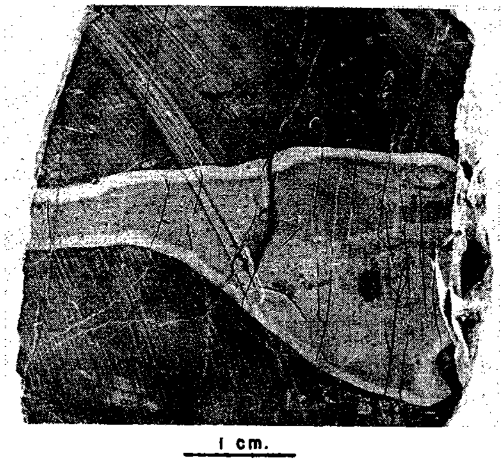


Fig. 2. Fractures in limestone and chert nodule showing relative anisotropy to the propagation of elastic waves generated by a quarry blast.

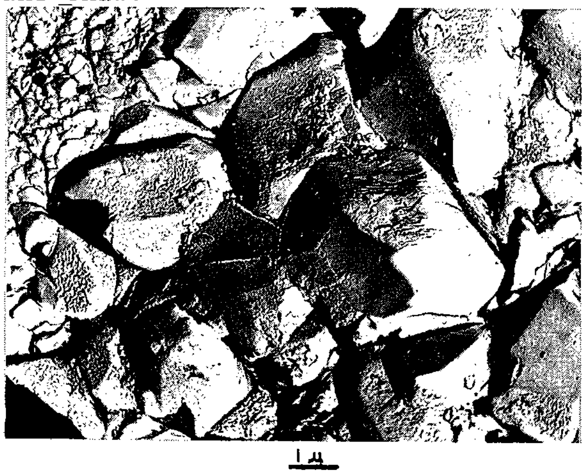


Fig. 3. Nucleation and growth of quartz crystallites along a microfracture in a chert.

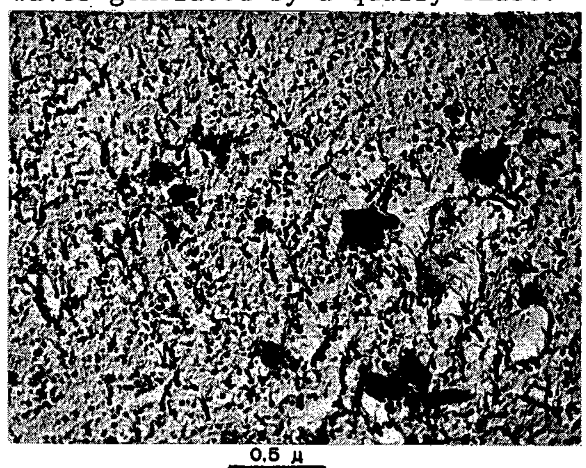


Fig. 4. Surface morphology of a silica gel (SMR-55-9112*). Average pore diameter is about 0.0025 microns. Photo by Hixon (1964, p. 75).

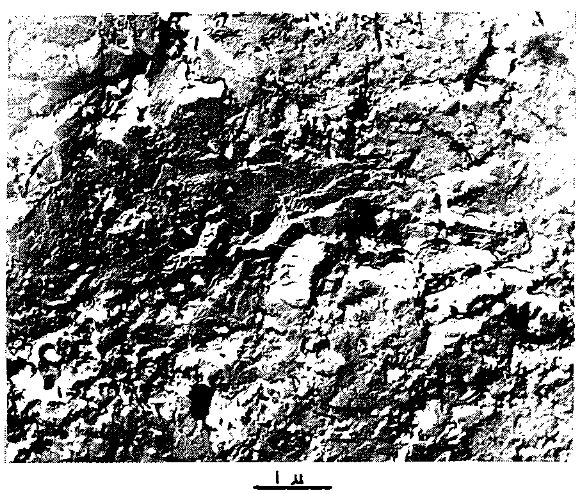


Fig. 5. Spongy surface morphology of chalcedonic chert.

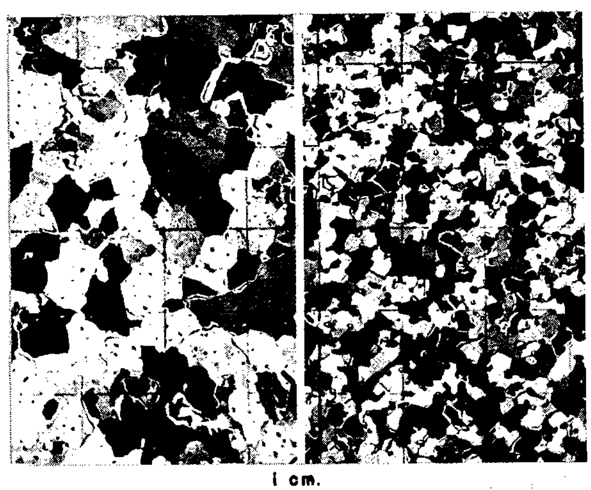


Fig. 6. Ice sections from depth of 71 (right) and 300 meters (left). Tiny blebs are entrained air. Note the decrease in bleb density with increase grain size. Photo by Robin (1962,p.3).

Observe in figure 2 the fractures in the limestone and the chert nodule. Note that there are more fractures in the chert than the limestone. This illustrates with some fidelity the anisotropic nature of chert when subjected to elastic waves generated by a quarry blast. By analogy the chert responds in the same manner to thermal shock. When the chert was heated in a laboratory oven to 80°C and then air quenched, the chert spalled in the same manner as the nodule in the limestone. Electron optical inspection of the spalled surfaces (Fig. 3) showed that they occurred along micro-fractures which were partially or wholly cemented by megaquartz or "crystallitic" chert.

Figure 4 illustrates a surface morphology of a typical gel. Differential thermal analysis of this gel revealed that the gel possessed a large amount of absorbed water which was driven off at about 158°C and that it possessed no crystallinity. It is suggested that these "primary" pores housed the water driven off during the heat. The water is also purged out of the system towards the end of the devitrification or polymerization sequence, because of grain growth and the formation of crystallized silica.

In the 24 September 1965 issue of Science (v. 149, p. 1501-1503) Peterson and von der Borch reported that chert is precipitating as a gelatinous-opalcristobalite in saline-alkaline lakes of Coorong Lagoon, South Australia. They stated that the depositional lake environment alternated between a pH (9.4 to 10.2) and pH (7.0 to 6.5). This alternation respectively causes dissolution of detrital silica and subsequent precipitation. This recent report gives compelling support to the devitrification or polymerization concept presented in this paper.

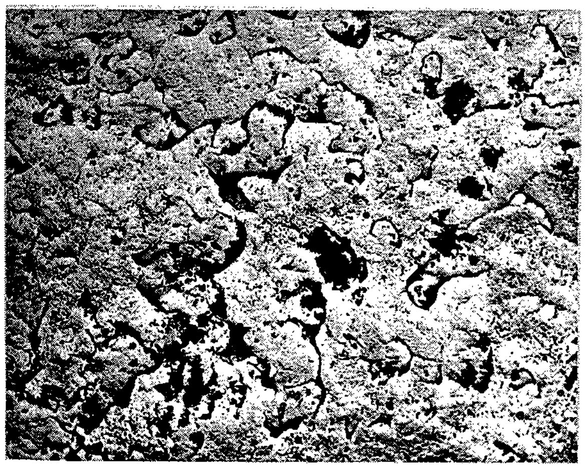
Spongy morphology is illustrated in figure 5. The term, spongy morphology was introduced by Folk and Weaver (1952) and supported by Iwao (1953), Bates (1958), Pittman (1959), and Monroe (1964). Chalcedony exhibits this type of morphology. To this date no electron optical studies have revealed the fibrous structure so characteristic of a petrographic section of chalcedony.

Figure 6 illustrates the recrystallization of ice upon increase burial. Note the grain growth and attendant expulsion of air bubbles. By analogy one could apply this process to chert as the chert passes thru the devitrification or polymerization sequence.

Transitional state in the devitrification sequence to the intermediate chert is exhibited by figure 7. The grain boundaries are starting to develop as the pores density decreases.

Figure 8 shows an intermediate chert morphology. Observe the general spherical pores which are about 0.04 microns in diameter and are concentrated along the irregular chert boundaries. These pores according to Hixon (1964) contain water. Note the similarity of this micrograph to figure 6 which illustrates recrystallization of ice upon increase burial.

Polygonal surface morphology is illustrated in figure 9. It is assumed that this polygonal morphology represents a transition stage in the devitrification sequence between the intermediate and crystallite states. However, this polygonal morphology may be the effect of a natural solvent

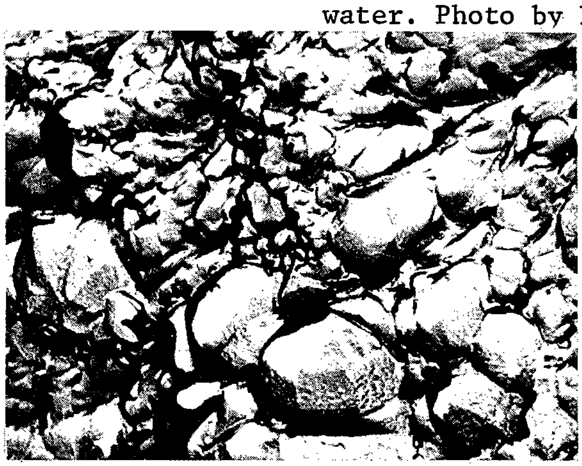


1 μ

Fig. 7. Early development stage of intermediate chert surface morphology, associated with depolymerization and subsequent loss of water.

0.5 μ

Fig. 8. Intermediate chert morphology. The pores (ca. 0.04 microns) are concentrating along developing grain boundaries and contain some water. Photo by Hixon (1964, p. 47).

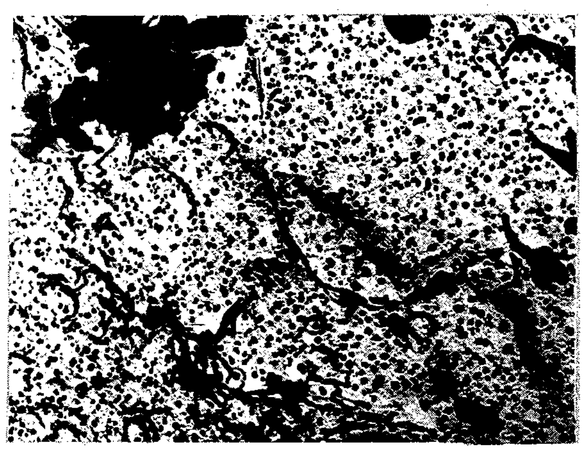


1 μ

Fig. 9. Polygonal chert surface morphology.

1 μ

Fig. 10. Chert crystallite morphology. The end-product of the grain growth sequence (devitrification).



0.5 μ

Fig. 11. Porous surface morphology of white, punky, and highly weathered chert.

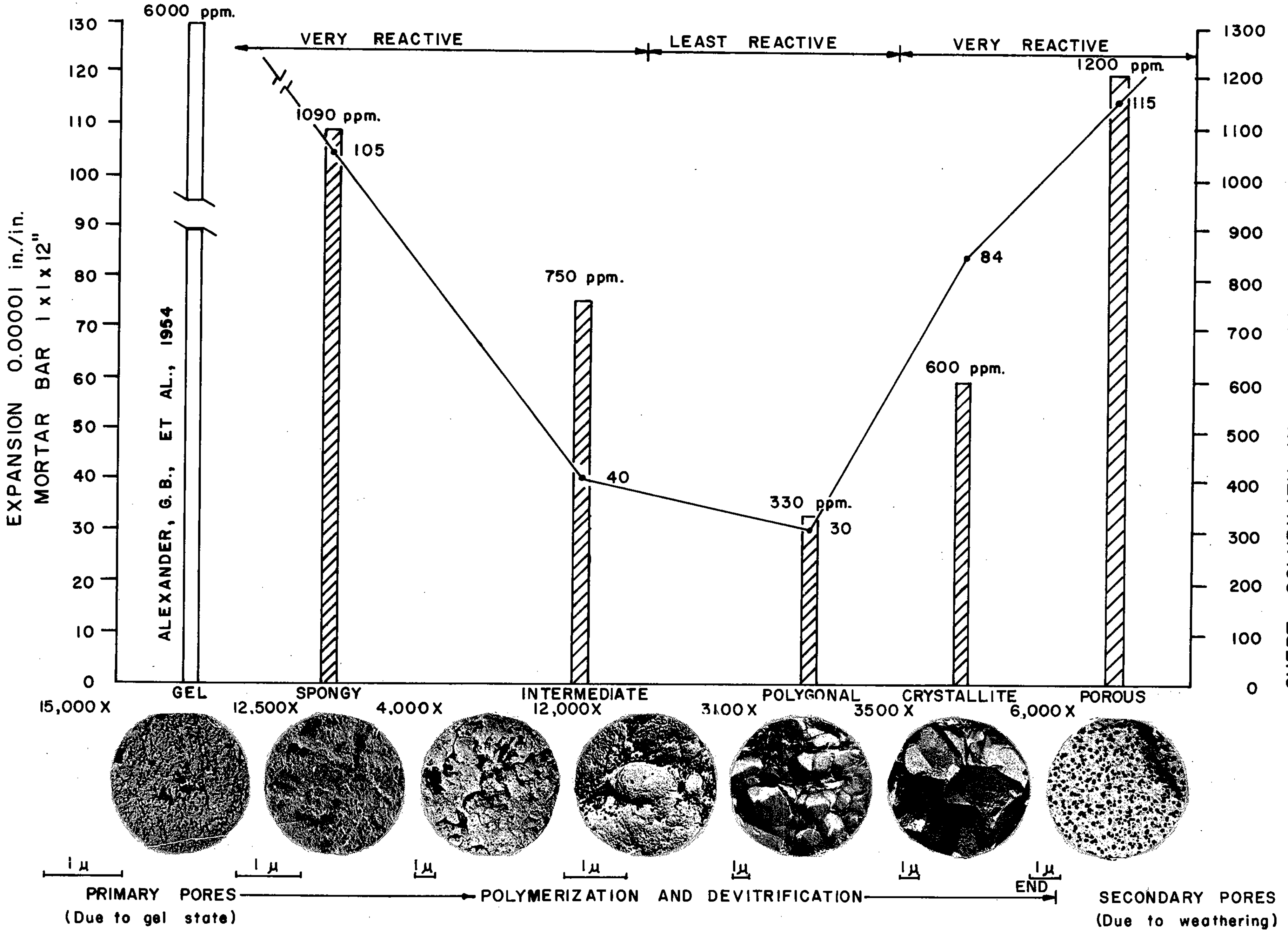


Fig. 12. ELECTRON OPTICAL SURFACE MORPHOLOGIES OF CHERT

TABLE 1

PORE DATA ON ELECTRON OPTICAL

MORPHOLOGIES OF CHERT

TYPE SURFACE MORPHOLOGY	SPECFIC GRAVITY	AVERAGE PORE SIZE IN MICRONS	PORE DENSITIES PORES/μ^2	AVERAGE TOTAL POROSITY
Porous (secondary)	2.10 - 2.40	0.50	300-400	20%
Crystallite	2.40 - 2.55	0.22	Void Space Primarily Interstitial	10%
Polygonal	2.55 - 2.65	0.14	Void Space Primarily Interstitial	4%
Intermediate	2.45 - 2.60	0.46	40 - 100	6%
Spongy (primary)	2.35 - 2.45	0.1 - 0.0015	400-1000	12%
Gel (SMR-55-9112*)	1.9 est	0.003	800-2400	Variable, but Usually high

(weathering). It is known that polygonal hillocks can be formed by the action of a solvent on a crystal face because the rate of dissolution varies with crystal direction. (U.S.Army Signal Corps, 1942, p. 52), (Hale and Hurlbut, Jr., 1949, p. 597), and (Buckley, 1951, p. 304-338). For example, a polished sphere of quartz when placed in a solvent is attacked at very different rates along different radii. As the sphere dissolves, it gradually assumes a rounded polyhedral shape. The distribution of faces on the polyhedron conforms to the symmetry of the quartz.

Figure 10 exhibits the typical crystallite morphology, the end product of the grain growth sequence.

Figure 11 illustrates the porous surface morphology of a white punky highly weathered chert. This type of secondary morphology can be readily distinguished from the primary porous morphology by the larger average pore diameter as well as the lower pore density.

The data exhibited by figure 12 and table 1 give compelling support to the concepts presented herein. Study these illustrations and observe that surface area decreases from the gel morphology to the polygonal morphology where the packing density of particles is the greatest and the total porosity is the lowest. The surface area and porosity again increase from the polygonal to the crystallitic and secondary porous morphologies. The void spaces in the polygonal and crystallite states are not due to cellular pores but to voids between the interstices of the grains. The degree of chemical reactivity as illustrated by the mortar-bar test expansion values and solubilities can be explained by surface area, packing density and dissolution along different crystal directions.

SUMMARY

Porous, spongy, and crystallitic cherts exhibited (1) the least freeze-thaw durability; (2) the highest solubility; and (3) the greatest expansion due to alkali-chemical reactivity, whereas the intermediate and polygonal chert morphologies showed the lowest total porosity and smaller pore densities and subsequently they exhibited better cryothermal durability and lower chemical reactivity. Polygonal cherts, however, are most susceptible to failure by thermal shock. These stresses are related to microfractures that inordinably expand due to changes in temperatures.

Laboratory tests demonstrate that cherts rich in chalcedonic silica (spongy morphology) exhibit alkali-aggregate reactivity. Mortar-bar tests made with crushed chert and high-alkali cement (2.5% Na_2O) showed harmful expansions due to reactivity whenever the proportion of chert, regardless of surface morphology, exceeded 10 per cent.

Comparison of geological occurrence, petrography, geochemistry, electron microscopy, x-ray diffraction, differential thermal analyses and the origin of the cherts studied reveal many differences. The most important differences are the degree of structural disorder, and crystallinity, surface morphology, specific gravity, pore-size distribution and total porosity, grain packing density, fibrous silica content, and degree of weathering and microfracturing.

The correlation of highway laboratory tests with techniques of electron microscopy and x-ray diffractometry shows great promise in predicting the service records of cherty aggregates.

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ENGINEERING GEOPHYSICS: ITS USE AND ABUSE

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This paper is concerned only with the two most popular tools and/or techniques used in Engineering Geophysics -- refraction seismic and electrical earth resistivity.

It is our contention that these tools can be of inestimable value to the Highway Geologist and Engineer but as with all tools knowing how not to use them may be as important as knowing when to use them. We will look at a few of the most troublesome areas and misconceptions which occur too frequently, then briefly survey the background and finally suggest ways to prevent mistakes.

To Highway Geologists and Engineers, the Earth Electrical Resistivity Measurement method is perhaps the best known and older of these two means of investigating the earth's shallow sub-surface. With modern equipment, earth resistivity measurements can be made very rapidly over any given area and the operator comes up with some very valuable and useful information.

All such Electrical Reistivity - ER - equipment (Figure 1), whether this high-powered, solid-state electronic Model ER-2 or the classic Gish-Rooney unit, is responsive to -- and measures -- variations in the electrical resistance of the various earth materials encountered in the traverse.

This resistance is largely dependent upon the amount and salinity of the contained water. To make this measurement, four electrodes are placed in the earth and current is caused to flow between two outer electrodes (Figure 2) past two inner electrodes, from where a possible voltage drop is measured.

It is important to remember that the measured quantity is not resistance of a finite homogeneous material (Figure 3). It is instead a measurement we call apparent resistivity - (R) - which represents a weighted average of all the true resistivities in a fairly large volume. And the measurement is influenced more heavily by the earth material close to the surface than the material at depth.

This measured resistivity value decreases (1) with the amount of water contained in the pore spaces; and (2) with the salt or ion content of the water in the pore spaces.

The presence of high resistivity material at depth (Figure 4) will force the current closer to the surface, giving a higher value in the potential measurement.

The opposite situation exists where a low resistivity material, such as clay, underlays a high resistivity material, such as a clean sand or gravel.

But what occurred when sand and gravel overlayed a fresh sandstone? Or when a rock bench occurred under a heavy bouldery cover? The electrical resistivity method was inapplicable in both cases, since the needed electrical contrast between the materials didn't exist.

However, a seismic velocity contrast did exist in both of these situations and the miniature engineering seismograph was used to check out both areas.

This is the latest model (Figure 5) in the MD line of engineering seismographs, the Model MD-3 Auto-Reset Engineering Seismograph. It is thirteen years away from our primitive cathode ray tube oscilloscopic device and nine years younger than its famous elder brother, the Model MD-1 Engineering Seismograph.

The MD-3 is the most advanced seismic tool in the world today. It is presently in use on non-destructive testing in fields as varied as an attempt to correlate seismic velocities with compaction of various materials used in highway construction, where the impact stations are separated by a very few inches, and is also being satisfactorily applied to dam site foundation investigations in the Middle East, where the depth of interest regularly exceeds 100 feet.

Among the unwanted fruits of success of the single-trace seismic technique has been the expectation that the tools can do practically anything when it comes to determining what is underneath the ground. Unfortunately, too many people have concluded that the long-awaited magic box has arrived to solve all subsurface problems. As you know (and even granting the exotic applications which have proved practical), this isn't true. We are interested in having the tools appreciate for their own worth and we would like to call out a few cautions.

The portable single-trace seismic timers we call Engineering Seismographs respond to variations in seismic velocity of earth materials. This is largely dependent upon the hardness or degree of consolidation of these materials.

With the Engineering Seismographs you are able to distinguish between rock and soil, between consolidated materials (such as hardpan or cemented gravels), and unconsolidated materials (such as loose gravels); also between compacted soils and loose soild, and between firm rock and weathered rock. But it is necessary, in the refraction method, to have the less well-consolidated material overlying the tighter material.

With the seismic tools, a new technique of delineating "rock" from other materials gained prominence. A correlation between seismic velocities of rock with stability, bearing capacity, moveability, etc., has been established by various organizations. The ripping applications is the most famous. This slide (Figure 6) showing a job in Washington gives another example of rock classification with seismic tools.

This chart (our apologies for the lack of clarity) gives information on the line number and date, along with seismic exploration data such as geophone location, length of traverse and the direction, elevation at the geophone, and the depths and velocities of the material. In the General Information Section, this statement is found: "Seismic information is based on data obtained with the refraction seismograph Model MD-1 manufactured by Geophysical Specialties Company. Impact was by an 8 pound hammer. Seismic data is intended to portray general characteristics of materials in the vicinity of the seismic line. But interpretation of the data shall consider limitations inherent in the equipment and method of exploration. In general, common materials will have velocities lower than 4,000 feet per second. But occasionally common materials will have velocities considerable higher, depending upon physical characteristics and the degree of consolidation. Bedrock will generally have seismic velocities higher than 6,000 feet per second, but occasionally will have much lower, depending upon the type of rock and degree of weathering."

One potential problem here is that one man's rock may be another man's gravel. The gneiss on this project (with a velocity around 6 - 8,000 feet per second) was a very friable material that, although in place was tight enough, when removed from quarry failed to hold the degree of hardness desired in certain fill applications. In a situation like this, the contractor seeks out first those areas where the highest velocities were closest to the surface, where they were largest in area, and, of course, nearest to his operation. This information was available. The gneiss (with a velocity of 10,000 and over) would have made excellent, long-life rip rap. The relatively low velocity (6,000 fps) did not.

As I said earlier, what is bedrock to one may not be bedrock to the other. For this reason, we strongly emphasize that contractors and engineers have greater knowledge of geology to accurately determine whether the seismic velocities have any useful meaning.

The most famous application of the MD Engineering Seismographs has been their exclusive selection by Caterpillar, Euclid and International Harvester, along with several other ripper manufacturers, to make rock rippability determinations. These next four slides (Figures 7, 8, 9, 10) show a tractor going through Georgia granite that ranges in velocity from 3,500 feet per second up to where the rock velocity was over 12,000 feet per second.

The importance of rock identification is emphasized in this next figure (Figure 11). These ripping charts take this difficulty of differing seismic velocities in different types of rock and point out that the rippability of one particular tractor mounting one type of ripper has an ability to rip rocks with differing seismic velocities at different production rates.

It is at this point that things get even stickier. Not only do different rocks give different seismic velocities, but the question then becomes "how do you know if you are getting the right velocity?".

You begin by laying the equipment (Figure 12) out along the line and measuring the time between when the energy source (a hammer, in this case) strikes the ground and when the shock wave from that hammer blow reaches a geophone pickup (Figure 13). If the material beneath the overburden is tighter, this more well-consolidated portion will allow the shock waves to return to the geophone pickup quicker and the time plotted on the graph versus the distance from the pickup to the impact point will be less for that increment.

The shock wave is literally a wave motion passing through the soil. The refraction seismic interpretation theory is based upon catching the first motion resulting from the hammer impact. (Figure 14). Sometimes these smaller amplitude first arrivals are obscured, and only the larger movements coming from the negative side of the cycle or the later, much larger, positive arrivals are discernable. All of the interpretation formulas are based upon first positive readings and it is impossible to compute equivalent formulas for the first negative (or later) readings. And since the later arrival times are not constant and predictable, it is not possible to use them through subtracting a constant time correction, for an assumed first positive reading.

We have incorporated electrical circuitry into the MD Engineering Seismograph which allows us to determine whether we are getting the up or positive side of the wave motion or the down and negative side. We then plot all of the time arrivals as has been done on this example from a bauxite mining company (Figure 14A). As you can see on this slide, and more clearly on the next slide (Figure 15), it can be seen that the time between the first positive wave motion arrival and the first negative may vary considerably. The difference is of a wide range and depends, in a complicated way, on the sub-surface conditions and is not easily predictable.

If you were attempting to make a rippability determination, it would be necessary that you tie down the true seismic velocity. If you remember in the charts that I showed earlier (Figure 11), the 8,000 feet per second material could, in many instances, be ripped. The 12,000 foot material could not. These two slopes on the graph in Figure 11 come from the same material, but result from measuring two different aspects of the wave motion. Unfortunately, it is not possible to compute what first positive readings would have been by subtracting a constant time correction from the first negative or second positive readings. The necessary correction is not constant and not predictable.

Further complications: One cannot assure himself that even if he is getting first arrival times and graphing properly that he is determining the true velocity of the sub-surface material. In the event that this sub-surface material is sloping away from the earth surface, the man in the field with the engineering seismograph would, if he were to run his line in two directions, find that he has two different "characteristic" velocities for one sub-surface material.

An additional traverse, as shown in the next figure (Figure 16), would have to be run in the reverse direction and the true rock velocity would be found to be roughly equal to the average of the two second velocities. You are also able to determine the depth of overburden at each end of your line. This is the most conventional way of checking for dipping layers and ascertaining true velocities of sub-surface. It has several drawbacks which, for now, we shall ignore. (See Geophysical Engineering Handbook).

In place of the reversed profile method, we would like to suggest another simple and quite useful technique. This "dual geophone" method is suggested as a quick means of detecting and estimating dip on sub-surface layers, estimating true sub-surface velocities for dipping layers, and drawing improved travel-time graphs with greater certainty as to which points lie on the same line segment, and all without reversing the seismic profile.

The "dual geophone" method utilizes two geophones relatively close together to get an apparent velocity at the geophone end, corresponding to every hammer station. The method is somewhat less accurate than a complete reversal of the seismic profile, but it is extremely simple, much faster, and usually sufficient.

A typical value for X_{AB} (Figure 17) is ten feet, although 20 feet may be preferable at larger hammer distance or where high velocity layers are present. Data is taken as with the conventional method, except that at each hammer station two geophones report.

To determine presence of dip, direction of dip and true velocity, without reversing the profile, we work as follows: Suppose first that the sub-surface boundary is dipping downward along the line of hammer stations, as shown here. The usual time graph appears as is shown on the left of Figure 17A. The apparent velocity for the second slope is lower than the true velocity would be. In the absence of a reverse profile, the interpreter cannot tell whether this second velocity is equal to, lower than, or higher than a true V_2.

However, if we consider apparent velocity V_{AB} measured across the geophone spread (where it is underlined in A' to B') we observe that this area represents a small segment of the reversed profile. And this V_{AB} is higher than the true velocity for V_2.

From this we arrive at the following rules for interpretation:

> If V_{AB} is higher than the apparent V_{2A}, the dip is downward along the hammer line (as shown in this figure).
>
> If V_{AB} is lower than the apparent V_2, the dip is upward along the hammer line. True velocity can be approximated by:

$$V_2 = \frac{(V_{2A} + V_{AB})}{2}$$

Several different sub-surface (horizontal layers and vertical boundaries) structures can produce a travel-time graph that is similar. It can occur where there is a low velocity overburden lying over a high velocity rock, both planes parallel. It may also come from a situation where the seismic traverse line passes across a vertical interface between two types of material, the first material a loose, low velocity type and its neighbor a well-consolidated, high velocity type. A third example can be where a very large boulder lies near the surface and horizontally adjacent along the traverse to low velocity overburden.

The usual technique for separating these possibilities is to reverse the profile, or move the geophone to the left or right and take a second seismic profile parallel or 90° from the base line.

The dual geophone method can quickly separate these possibilities. The apparent velocity across the geophone spread V_{AB} will equal the V_2 for horizontal layers, whereas it will equal V (the first velocity) in the two other cases.

No matter what the sub-surface structure may be at the various hammer stations (Figure 18), V_{AB} depends only upon conditions in the vicinity of A'B' -- so long as the seismic wave is refracted in the second material.

When the seismic wave is refracted in the third material, V_{AB} will be different, because it then depends upon the conditions along A"B".

Another quick means to determine if the sub-surface strata is parallel to the surface is to run out on an arc in either direction for a significant distance (30 degrees seems to be a desirable minimum) at any desirable point along the seismic line, to see if the lapsed times are equal. If the time lapse on radius differs, that time which is less indicates something exists in the sub-surface which has hastened the return. If the velocity is slower, as indicated by a time later than that given on the base line, then something in the sub-surface has impeded the return of the shock wave to the geophone. (Figure 18A)

One of the most seductive thoughts on means of monitoring seismic information from a hammer impact single-trace unit is that of learning something about the character of the sub-surface materials through viewing the seismic wave form on a cathode ray tube. The original 1953 MD Engineering Seismograph was such a unit. Its design, as well as other similar CRT devices, was based on work done in South Africa by Professor D. I. Gough, formerly with the University of Witwatersrand. Seismologists and electronics engineers are particularly intrigued with the possibilities thought to be inherent in such a device.

Seismic wave forms are conventionally viewed in three ways:

a. On a permanent oscillographic record produced, normally, on multi-trace seismic equipment for deep exploration.

b. On a permanent photographic record from a cathode ray tube trace.

c. By visual observation of a CRT trace, without a permanent record, but usually with long persistance phosphor on the CRT face, to make interpreting as easy as possible.

The permanent oscillographic record is the preferred method and is essential for multi-trace recording (except for polaroid photographic records). It provides an unlimited record length and (providing the paper speed is fast enough) all frequency respose and resolution one could desire. An oscillogram permits detailed inspection of the seismic wave forms and makes possible, sometimes, the detection and timing of later arrivals.

However, advantages gained from studying oscillograms are least likely to be useful at short distances and on shallow seismic problems encountered in engineering practice.

Yet, the idea that there was a possibility of identifying sub-surface materials by a "characteristic" shape of the shock wave as displayed momentarily on a cathode ray tube remained intriguing and we entered into a rather large program of experimentation. The following slides are the results of a very minor portion of the total program.

We attempted, in this continuing study, to define a set of standard wave forms to be used for non-destructive testing and identification of sub-surface earth materials.

We found that the wave forms produced by any given material depended for its size and shape upon many variables:

a. The orientation of the geophone pickup was critical to the wave form (Figure 19).

b. Amplification level was critical, as well as was the distance away from the impact source.(Figure 20). This slide also shows the negative and positive assymetry of the wave form. It is interesting to note the preponderance of the first three cycles was on the uppor or plus side of the horizontal "at rest" plane. The earth material (Figure 21) was the same over this entire traverse.

c. On this shot (Figure 22), the vertical sensitivity on the oscilloscope was changed and rendered the changes noted in this picture, even though the material was the same.

d. A change in the oscilloscope time scale (Figure 23) gave an apparent change in the wave shape, and there does not appear to be a consistency in this change which would allow for prediction.

e. A change with the geophone placement on **and** within the same material (Figure 24) brought about the wave changes shown here.

f. Changes at, and with, the impact point (Figure 25) brought about changes in the wave shape. The distance, in each case, between the impact point and the pickup was the same. The changes which occurred with each change in energy source were inconsistant and non-repeatable. Wave forms eminating from one material were drastically changed in shape, depending upon whether it came as a direct wave or as a refracted wave.

We have concluded, therefore, that it is not possible to identify sub-surface materials by means of what is sometimes referred to as "characteristic wave forms" as displayed momentarily on the cathode ray tube. Our firm continues to offer an oscilloscopic version of our own equipment, but this has been found useful only for introducing students to the idea of seismic waves and their measurement.

Seismic wave forms depend upon many factors, of which the identity of the material in the refracting layer is one of the least important. The two most important factors are the characteristics of source and whether the seismic wave has traveled along a direct path to the geophone or has been refracted in some kind of subsurface layer. An analysis of this second factor was first published by Sir Harold Jeffreys of Cambridge University in 1926. Conceivably, viewing seismic wave forms could be used to distinguish between direct waves and refracted waves, but this would have no practical value since the travel-time graph provides that distinction very simply and directly.

A third factor which influences seismic wave forms is absorption of high frequencies along the path. This, in turn, depends principally upon the <u>length</u> of the path in the <u>near-surface</u> soil layer. It can be seen that the depth to the refracting layer could influence the wave form, but not the identity of the refracting layer. In any event, this depth factor produces only gradual variations in the wave form. Furthermore, the wave form changes due to this factor would depend not only upon the depth of the refracting layer but upon a multitude of uncontrolled properties in the near-surface materials. That is, while these material properties are not principle causes of wave form variations, they do have some effect and these effects vary according to the homegeneity or heterogeneity of the material.

The preceding statements have been verified by abundant experimental evidence. The most extensive investigation was that carried out by Dr. M. E. Szendrei of the South African Roads Research Institute. He obtained several thousand photographs of seismic wave forms produced by various impact sources. Even though the sub-surface material was consistent, the seismic wave forms varied according to the impact source. (A complete set of Dr. Szendrei's photographs is in the possession of Geophysical Specialties and can be duplicated upon request).

In conclusion, please don't expect more from present day single-trace seismic equipment or the portable electrical resistivity tools than you would from any other tool, whether your drill rig or the sophisticated electronics and nuclear devices operating world-wide today. The day of the magic box has not yet arrived, but the geophysical engineering tools that are available will do until that day arrives.

ASPECTS OF USING A BOREHOLE DEFLECTOMETER TO DIAGNOSE AN UNSTABLE ROCK SLOPE

By

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INTRODUCTION

This paper will briefly outline the practical aspects of an investigation, in which the bore hole deflectometer was utilized to help define the nature and extent of an unstable rock slope. The deflectometer and its principles of operation are described as well as the location of the slope. The geology of the area around the rock slope is given; then, a short history of the unstable slope up to the time of the investigation is presented. In addition, some pertinent engineering and construction features of Interstate Highway 40 are included, I-40 being the route on which the slope occurs.

The methods and procedures of the investigation necessary to the utilization of the deflectometer and its accessories are described. This includes the preliminary investigation, diamond drill holes and the instrumentation of the drill holes. Furthermore, the system of recording the data and the format of the data sheets are discussed.

The summary contains the interpretation of the investigation and the resulting corrective and preventative procedures which the Highway Commission will follow in controlling the unstable slope.

THE DEFLECTOMETER

The deflectometer used in the investigation is a portable unit, providing measurements in one dimension only. It is a product of Terrametrics, Incorporated, and has the trade name of "Electric Portable Borehole Chain Deflectometer."

The deflectometer consists basically of three components. They are, the upper cylindrical housing unit containing the sensing mechanism, the lower probe head, which is attached to a hollow shaft that is in turn connected to the upper housing unit by an universal joint, and a piano wire tensioned from the lower probe head through the universal joint into the upper housing. There are three sets of ball rollers for positioning the deflectometer in a cased bore hole. Two sets of rollers are mounted in the upper housing unit, and the remaining set is in the lower probe head.

The space relationship existing between the piano wire and the sensing mechanism, provides a means of measuring the nonlinuarity of the upper housing and lower probe head when the deflectometer is positioned in the bore hole. A pair of reluctance transducers situated near the universal joint acts as the sensing mechanism and relays a signal to be amplified by a readout unit. The signal varies according to the width of the air gap between the piano wire and the transducers. The deflectometer has a measurable nonlinear orientation of six tenths of an inch, and a sensitivity of one thousandth of an inch.

The deflectometer has two accessories, a readout unit, and a zeroing tube. The readout unit is a null balance device producing a three digit number which can be converted into thousandths of an inch displacement by a calibration chart. The readout unit can be powered by a 56 volt D.C. power pack, a commercial D.C. to A.C. inverter, or if available, 110 A.C. voltage.

The readout unit is connected to the deflectometer by a shielded cable, of known length, which is stored on a drum and let out as needed.

The zeroing tube acts as a carrying case for transporting the instrument and is used to balance the electrical system before the deflectometer measures the nonlinuarity of a cased diamond drill hole.

The procedures and techniques of using the deflectometer will be discussed later.

LOCATION OF THE ROCK SLOPE

The slope investigated occurs in the mountainous region of western North Carolina and is located in a highway cut section. Before road construction, the slope area formed a ravine providing drainage for a large watershed.

The unstable slope is currently approximately 1600 ft long and approximately 200 ft wide. The slope is crossed by I-40 and a service road, which provides access to the Carolina Power and Light Company dam on Waterville Lake. The toe of the slope is adjacent to Waterville Lake.

After road construction had started, a noticeable amount of movement occurred in the partially constructed roadway of I-40. From a consideration of the consequences that might result from a complete slide out of material into the adjacent hydroelectric reservoir, the North Carolina Highway Commission decided to utilize the most advanced techniques available to determine the mechanics and magnitude of the moving mass. These techniques were to consist chiefly of utilizing the deflectometer, and in addition, make a highly detailed field study of the slide area and surrounding terrain.

In order to better understand the situation existing when the rock slope showed itself to be unstable, a brief outline of the design and construction of I-40 in the vicinity of the potential slie will be given.

I-40 IN WESTERN NORTH CAROLINA

There are several interesting features connected with the design and construction of this Interstate Highway in the mountainous region of North Carolina. This particular route was to provide access between the populated areas of western North Carolina and eastern Tennessee. Of necessity, the proposed road would have to cross some of the most rugged mountainous terrain in the eastern United States. The route chosen was to follow the Pigeon River for thirty miles.

Some statistics associated with the first three grading projects let to contract will give an idea of the size and nature of the construction problems encountered.

The first three projects, beginning at the Tennessee State Line, total 12.7 miles. Within this distance there are three tunnels, each being over 1000 ft in length. The total excavation was 11.2 million cubic yards, of which 87 percent was rock. The total cost of construction, excluding paving was $14,500,000.

As for alignment, the maximum degree of horizontal curvature is six percent and all horizontal curves include 200 ft spirals. Furthermore, the maximum uphill grade is three percent, while the maximum downhill grade is five percent.

The alignment parallels the lake shore of Waterville hydroelectric reservoir, and consequently, the possibility of fill slopes toeing up in the lake was viewed with alarm by the owners of the lake, the Carolina Power and Light Company.

At one time, the preliminary survey line was within 200 ft of Walters Dam; however, it was later moved away from the lake shore to prevent any possibility of blast damage to the dam as shown in Figure 1.

Because this route follows the Pigeon River gorge, there are tremendous cut slopes on the uphill side of the centerline and conversely, long thin fills on the downhill side. The majority of the cuts are well over 100 ft high and in rock. The deepest cut on the centerline being 365 ft. Most of the rock cuts are in hard, but highly fractured quartzite. This rock appears to stand reasonably well on a cut section consisting of 30 ft benches at 50 ft vertical intervals.

GEOLOGY

The geology of the lower Pigeon River in North Carolina has been documented by many people. The latest work is that of Hadley and Goldsmith (1).

The rocks in which the unstable slope occurs belong to the Ocoee Series of late-Precambrian age, and are a thick mass of clastic meta-sediments.

The unstable rock slope occurs in the Longarm Quartzite, a member of the Snowbird Group. This member consists of light colored feldspathic quartzite which contains minor amounts of magnetite and biotite. The bedding units generally are 50 ft or more in thickness.

Evidence from diamond bore holes indicates that the unstable slope is in a shear zone associated with the Greenbrier Fault, which is the major structural feature of the Great Smoky Mountains. Moreover, the slope is located approximately three quarters of a mile from this fault. The Greenbrier Fault formed early in the tectonic history of the region and is a low angle thrust fault of 25° to 35°, dipping south-southeast with a displacement of about 15 miles.

In the area of the unstable slope the Longarm Quartzite consists of silicified arkosic sandstone interbedded with sericitized phyllitic shale. The strike of the bedding is N20°E to N35°E and the dip is 30° to 35° SE. Furthermore, the unstable slope is located on the southeast flank of an anticline, whose axis parallels the strike of the country rock.

The material that is undergoing movement is sliding south-westward generally along the strike. This movement probably results from a combination of joint systems, slicken sides, and the intense weathering in the shear zone.

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[1] Geology of Eastern Great Smoky Mountains, North Carolina and Tennessee, by Jarvis B. Hadley and Richard Goldsmith, USGS Professional Paper - 349B, Washington, D. C., 1964

Figure 1. View of Waterville Lake and Dam showing proximity to slide area

The beds of the Longarm Quartzite are well jointed and the joints can be divided into the following three major sets:

1. A vertical set that strikes N50°W.
2. A set that strikes N20°E and dips 65° NW.
3. A set that strikes N25°W and dips 50° SW.

The rocks have been sheared causing the fine grained beds to become phyllitic. Shear zones are noticeable on either side of the unstable area, and can be seen cutting directly across the beds as shown in Figure 2.

HISTORY OF ROCK SLOPE

Although the quartzite through which the road is built, as already mentioned, is very hard and resistant to weathering, there have been several small to moderate slides and rock falls along the graded sections of the roadway. The unstable slope in question had particular significance because of its close proximity to Waterville reservoir and Walters Dam.

The peculiar nature of this potential rock slide was first noticed during routine boring investigations. The unstable slope and surrounding area are shown in Figure 2. As the photograph shows, the roadway is not a full bench section, and the embankment toes up very close to the lake shore. To insure that the embankment did not fall within the high water line, it was originally decided to construct a retaining wall across the toe of the fill. No borings were made in advance of construction, as hard rock was thought to be very close to the ground surface. However, when excavation for the retaining wall foundation was started, no suitable foundation condition existed.

At the request of the Highway Construction Department, a series of borings were made across the proposed retaining wall site. The borings revealed that bedrock did not exist within 50 ft of the elevation chosen for the wall footing. Moreover, the borings revealed a very odd condition. That is, depths to hard rock varied from 10 ft to 50 ft within a very short distance. This tended to suggest a narrow deeply weathered zone, possibly having resulted from or at least influenced by fault action.

Because of the above mentioned condition, it was decided to key the fill with benches cut into the original mountain side. This was done and the fill was constructed to a height of 65 ft, this being 30 ft below finished grade.

At this point in the construction, a crack developed at the back of the first bench above the roadway. A short time later, a second crack developed in the second bench above the roadway. These cracks were in sound rock, although well jointed; so, it was decided to rebuild the cut on a flatter slope.

While the plans were being revised for the reconstruction of this cut slope, a V-shaped crack developed in the roadway and was first noticed on December 7, 1964. The crack was back of the fill material and was seemingly in the original ground. The fissure in intersecting the roadway appeared to cut free a 100 ft wedge shaped mass of rock. Furthermore, the fissure could be traced to the lake surface. The fissure cut through a pinnacle of quartzite on the upriver

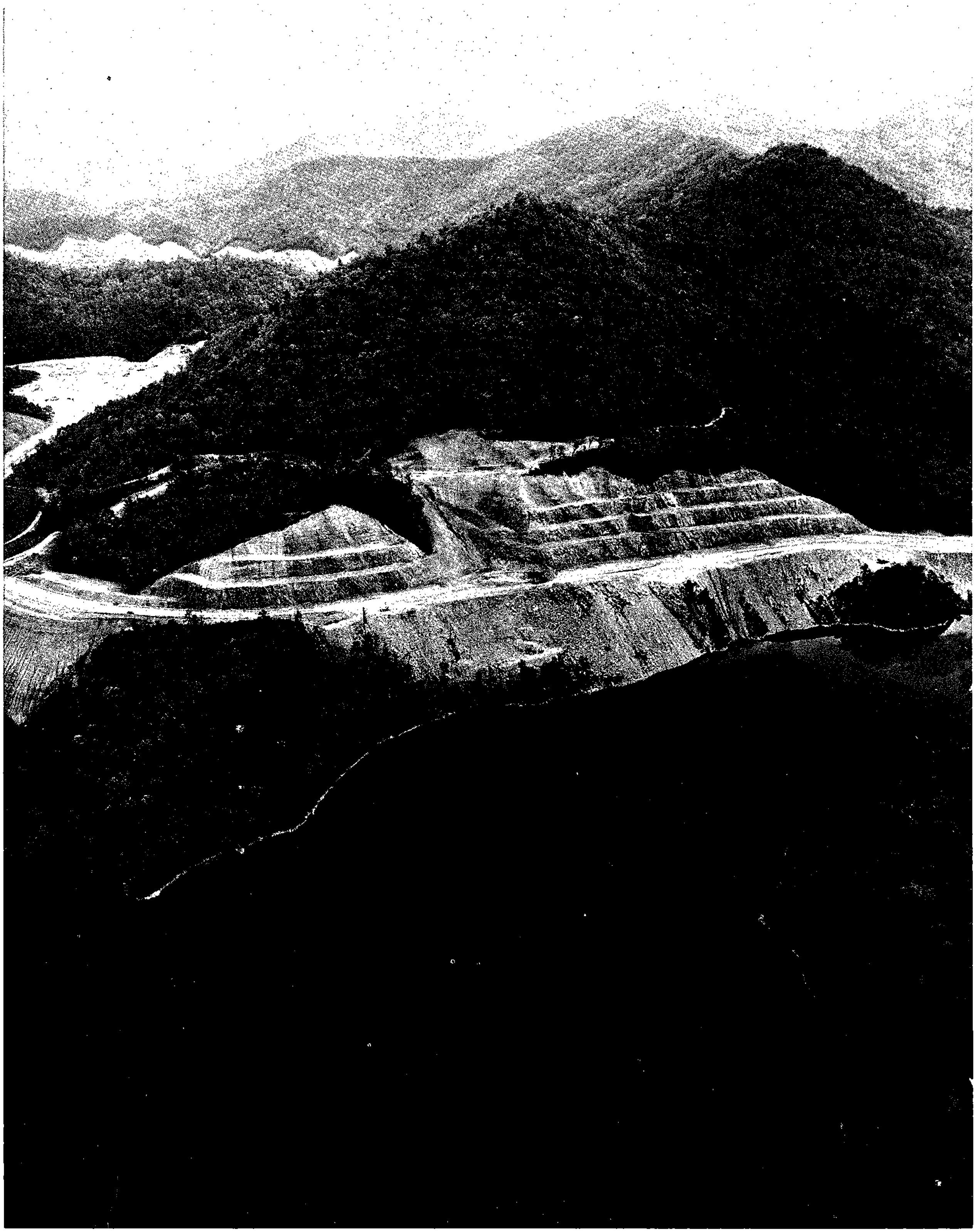


Figure 2. Oblique view of slope after removal of slide debris

side, and extended through the original shoulder on the downriver side. Further, an examination of the area disclosed two fault scarps above the Carolina Power and Light Company service road, which is above I-40 and is evident in Figure 3.

As a stop gap measure, the material freed by the fissure in the cut bank was removed and the slope made flatter; approximately 175,000 cu yd of material was removed. On the other hand, the crack in the I-40 roadway was covered and the fill brought up to grade.

In the meantime, three base lines were set along the service road, the shoulder of I-40, and the lake shore. Subsequent survey checks showed that the average movement was about one hundredth of a foot per day, although movement did not occur daily.

The Highway Commission decided to investigate the unstable slope extensively because of the magnitude of the moving mass, and because of the hazards presented to Waterville Reservoir and Dam in case the large earth mass were to slide out completely. The investigation was to utilize of the most modern equipment available.

The normal methods of investigation and subsequent corrections, which had been employed on smaller slides in the past, were felt to be inadequate or at least inappropriate in this case. Previously, at least two land slides, which had occurred in the mountainous region of North Carolina had been stabilized by blasting with dynamite and/or grouting with a fine sand-cement mixture. Obviously, this technique could not be tried here without further endangering the reservoir or the dam.

In the problems associated with tunnel construction, the Highway Commission became acquainted with Terrametrics, Incorporated, and learned of the advances in the field of rock mechanics, particularly insitu measurements of strains and rates of movement.

Here, it seemed, was a potential method of determining the most cardinal factor in the investigation of a moving earth mass, the location of the sliding zone. Once this was known, and in combination with surficial surveys and boring logs, a rational means of proceeding with possible corrective measures could be followed. Because this project was already under construction, it was urgent to determine these factors as soon as possible. Within the time available, rock strain indicators seemed the most promising.

With these things in mind, the Highway Commission purchased the Electric Portable Borehole Chain Deflectometer and accessories.

INVESTIGATION

The purpose of the investigation, as can be surmised, was to gather and interpret scientific data relative to the stability of the rock slope. The investigation was to provide answers concerning the delineation of the active movement zone on the surface, the measurements of direction, and rates of movement throughout the mass, the depth to the interface between the moving and the stable material, and a survey of ground water conditions. The investigation consisted of a preliminary survey, diamond drilled bore holes, instrumentation of

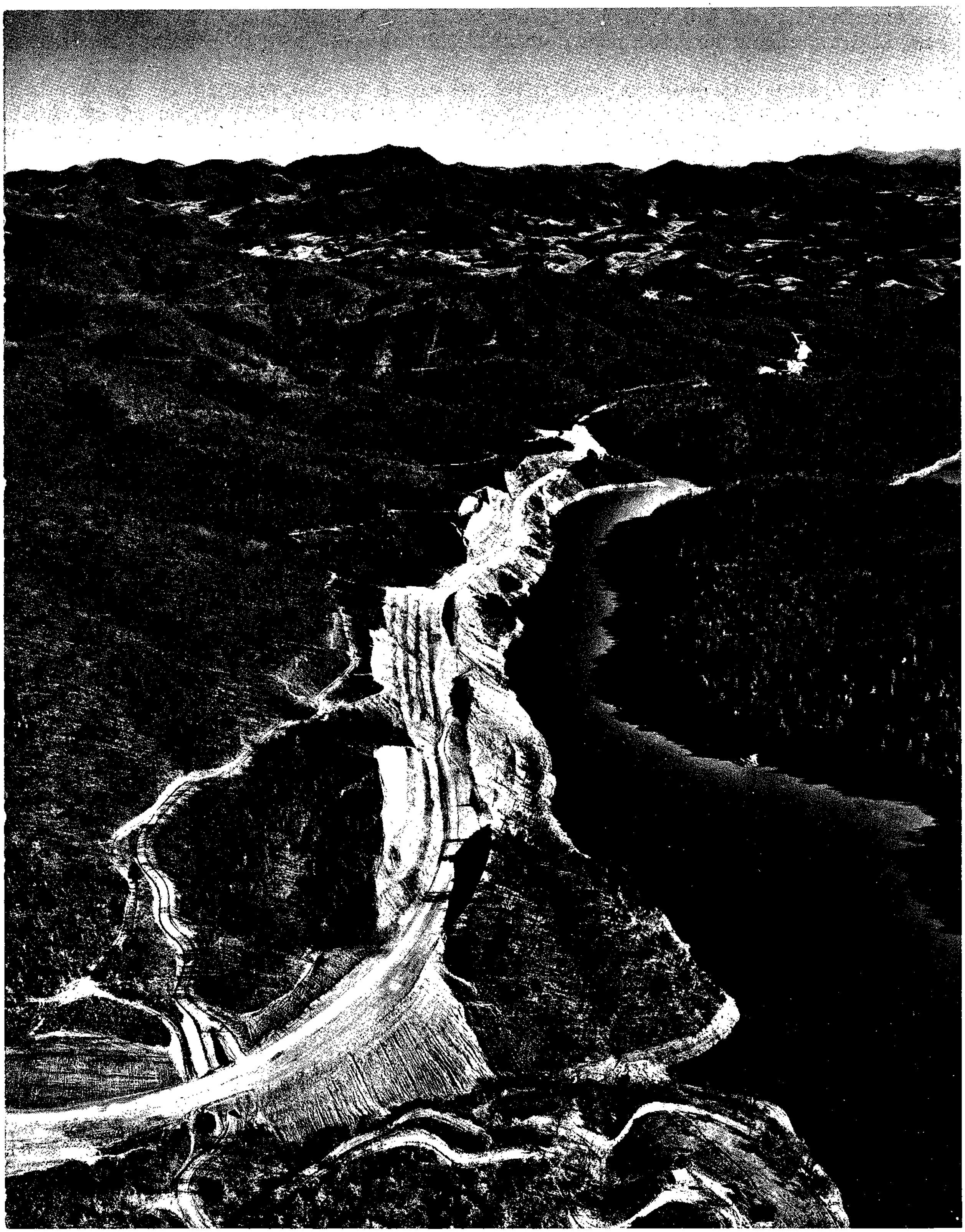


Figure 3. View of slide aréa prior to movement

the bore holes, and utilization of the deflectometer.

Preliminary Survey

The plan for the investigation first called for a surficial geologic survey with auger borings. This initial phase was to provide information necessary to determine the location and orientation of the holes in which the deflectometer was to be utilized. Of specific interest were the depth to hard rock and the nature of the overburden.

With these factors in mind, a series of auger borings was taken with a truck mounted Mobile B-52 drill in the early summer of 1965. The borings were made along the service road at about 20 ft intervals, and along the grade of I-40 at approximately 15 ft intervals.

The auger borings on the service road indicated a narrow, very deeply weathered zone of saprolite as deep as 80 ft. The deeply weathered zone was 50 ft to 70 ft wide and was bounded on either side by less weathered material that offered auger refusal at shallower depths. Material that appeared to be mylonite was recovered in the deeper holes and the water table was encountered at approximately 40 ft.

This information seemed to indicate a necessity for a diamond drilled instrument hole at the service road grade. It appeared that a bore hole here would fully penetrate the weathered zone and extend into uninvolved rock. Furthermore, the likely orientation for this hole would be normal to the weathered zone.

The auger borings along the I-40 grade did not penetrate over a few feet before hard quartzite was encountered, and the weathered zone discovered on the service road was not evident. However, the borings made for the proposed retaining wall, mentioned earlier, substantiated the existance of the weathered zone below the I-40 grade. While the auger borings were in progress the surface geology was mapped. The findings of this survey were presented earlier.

From a study of the cross section through the potential slide area, and from a consideration of the auger borings, the logical site for the first instrument hole would be on, and normal to, the grade of I-40. The first hole was to be at about 45° to the horizontal and extend to a depth necessary to penetrate the slide interface. The most suitable site for hole number one appeared to be about the middle of the unstable mass.

It appeared probable that the depth to the interface could be several 100 ft. With this in mind, it was decided to contract for a diamond drill footage of 300 ft. If from an inspection of the cores, the interface was clearly evident at a shallower depth, the remaining drilling footage could be used for other instrument hole sites on I-40 and the service road.

The drilling contract was let at $12 per foot, plus a mobilization fee of $585.00. The holes were to be drilled with a special size diamond bit which would provide a hole diameter of 3.62 in. The contract also called for the driller to provide an open hole of the specified diameter, the full depth cored, and install a 4-in. I.D. steel pipe five feet long as a permanent hole collar.

Further, the driller was to assist in the installation of the polyethylene plastic casing required for the instrument.

Bore Holes

Bore hole one was cored to a depth of 86.8 ft and a core log for this hole appears in Figure 4. The rock recovered was chiefly a quartzite; however, there were also some hard shale and phyllite fragments recovered between the depths of 20 and 40 ft. Figure 4 depicts the percentage of recovery by the symbols used. Furthermore, in computing the percent recovery, all rock fragments 4-in. or less were ignored, the assumption being they had resulted from old fractures. This hole was terminated at 86.8 ft, because at the time of coring the last 20 ft of core was thought to be sound rock.

While hole one was being prepared for the deflectometer, hole two was started. The location of hole two was determined by a consideration of what was found in hole one.

Hole two was located in an area thought to be beyond the movement zone, parallel to and on the I-40 grade, also at 45° to the horizontal. This hole was to provide information that would confirm the data obtained in hole one and locate the upstream limits of the moving mass. Hole two was terminated at 78.2 ft, because it also appeared to bottom in several feet of sound rock. A core log for this hole appears in Figure 5, and photographs of the cores recovered appear in Figure 6. After the casing had been installed in hole two, hole three was started on the service road.

Hole three was to be drilled to a depth equal to the remaining contract footage. This hole was bored from outside the apparent active zone right to left, parallel to and on the service road, also at 45° to the horizontal.

A core log for hole three appears in Figure 7. This hole encountered the weathered material found by earlier auger borings and confirmed a shear zone within the unstable mass. Hole three was completed approximately four months after drilling had started on hole one. The drilling contractor had difficulty obtaining a supply of bits and alleged to have gotten only one foot per bit in the quartzite located between 20 and 50 ft in hole three. The location of the instrument holes one, two and three are shown in Figure 8.

During the time that holes number two and three were being cored and prepared for the instrument, deflectometer readings were being taken in hole one.

Instrumentation

After hole one had been cored to a depth that appeared to insure penetration of the hole well into sound rock, the coring was terminated and preparations were made for utilizing the deflectometer.

Use of the deflectometer required that the hole be cased with 3-in. I.D. polyethylene pipe grouted in place. Furthermore, an orientation plate to position the instrument was required at each hole collar. The plate was to be anchored in a pad of concrete placed around the 4-in. I.D. steel pipe forming the hole collar. When these steps in preparing the hole had been completed, a survey

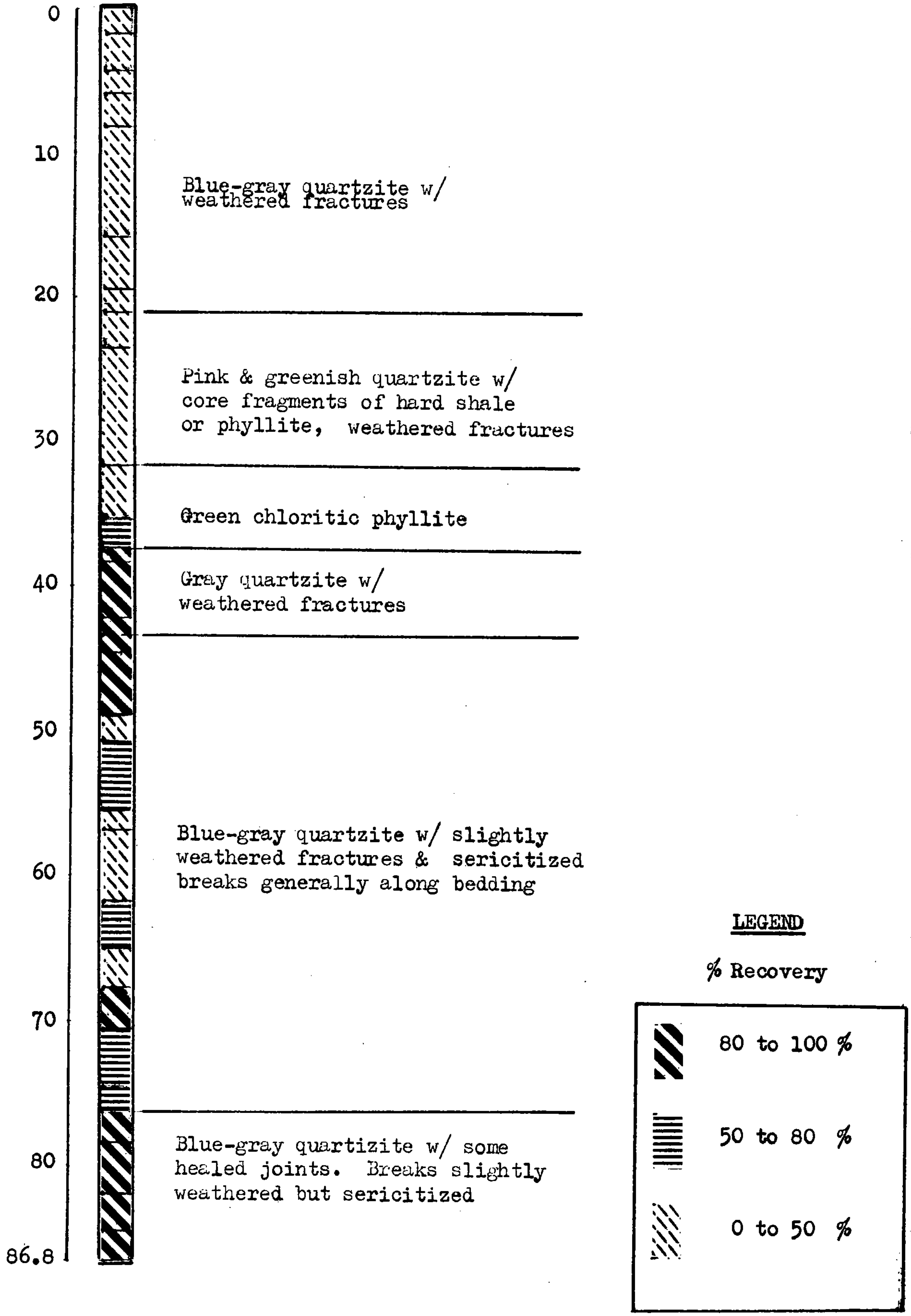


Figure 4. Coring Log for Bore Hole One

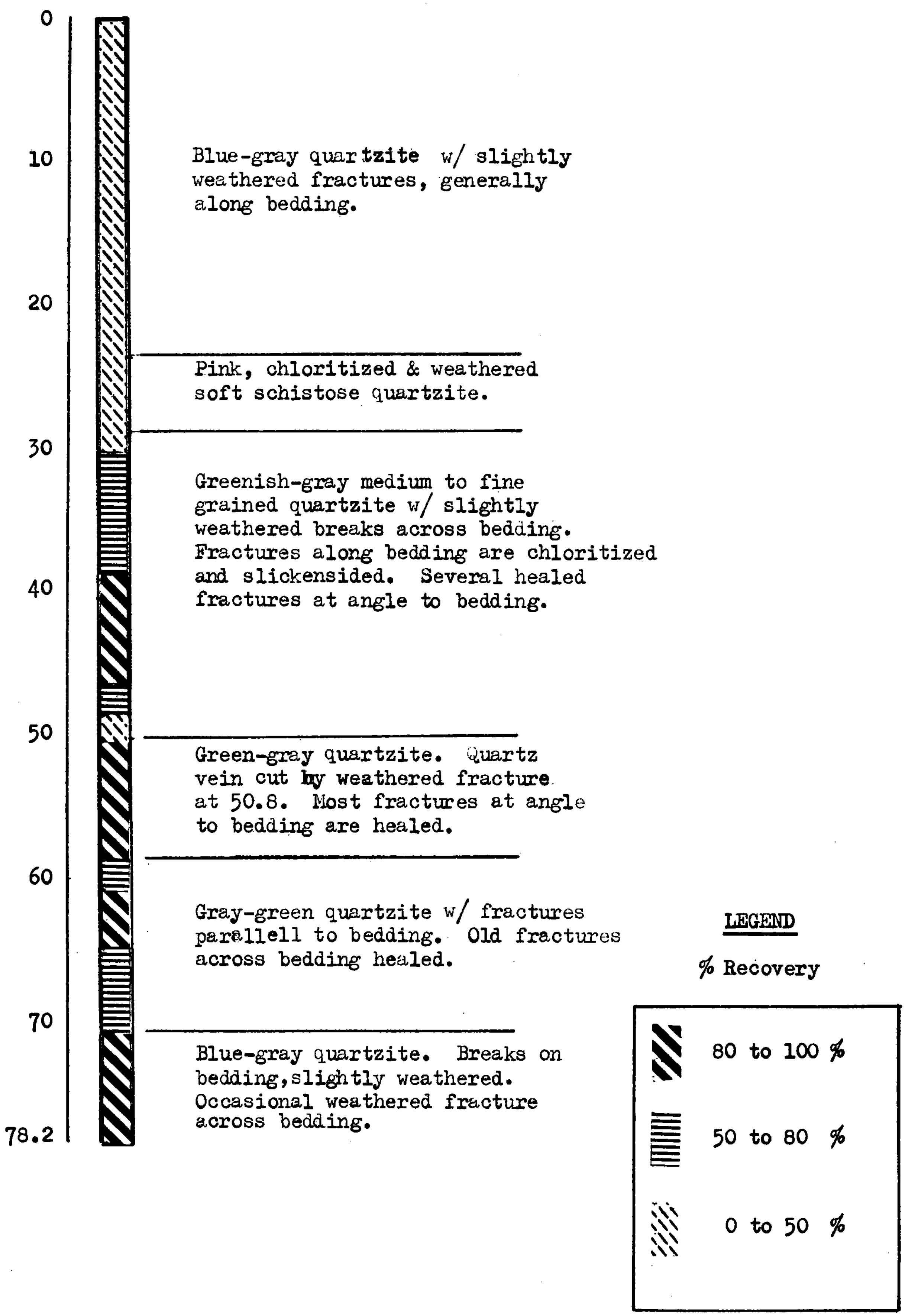


Figure 5. Coring Log for Bore Hole Two

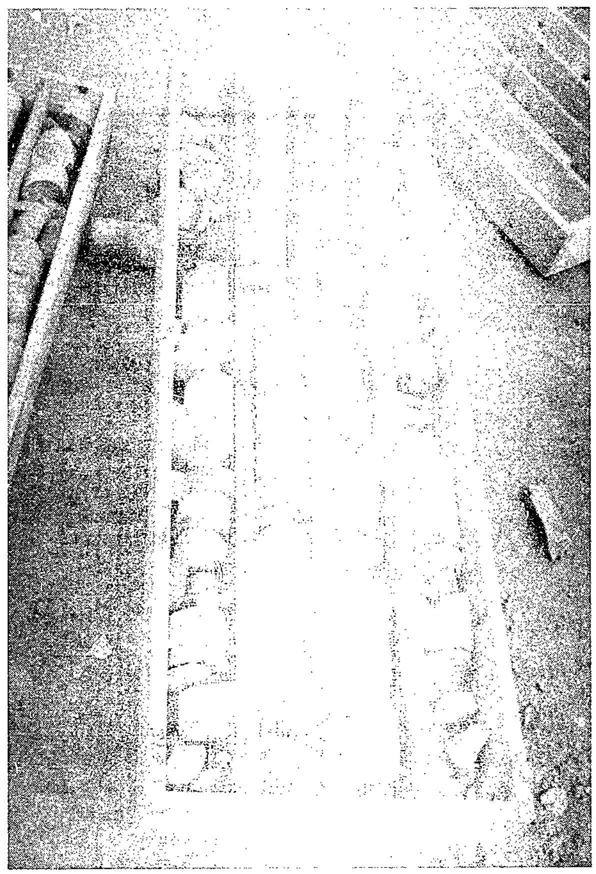


Figure 6. Part of core obtained from instrument Hole Two

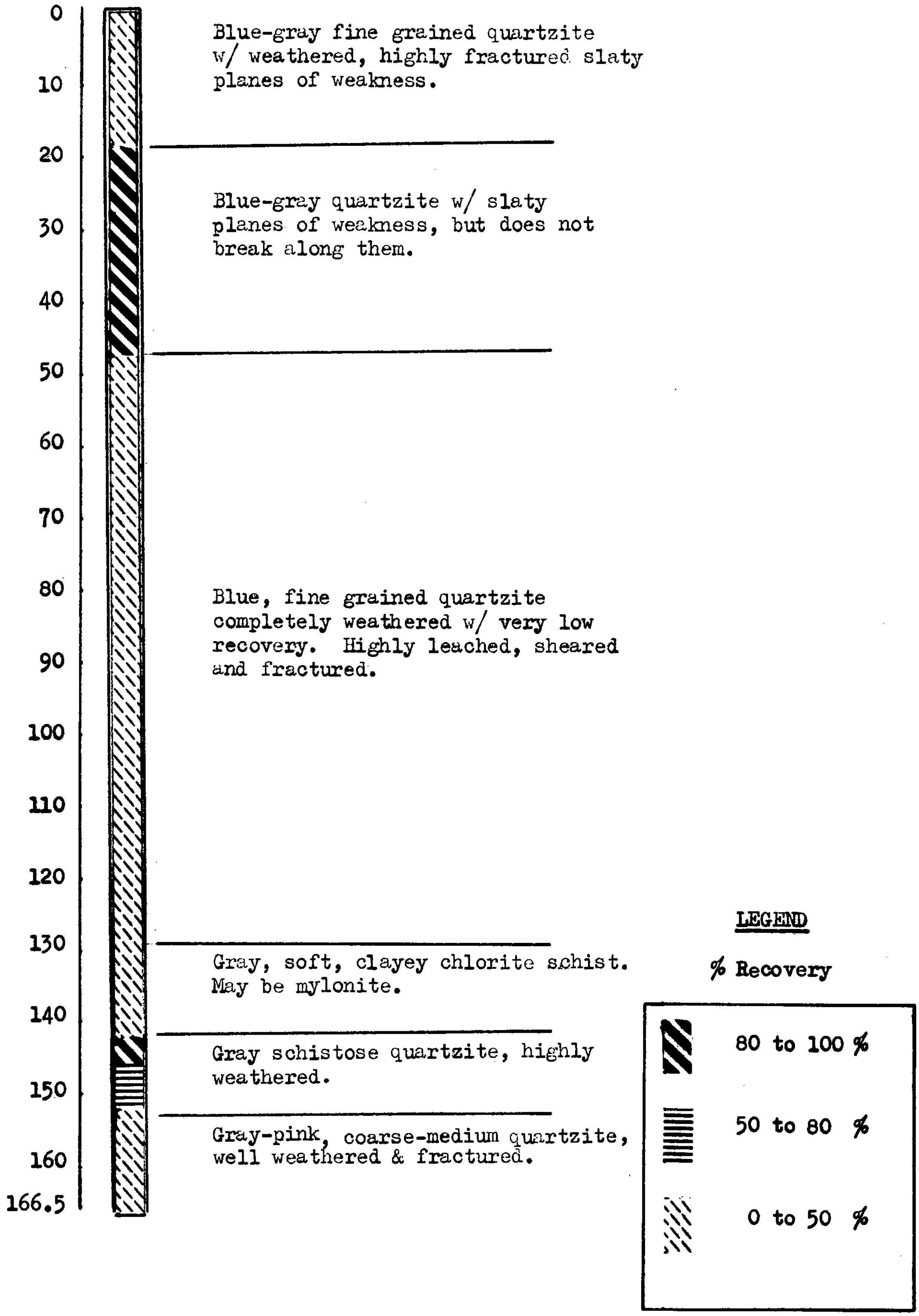


Figure 7. Coring Log for Bore Hole Three

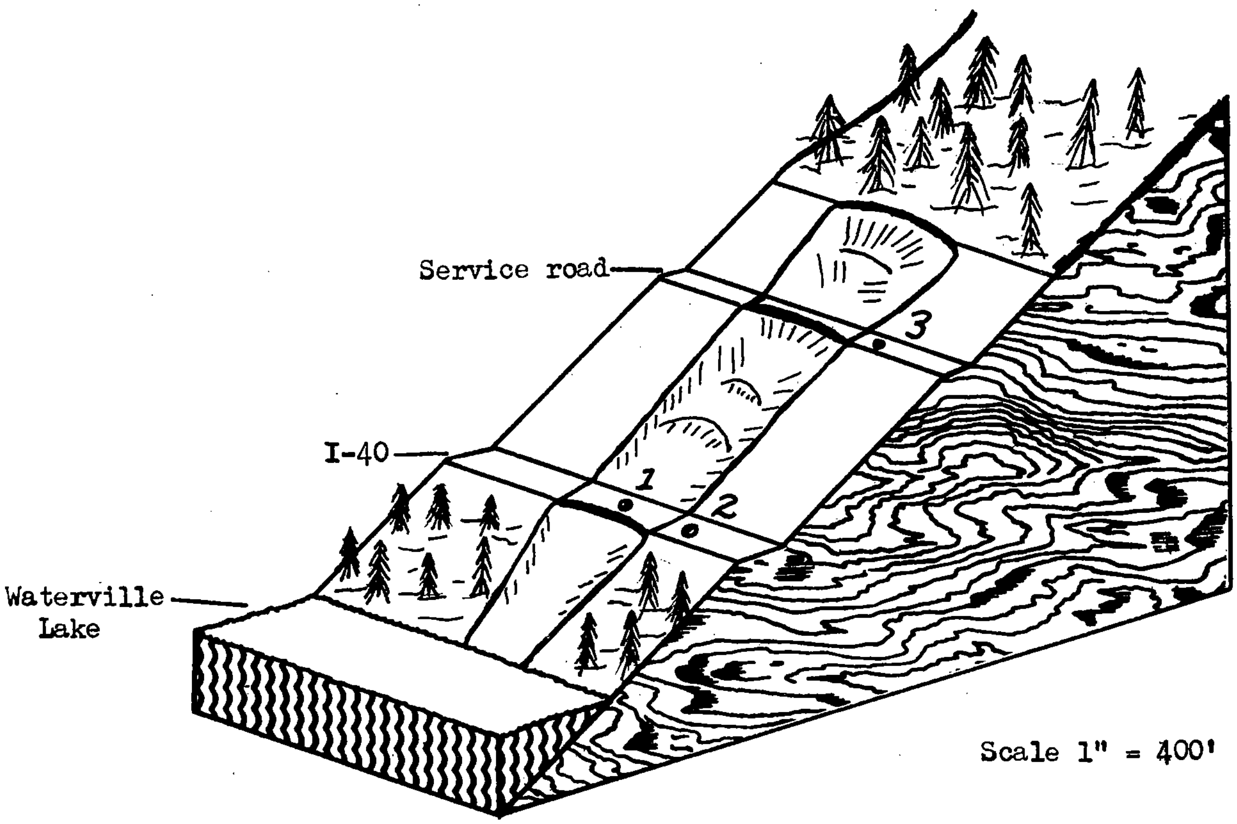


Figure 8. Location of Installations One, Two and Three

with the deflectometer could begin.

Casing the Hole:

The casing was to consist of 3-in. I.D. polyethylene pipe having a wall thickness of 3/16 in. The pipe was purchased locally for about 10¢ a foot. Because there was no known method of jointing sections of pipe together, lengths long enough to case the holes continuously were ordered.

To insure that the casing was installed to the full depth of the bore hole, it was originally intended to place a downhole anchor at the bottom of the hole. The downhole anchor would have a sheave block and cable attached to it. The cable was then to be connected to a bail on a steel plug fitted into the bottom end of the casing pipe. With this mechanism, the casing was to be pulled to the bottom of the hole.

This procedure proved to be impractical because of a combination of factors, the chief factor being the size of the bore hole. The inside diameter proved to be too small to provide clearence between the outside of the casing and the hole wall for the cable. The cable could not be brought up inside the casing because of the design of the steel plug in the bottom end of the casing. The plug was machined steel, tapped for screws and fitted with an O-ring seal made to fit inside the casing. It was connected to the casing by screws running through a steel band outside the casing and into the steel plug.

As already mentioned, the steel plug had a bail welded to it, which was to guide the casing and provide a connection for the pull cable. Furthermore, the plug had a machine drilled opening to provide an outlet for cement grout. A 1-in. I.D. polyethylene pipe was connected to this opening and extended the entire length of the casing. The 1-in. I.D. pipe was used to feed grout to the bottom of the hole, so it could be grouted upward.

Thus, the casing was pushed into the bore hole by hand and with the aid of the drill carriage. This proved to be relatively successful; however, for a really deep hole, the original idea of a down hole anchor would be necessary.

The casing was pushed to within a few feet of the bottom of hole one before refusal resistance was encountered. Water in the hole greatly added to the difficulty of placing the casing. After the polyethylene casing was in place, grout was forced from the bottom up, between the outside casing wall and the bore hole wall.

Grouting:

The grout consisted of a mixture of one bag of cement to ten gallons of water. This made a very brittle but easily placed mixture. At first the grout was poured with the aid of a funnel into the 1-in. I.D. polyethylene pipe at the hole collar. The hydraulic head was to force the grout up the annular space around the casing; however, this technique proved unsuccessful. Instead of utilizing gravity flow, it was found that a Moyno pump placed the grout more satisfactorily.

While the grout was being placed, a plug was inserted in the top of the 3-in. casing and around the 1-in. pipe. The casing was then put under 20 psi air pressure. This was done to insure that the casing maintained a perfectly round cross section.

The grouting procedure was followed in both holes one and two. Each hole required between 30 and 50 gallons of cement grout. Hole one seemed to grout satisfactorily with the grout returning to the surface and, therefore, insuring that the casing was completely grouted in. In hole two the grout was never returned to the top of the hole, and it was considered impossible to accomplish the complete grouting in of the casing. This inability to grout hole two to the surface probably resulted from the jointed and broken nature of the rock providing voids for the grout to escape.

After the grouting mixture had hardened, which required about 24 hours, the casing was trimmed and the 1-in. I.D. pipe burned loose from the steel plug.

This pipe was originally wrapped with wire that would heat up when an electrical current was induced in it. Several experiments with different diameter wires were conducted in conjunction with power from automobile batteries to insure that the pipe could be removed when in the bore hole.

After the casing was trimmed and the 1-in. I.D. grout pipe removed, preparations were made to anchor the orientation flange in a concrete pad.

Installation of the Orientation Flange:

The orientation mechanism consisted of a slotted positioning unit, which orients the probe, welded to a circular flange which in turn can be bolted to the base plate at 15° intervals. The base plate consists of a circular plate with machined holes at 15° intervals welded to a 5 ft steel tube. The 5 ft tube slides over the 3-in. I.D. casing and inside the 4-in. I.D. steel collar. In this position a concrete pad is formed at the top of the hole and the base plate is anchored in the concrete with 3/4 inch bolts. A series of photographs of this assembly appears in Figure 9 through Figure 11.

Figure 9 shows the base plate ready to be positioned inside the 4-in. I.D. steel collar. The grout pipe and wiring used to free the grout pipe from the bottom plug are shown threaded through the base plate. The 4-in. I.D. steel pipe is shown with the concrete pad placed around it.

Figure 10 shows the base plate ready to be anchored in the concrete, while Figure 11 shows the completed operation with the handling rods and deflectometer down the hole.

After the concrete pad hardened, the hole was ready to be investigated with the deflectometer.

Method of Using Deflectometer:

The deflectometer is lowered down the hole by 1-in. hollow steel rods made in 10 ft sections. These rods weigh 11 lb each and were made by the Highway

Figure 9.

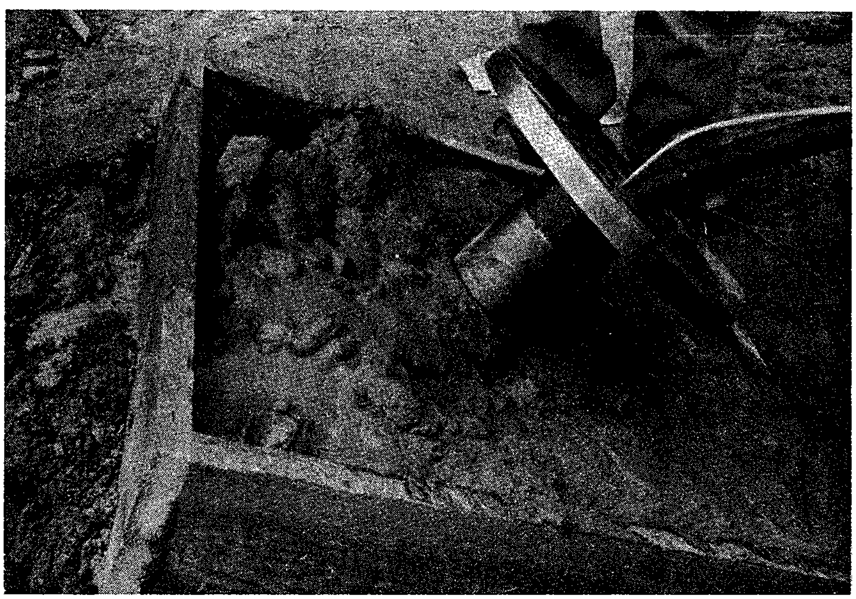


Figure 10.

Installation of indexing mechanism

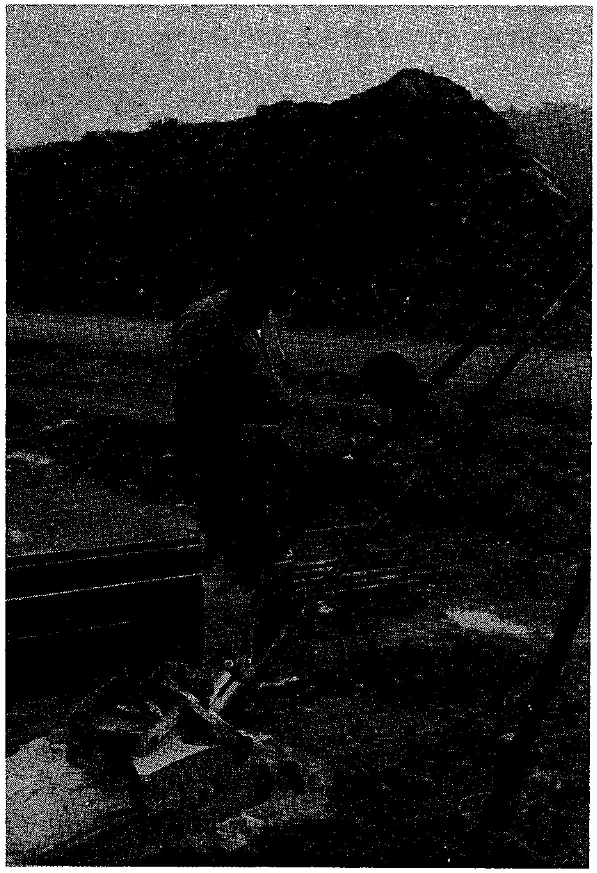


Figure 11. View of installation one with the deflectometer positioned in the bore hole

Commission machine shop.

The lead rod is connected to the upper housing unit of the deflectometer with a threaded pin. A 1/8 inch aircraft cable is also fastened to the lead rod and is wound on a hand operated winch at the hole collar. The cable acted as a safety device and was utilized in raising and lowering the deflectometer.

The additional rods are also connected together by a threaded pin. This insures that the rods can only be fastened together in a predetermined position, as the pins will connect the rods only if they are positioned correctly.

The rods have holes bored every 30-in. for the indexing handles. The indexing handles are used to orient the rods and subsequently the deflectometer, by sliding into the slots machined into the orientation flange positioning unit.

A graphic description of how the probe measures the nonlinuarity of the bore hole is depicted in Figure 12.

Before the deflectometer is lowered down the bore hole, a dummy instrument is connected to the handling rods and lowered to the bottom of the hole. This is done to insure that the bore hole is clear and that the deflectometer will not become lodged.

The dummy instrument consists of an upper housing unit, but it does not have the lower probe head. On the upper housing, the dummy has the two sets of ball rollers positioned the same as the deflectometer. It was later learned that the dummy instrument did not have the same diameter through the roller sockets as the deflectometer. More will be said about this in connection with hole two.

While taking deflectometer readings in hole one, it was found that power from an automobile battery provided the best results. It was learned that the readout unit power pack had batteries supplying a 52 volt working potential, instead of the 56 volts required. The batteries would operate at a peak capacity of 56 volts for the first hour or so, but then dropped to a 52 volt working potential. The result was what appeared to be a drift in the electrical system.

A small trailer was set up near hole one and two to provide storage for the equipment, and a canvas was erected over the holes to provide protection from inclement weather while taking readings as shown in Figure 13.

To begin the readings, the deflectometer is placed in the zeroing tube and the electrical system is balanced. In the meantime, the dummy instrument is lowered down the bore hole to insure that it is clear. Usually two men can perform the work efficiently. One man records the readout data, while another lowers and positions the deflectometer with the aid of the tripod and winch shown in Figure 11.

After the dummy is removed from the hole, the deflectometer is taken from the zeroing tube and connected to the handling rods. It is then inserted in the casing and lowered to position number one; position number one being 30-in. below the bottom of the 4-in. I.D. steel pipe used for the hole collar. Readings

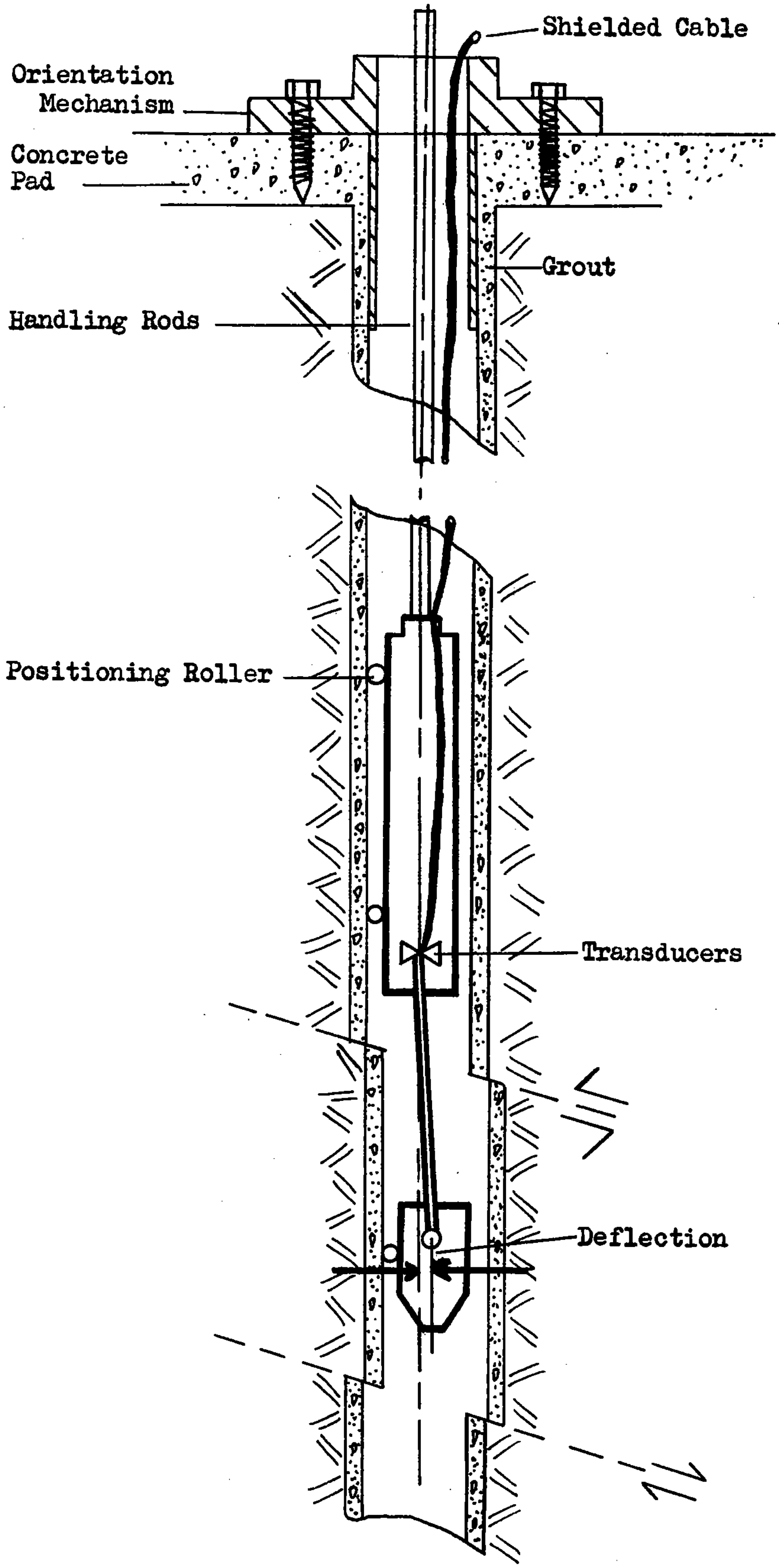


Figure 12. Schematic Diagram of the Deflectometer Positioned in an Installation

Figure 13. Installation one on the grade of Interstate 40

are then recorded for subsequent positions every 30-in. down the bore hole. The readings are all taken in the same orientation position going down the hole. It was found that it is necessary to take two readings at each position. This was done by readjusting the instrument slightly. The average reading was then taken to be used in the calibration chart.

Lowering the deflectometer from one reading postion to the next requires that the handling rods be winched upward; so the indexing handles are free of the slots in the orientation flange and can be removed and inserted in the holes for the next position. After the bottom of hole is reached, the orientation flange is rotated 90° and the instrument brought back to the surface obtaining a reading every 30-in. in the same manner as described above. After the instrument had measured the deflections in the two orthognal planes under surveillance, it was replaced in the zeroing tube and the electrical drift checked. This entire operation requires about three hours when two men are surveying a 70 ft hole.

The first readings in each plane described the original postion of the bore hole from which future readings can be compared.

Data and Records:

The three digit number obtained from the readout unit is transcribed onto a data sheet previously prepared. As mentioned earlier, two readings are averaged to give a more accurate value of the readout number for a particular position in the bore hole. This number is then used in conjunction with the calibration chart, furnished by the manufactor, to determine the displacement. It was found that the recorder had time to enter the calibration chart and convert the readout number into a deflection while the deflectometer changed positions. A copy of the data sheet appears in Figure 14.

The readout unit assigns a plus or minus direction automatically. The displacements measured in the two orthogonal planes investigated in hole one are up and down slope and up and down river, each parallel and through the bore hole. Plus directions being upslope and upriver.

To obtain the polygonal line that describes the location of the bore hole in a particular plane, the displacements are added algebraically, and the cumulative values and the depths down the hole are treated as coordinates and plotted on a graph similar to the ones in Figures 15 and 16. These graphs represent the two orthogonal planes that intersect along the axis of hole one. The scale along the absissa is in inches while the ordinate represents depths along the assumed axis of the hole as determined by the orientation of the deflectometer for the first deflection reading. As a result of the 5 ft steel pipe at the hole collar, the assumed axis is the same for various surveys.

For convenience in computing the deflection coordinates, 1000 was assumed as the value of the starting position. This insured that positive numbers would always be used in the computations.

After the coordinates of the bore hole for a particular survey are determined, the last column on the data sheet is used for recording the change in the deflection coordinates between the most recent survey and the first survey conducted.

Installation 1 Preliminary Zeroing Data: -- Date: 8/20/65 Sheet 1 of 1

Orientation: Up-and-Down Slope

Depth	Time	Readout Number Trial 1	Readout Number Trial 2	Deflection inch x 10^3	Coordinate	Coordinate Change	8-18-65
					1000		
2.5	9:00	+161	+159	+168	1168	-0001	
5.0	9:03	+226	+228	+229	1397	+0007	
7.5	9:07	-053	-051	-057	1340	0000	
10.0	9:09	+038	+039	+042	1382	+0014	
12.5	9:10	+010	+011	+010	1392	+0036	
15.0	9:11	+009	+012	+010	1402	-0058	
17.5	9:12	+017	+016	+017	1419	-0073	
20.0	9:14	-046	-049	-052	1367	-0104	
22.5	9:16	+152	+148	+159	1526	-0098	
25.0	9:17	-058	-060	-065	1461	-0127	
27.5	9:18	+030	+029	+032	1493	-0133	
30.0	9:21	+099	+095	+105	1598	-0129	
32.5	9:22	-028	-029	-030	1568	-0166	
35.0	9:24	+020	+022	+022	1590	-0180	
37.5	9:25	+128	+128	+137	1727	-0173	
40.0	9:28	+017	+016	+017	1744	-0186	
42.5	9:30	-127	-128	-143	1601	-0220	
45.0	9:35	+172	+178	+175	1724	-0268	
47.5	9:36	+060	+060	+066	1850	-0208	
50.0	9:38	+133	+135	+143	1993	-0195	
52.5	9:40	-036	-036	-038	1955	-0196	
55.0	9:41	-044	-044	-046	1909	-0212	
57.5	9:42	+030	+027	+031	1940	-0195	
60.0	9:43	+077	+077	+084	2024	-0177	
62.5	9:45	-098	-098	-110	1914	-0140	
65.0	9:46	-026	-028	-029	1885	-0125	
67.5	9:47	+121	+120	+129	2014	-0116	

Figure 14. Format of Portable Deflectometer Data Sheet

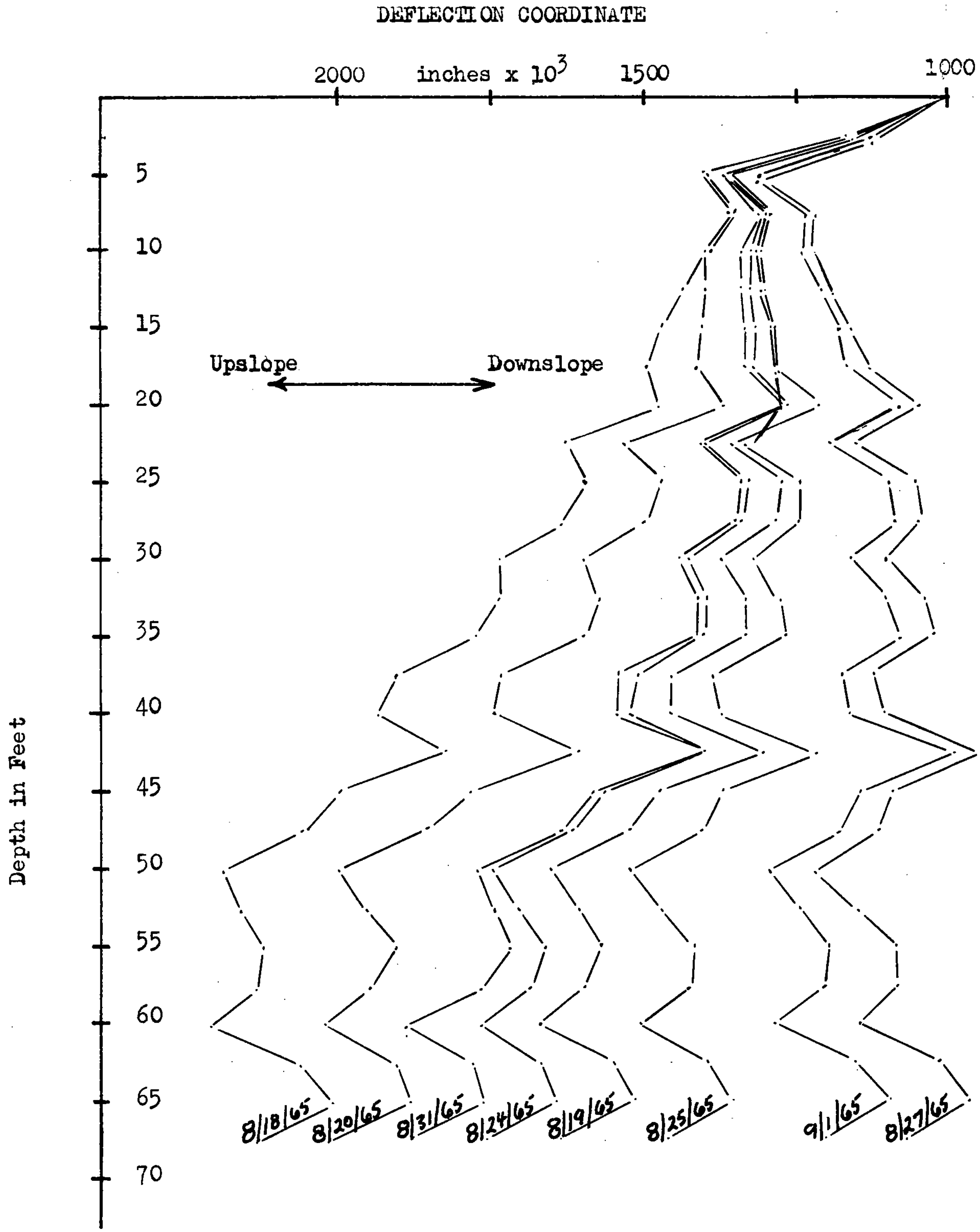


Figure 15. Relative Displacements in the Up-and-Down Slope Plane Investigated in Installation One

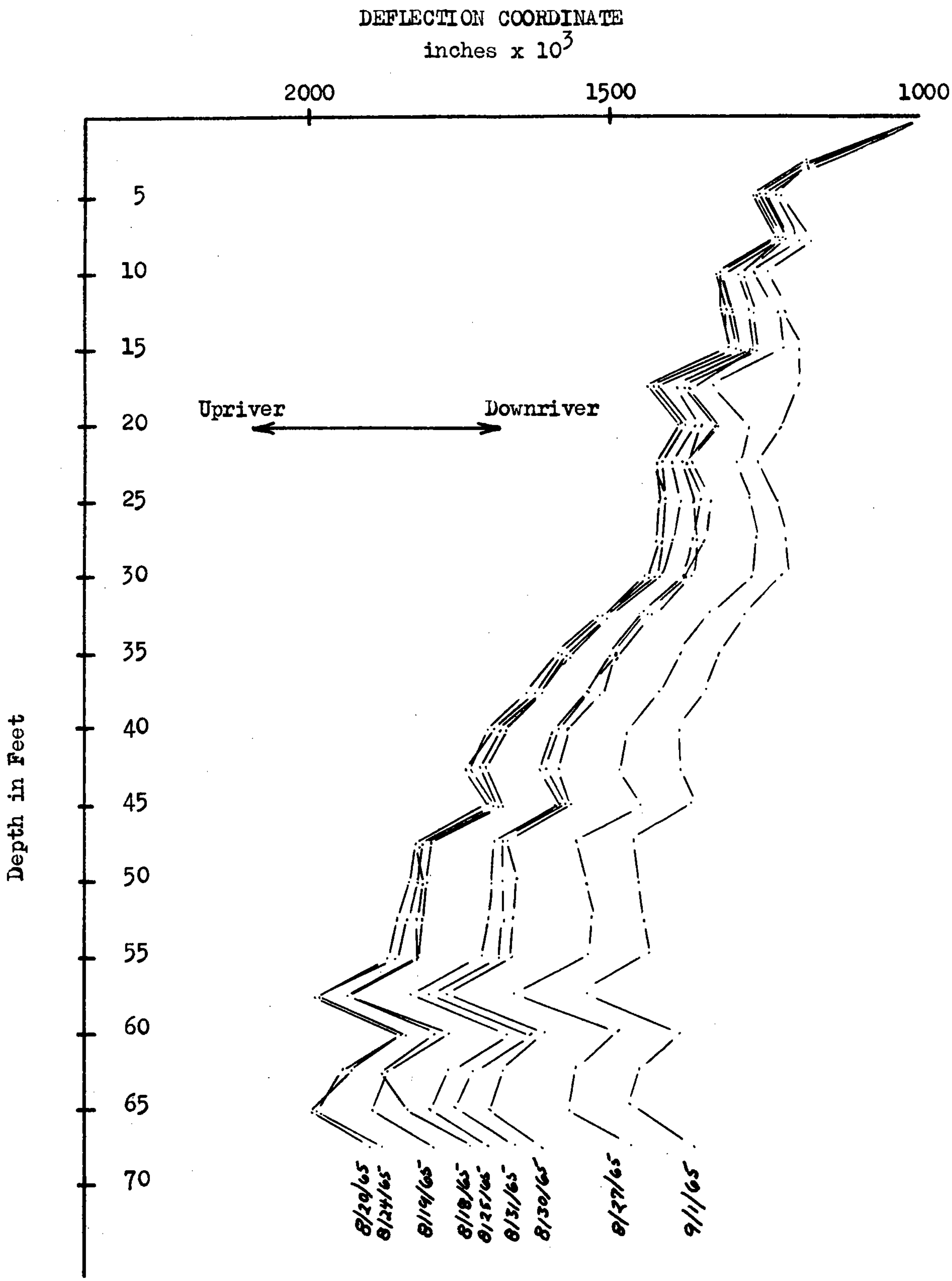


Figure 16. Relative Displacements in the Up-and-Down River Plane Investigated in Installation One

The data can be presented in several ways after it is recorded and the method presented above was chosen because of the circumstances involved.

SUMMARY AND CONCLUSIONS

The environment of the unstable slope and its location have been outlined as well as the history of events leading up to the time movement was first noticed. Also, the general plan of the investigation has been presented, which included preliminary borings and surface mapping of the geology in the slope area. Furthermore, the description of the data obtained with the deflectometer in installation one is included. The methods of preparing diamond bore holes for the deflectometer and the methodology of using the deflectometer have also been discussed.

The interpretation and analysis were made without benefit of deflectometer surveys in installation three but included a small amount of data from installation two. The deflectometer was lodged down the hole in installation two shortly after a survey of this installation was started. After recovery of the deflectometer, it was returned to the manufacturer for repair, which curtailed the data obtained with the deflectometer.

The polyethylene casing in installation two did not have a uniform inside diameter of 3 in., being somewhat smaller in places. Therefore, the deflectometer did not move freely inside the casing, and finally lodged at a depth of approximately 50 ft. On the other hand, the dummy instrument moved freely inside the casing, and therefore indicated that the hole was clear for use with the deflectometer. As it was learned later, the brackets holding the positioning rollers on the dummy instrument were flush with the housing, while on the deflectometer the roller brackets protruded approximately 1/8 in.

In trying to recover the lodged deflectometer, the upper housing separated from the lower probe head and sensing mechanism and was recovered with the handling rods. The remaining components were later recovered with the aid of a threaded outside tap.

Without sufficient deflectometer surveys in installation two to determine movements, the analysis of the unstable slope was made from a consideration of the boring data, deflectometer surveys in installation one, and a study of surficial features.

Interpretation and Analysis

Figures 17 and 18 show tension cracks in the cut slope above the grade of I-40, while Figure 19 represents a vertical cross section more or less parallel to the strike of the bedding and through installation one. More will be said concerning Figures 17 and 18 presently.

Figure 19 is a conception of the likely orientation of the strata existing within the unstable slope. The configuration of the strata is based on the boring records and includes the auger holes on the service road, diamond bore holes for the deflectometer installations, and the borings required for the proposed retaining wall. In this drawing the types of strata have been simplified into two

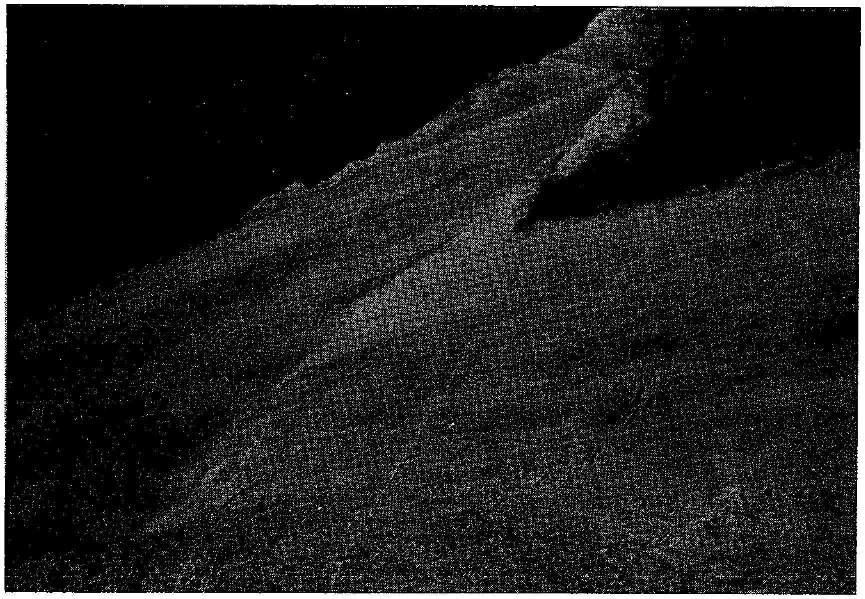


Figure 17.

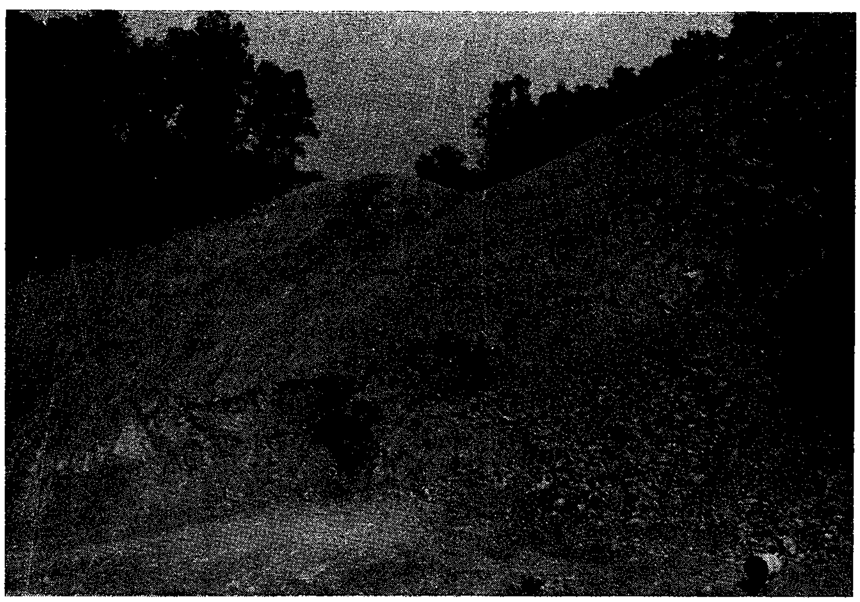


Figure 18.

Tension cracks in the slope above Interstate 40

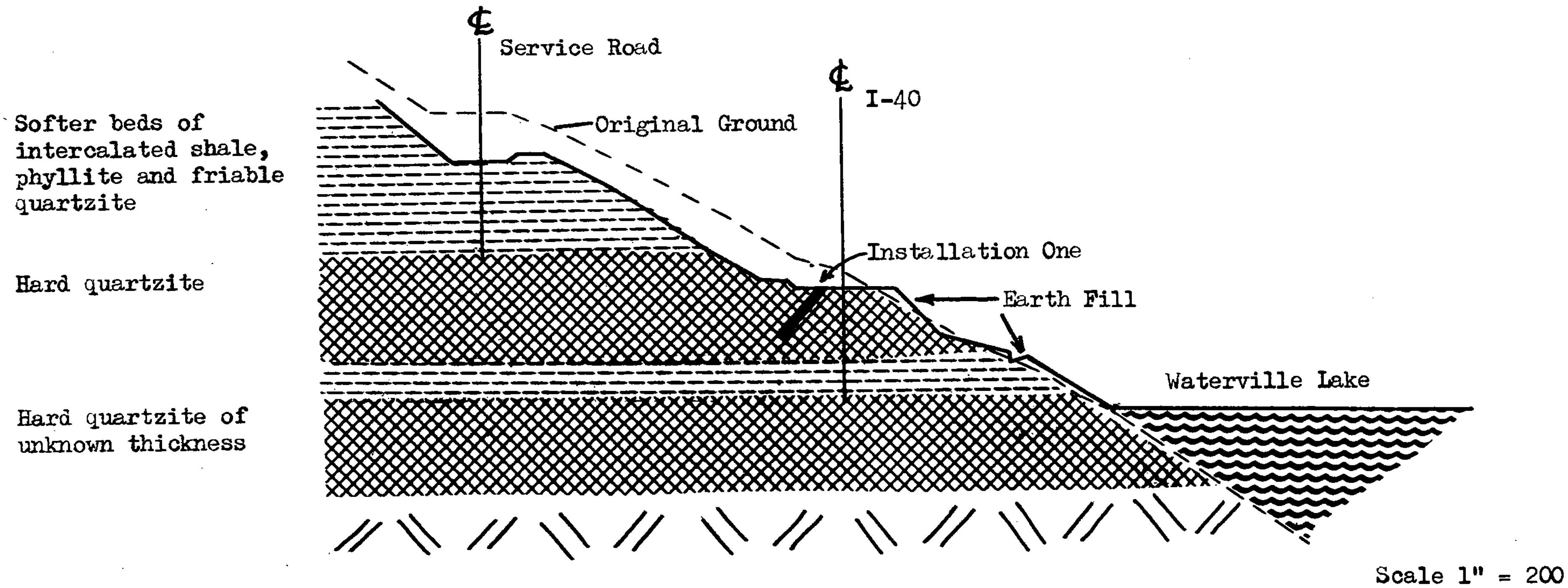


Figure 19. Vertical Cross Section Through Installation One

categories. One is the hard quartzite, which is well jointed, and the other is the softer more weathered beds of intercalated shale, phyllite and coarser grained quartzite.

Both types of beds have been affected by the Greenbrier Fault and occur within its complex. This is evidenced by the extreme jointing of the hard quartzite and is further substantiated by the shear zones in and adjacent to the unstable area.

Figure 19 is a greatly simplified schematic drawing of a very complex structural problem. The interpretation of movements in the unstable area are in part based on this conception of the rock structure and are as follows:

The grade of I-40 occurs in the hard unweathered quartzite stratum, while the service road occurs in the softer more weathered stratum overlying the quartzite. Below I-40 there is still another layer of soft weathered material as evidenced by the borings for the retaining wall.

Because I-40 traverses across the strike of the dipping country rock in the unstable area, the alternating hard quartzite and softer layers outcrop in rib-like fashion. This is substantiated by drill holes collared above and below the grade line of I-40.

The full depth to which a hole can be augered increases as drilling progresses toward the direction of dip until the next hard quartzite outcrop is reached. At this point auger drill refusal is at or very near the ground surface.

The movement observed along the slope as attested to by the base lines and the tension cracks occurring in the cut slope above I-40 in Figures 17 and 18, can be partially explained from a consideration of the structural relationship existing between the hard and soft strata.

In the soft and weathered strata underlying the quartzite stratum, as depicted in Figure 18, tends to move out and down under the weight of the overlying material, then the quartzite bed on which I-40 is situated must periodically readjust itself. The softer stratum above I-40, on which the service road is situated, is subsequently affected and exhibits tension cracks. On the other hand, the upper strata can slough off and slide onto the I-40 roadway because of its inherent unstable nature without the influence of the underlying material.

The deflectometer survey in installation one indicated that the bore hole did not penetrate through the moving zone into unaffected stable material. The deflectometer survey did indicate that relative to the top of the hole, the lower extremities were moving at a faster rate, although sporadically. Because of the initial belief that bore hole one extended well into sound rock, the top of the installation was never accurately referenced. Thus, all movements measured in installation one must be referenced to an arbitrary point on the bore hole axis in order to obtain relative movements within the hole. Therefore, the only indication of absolute movement is the base line data and the occurrance of tension cracks.

From a consideration of the above facts the North Carolina Highway Commission came to the following conclusion:

The slope is now resting on about the same angle of repose as that existing before roadway construction was started, and in addition, the removal of the material from the cut slope above I-40 has reduced the driving forces. Therefore, the stability of the slope with respect to a complete slide out should be about the same as that as existed before I-40 was constructed. Similarly, the magnitude of the problem, as presented earlier, has been reduced from one involving a complete slide out to one involving periodic movements in the roadway of I-40 and the service road.

Corrective Measures

Several methods of correction were considered and later determined infeasible. The possible corrective measures considered were: flattening the cut slope to reduce the driving forces; loading the toe of the slope to counter weight a circular movement; rock bolting; blasting and cement grouting of the movement zone; and internal drainage of the slope to reduce hydrostatic cleft pressure.

Flattening the cut slope above I-40 would entail changing the alignment of the service road and lengthening of the entire slope. On the other hand, nature will probably assist in future flattening of the slope angle, and inasmuch as there is a fairly wide catchment area at the toe of the cut on the I-40 grade, the slumping or falling materials can be retained in this area and removed as necessary. As materials continue to slump towards the roadway and are removed from the slide area, the slope will be approaching a final angle of repose and the load on the head of the sliding mass will be reduced.

The most economical control measure available appears to be that of constructing a counter weight at the toe of the moving mass, near the shore of Waterville Lake. There appears to be room for at least 50,000 cu yd of material above the lake shore; however, considerable planning would be required to locate a haul route for placing the materials. Furthermore, any material placed in the lake would be reduced to its bouyant weight, and in addition the Carolina Power and Light Company would object to any loss of storage.

Probably the most positive method of stabilizing the sliding mass would be to increase the resistance to movement along the slide interface. This could be accomplished by either drilling, blasting and grouting the interface or by rock bolting. There are certain drawbacks to either of these methods. The proximity of Waterville Lake and Dam prohibits any use of explosives. It is conceivable that a dynamite shot large enough to break up the hard quartzites sufficiently for grouting could break the entire unstable mass loose and send it into the lake. Other drawbacks involve the spacing and depths of diamond drill holes required for the installation of some type of rock bolting. Holes bored for rock bolt installations would of necessity have to be bottomed in uninvolved rock. These holes would likely be on the order of 100 ft or more in depth and spaced at very close intervals. At an average cost of $10.00 per foot for the drilling operation, this method quickly becomes uneconomical.

Very little has been said about the influence of ground water in the action of the moving mass, but it is certain that rapid drawdowns of the reservoir have a termendous effect upon the cleft water pressure within the slope forming materials above the reservoir shore line. There is at present no way to assess the importance of rapid changes in the cleft pressures towards the lake, but it can be readily seen that equilibrium of the fault weakened slopes would be adversely affected.

During the months immediately following the discovery of the original breaks in the I-40 roadway, a considerable amount of ground water was observed percolating from the toe of the cut just above the roadway. A short time afterwards it was discovered that surface water was entering the slide area above the service road cut as a result of spring discharge down the center of the ravine. This water was immediately diverted, but it is thought that other sources of ground and surface water exist and that a system of horizontal drains in the ravine above I-40 would be highly beneficial. The presence of Waterville Lake greatly complicates any attempt at installation of a horizontal system below the I-40 roadway, however, such a system would also have beneficial effects.

In view of the conclusion, that no catastrophic slide out is imminent, as already discussed, and that the movements taking place are sporadic and of small magnitude, it was decided that the best course of action would be to place a temporary pavement over the unstable area and adopt a "wait and see" position.

PART II

PANEL DISCUSSION ON CHEMICAL AND PHYSICAL REACTIONS OF CARBONATE AGGREGATES IN CONCRETE

T. L. Welp, Iowa State Highway Commission, Presiding

INTRODUCTION: T. E. McElherne, Materials Engineer, Iowa State Highway Commission

C. J. Roy, Dean, College of Sciences and Humanities, Iowa State University

PANEL MEMBERS:

1. Howard Newlon; Virginia Council of Research; Charlottesville, Virginia
2. Katharine Mather; U. S. Corps of Engineers; Jackson, Mississippi
3. David Hadley; Portland Cement Association; Skokie, Illinois
4. Ely O. Axon; Missouri Department of Highways; Jefferson City, Missouri
5. James R. Dunn; Dunn and Associates; Averill Park, New York
6. J. E. Gillott; National Research Council; Ottawa, Canada
7. John Lemish; Iowa State University; Ames, Iowa

EDITOR'S NOTE ON ORGANIZATION FOLLOWED IN THE PUBLICATION OF THE PANEL PRESENTATIONS

The order of presentation and content of each contribution to the panel discussion is outlined below. This follows closely the manner in which the panel was presented: The panelists agreed that the main topic should be "Chemical and Physical Reactions of Carbonate Aggregates in Concrete." David Hadley presented an introductory summary on carbonate aggregate reactivity as a background for the discussion. Mrs. Katharine Mather then followed with a brief resume of chemical test methods used in investigating alkali-carbonate reactions to provide the audience with a better understanding of the terminology and techniques. Various chemical aspects of carbonate aggregate reactions were presented in order by J. E. Gillott, Howard Newlon, E. O. Axon, and Katharine Mather. Following a brief intermission Howard Newlon reported on physical test methods used on aggregates. The physical aspects of carbonate aggregate reactions were presented by James Dunn and John Lemish. This completed the formal panel discussion which was followed by a question and answer period in which the panelists and audience participated. The entire panel was tape recorded and participants were given an opportunity to edit their remarks. In publishing the panel discussion the moderator's remarks have been omitted. Reference lists have been included where pertinent to each presentation. Although considerable delay, for which the editors take responsibility, was encountered, the panel when viewed in its entirety provides a summary for geologists on the current state of the art concerning alkali-carbonate reactivity.

Order and Topic of Panel Presentations

T. E. McElherne	Introduction
J. Lemish	Background on Carbonate Aggregate Behavior in Concrete
Katharine Mather	Test Methods Used in Investigating Carbonate Aggregate Behavior
J. E. Gillott	Concrete Performance as Related to Behavior of Carbonate Aggregates
Howard Newlon	Chemical and Physical Reactions of Carbonate Aggregate
Ely O. Axon	Concrete Performance as Related to the Behavior of Carbonate Aggregates
Katharine Mather	Waterways Experiment Station Experience with Alkali-Carbonate Reaction
Howard Newlon	Physical Test Methods Used in Investigation of Aggregate Behavior
James Dunn	Distress of Aggregate by Adsorbed Water
John Lemish	Concrete Weathering
Panelists and Audience	Questions and Answers

INTRODUCTION

by T. E. McElherne

Ladies and gentlemen, it is very nice to be invited here to provide an introduction to this symposium on aggregates and durability of concrete. According to Ted Welp, an introduction should be by an expert, but it should not provide answers. I am qualified to be here, not to provide the answers, but I assure you that I am licensed in the State of Iowa to be very critical of concrete. I am a licensed driver, like everybody else.

The durability of concrete can be approached from three different aspects: 1) as an engineer, 2) as a technician, and 3) as a geologist. I think the engineer in general is more interested in concrete in terms of whether a structure is satisfactory or not. I assure you that he has some work to do that is yet undone in achieving this. The technician is more interested in what takes place in concrete after it is made and placed; what does the water do; what do the impurities in water do and how do they react in a mass of concrete. The geologist, on the other hand, is more interested in the rock itself, and I believe this is why we are here today. The geologist has to observe things in the rock and be able to tell things about the rock by tests or other means; however, his main interest is in the rock. We have all three groups working on the problem. I recognize that, although their work is completely overlapping, each should be aware of the area in which he is working.

We currently have a considerable amount of research on durability of aggregates in concrete. We have research in all three areas of specialization, each of which produces different answers. This should bother people; it should bother them enough to continue their research. Difference in answers is not really a serious problem, because the results eventually will mesh together like gears, and that is the goal we are trying to reach. Should one person's approach be geology, his immediate conclusions would certainly be different from those with an approach of an engineer, but eventually they will meet on common ground. Sometimes this common ground appears to be far apart in the beginning. Sometimes we are on common ground and do not recognize it.

In defining the problem of concrete durability, I would like to charge this group with certain objectives. The first is to proceed with concrete aggregate research until we are on common ground and we can recognize this fact.

The second is to provide simple laboratory tests which can recognize things that are not desirable in rock. This might be through chemical tests, it might be petrographic analysis, or it could be a combination of a number of techniques. In addition, we must have a better understanding of how these undesirable properties interact with each other. Where more than one undesirable factor occurs in common with others, their effect may be additive, multiplied, or exponential, although any one property might not be too serious by itself. I think a good many of these rocks that people recognize as undesirable are fairly well understood. We don't know exactly why, but we can list a number of their properties, and we know that we are on the right ground when we put them on the undesirable list.

The third objective, and this is important, is how much of the undesirable property can be tolerated? If we are thinking in terms of magnesium oxide contents in the intermediate range, such as Dr. Lemish has discussed, then how much rock with such composition can be tolerated? What does it do to the life of the concrete? How much will the life of the concrete be shortened? This is important because we know in Iowa that we cannot use the best aggregates indefinitely. These aggregates are being depleted, and we will be forced into mixing high quality aggregates with aggregates from a known poor source because of the price. We are not interested in building pavements to last 100 years. We are interested in knowing how long a pavement will last, so we can build it the most economical way. If this involves using the best aggregates and hauling them across the state or from some other state, then we probably would want to do this. But, if it is known that using a cheaper aggregate having a shorter life is appropriate to the project and more economical, then this is what we want to use.

In summary, we want to get all these properties into a numbering system representing yardsticks so that we can measure rock and come up with reliable answers.

Iowa has been concerned with concrete durability, especially as it is related to aggregates, for a long time. I am happy to be here to provide this introduction and will not take any more time from the panel members, because I recognize they are indeed experts in this field. A number of these panel members have been in Iowa previously to discuss this problem in one way or another. We welcome them back and hope that you will have a nice time and a very fruitful meeting.

BACKGROUND ON CARBONATE AGGREGATE BEHAVIOR IN CONCRETE

by John Lemish

A brief written summary on carbonate aggregate behavior in concrete is provided by the editors as an introduction to the panel discussions. David Hadley of the PCA provided a thorough background during the panel discussions.

Coarse carbonate aggregates are widely used in concrete with great success and only in certain instances has their use been related to deleterious concrete behavior. Two types of carbonate aggregate behavior related to distress in concrete have been identified to date on the basis of service records. Research on such carbonate aggregates behavior has helped define several aspects of the problem and provided knowledge on their physical and chemical behavior.

The problem aggregates which to date have received most attention are those which expand in alkaline environments. A direct relationship between the expansivity of cylinders of aggregates soaked in sodium hydroxide solutions to expansion of concrete causing deterioration has been established in Canada and Virginia. Expansive aggregates have been widely studied by many groups throughout the country. Research indicates that the expansive rocks range in composition from dolomitic limestones to calcitic dolomites and have a texture characterized by a fine grained matrix of calcite and claysized particles containing larger rhombs of dolomite. They have a high content of clay size material and many of them are physically weak rocks which cannot pass physical acceptance tests. The expansion of the aggregate is considered to be closely associated with the chemical reaction called "dedolomitization". The dedolomitization reaction is considered to proceed as follows:

1. $CaMg(CO_3)_2 + 2NaOM = Mg(OH)_2 + 6CO_3 + NaCO_3$ or

2. $2CaMg(CO_3)_2 + 2NaOH = MgCO_3 \cdot Mg(OH)_2 + 2CaCO_3 + Na_2CO_3$

3. $Na_2CO_3 + Ca(OH)_2 = 2NaOH + CaCO_3$

The last reaction is a regeneration reaction proposed by Hadley in which the sodium carbonate reacts with the products of cement hydration represented here as calcium hydroxide to produce more sodium hydroxide. Research in dedolomitization indicates that it occurs to some degree in all dolomitic rocks in contact with alkaline solutions but only certain ones with the characteristic textures described above expand. Most of the dolomites shrink a slight amount. The actual mechanism of expansion is not agreed upon. Some refer to osmotic conditions developed as a result of dedolomitization, others believe that clays liberated during the reaction play an important role.

A second type of problem aggregates have been recognized in Iowa on the basis of concrete service records. They are dolomitic limestones or calcitic dolomites which pass all current acceptance tests, dedolomitize in alkaline environments, do not expand and cause deterioration in concrete within five to fourteen years after paving. Research on such rocks indicates they form negative type reaction rims and that physical and chemical changes occur both in the aggregate and associated concrete matrix. Lemish in his studies on the aging of concretes utilizing such aggregates has been investigating the aggregate-matrix interaction to see how it may be indirectly related to concrete deterioration.

A common association found with both types of aggregates are the reaction rims which occur peripherally on the coarse carbonate aggregates. These rims are the result of the interaction of the constituents of the carbonate rocks with their alkaline environment. Two kinds of rims have been recognized to date on the basis of acid etching are called the silicified and negative rims. The silicified rim shows a raised peripheral relief at the margins of the coarse aggregate when etched in acid. Such rims occur as a result of the interaction of silicate material with the hydroxyl ions to form a soluble form of silica. The silica is considered to be locally derived from the indigenous material in the rock. The negative rim results from the increased solubility during acid etching which results from dedolomitization. Illustrations of various rims can be seen in Katharine Mather's panel presentation on the "Waterways Experiment Station Experience with Alkali-Carbonate Reaction". The manner in which such rims effect the concrete matrix-coarse aggregate bond is considered a potential way in which the aggregate may effect concrete performance. The significance or contribution of these rims to the performance of the aggregate in concrete is a fertile area for future research.

In summary, carbonate aggregates react in order to adjust to the alkaline concrete environment. In some instances the reactivity of certain carbonate rocks is related to concrete deterioration. An expansion test in which cylinders of aggregate are soaked in NaOH solution has been devised which can efficiently recognize potentially expansive aggregates. The test cannot, however, differentiate any other type of non-expansive deleterious behavior such as in the case of Iowa problem aggregates.

TEST METHODS USED IN INVESTIGATING ALKALI-CARBONATE REACTION

by Katharine Mather

Ladies and gentlemen, Howard Newlon has been good enough to lend me this slide *(Slide 1 from Newlon) which summarizes the test methods used in investigating the effects of the chemical reactions that Mr. Hadley has just described upon concrete, mortar, and the rock in question.

Let us begin with ASTM Designation: C 157, Method of Test for Length Change of Cement Mortar and Concrete. One of the methods frequently used to pin down an expansive reactive carbonate rock is to make concrete bars or prisms using the suspect coarse aggregate and place the concrete in moist storage. Before the bars are put into the curing room or tank, the reference length of each is determined in the length comparator. Subsequently, with increasing periods of moist storage, the length changes are determined. There is good agreement in trend of results between the concrete expansions developed in C 157 test and the rock expansions developed in the rock cylinder test.

Another method, ASTM Designation: C 227, does not seem to me to be very useful in this application. The suspect aggregate is made into sand and tested in mortar bars containing high-alkali cement, which are stored over water. Length change of the bars is measured periodically. The volume of the largest sand grain in the mortar bar is frequently so small that volume changes in dedolomitization are not large enough to produce unequivocal evidence of expansion. This happens to be my personal judgment of the usefulness of the mortar-bar test, which has many other applications. Some of my colleagues may not agree with me.

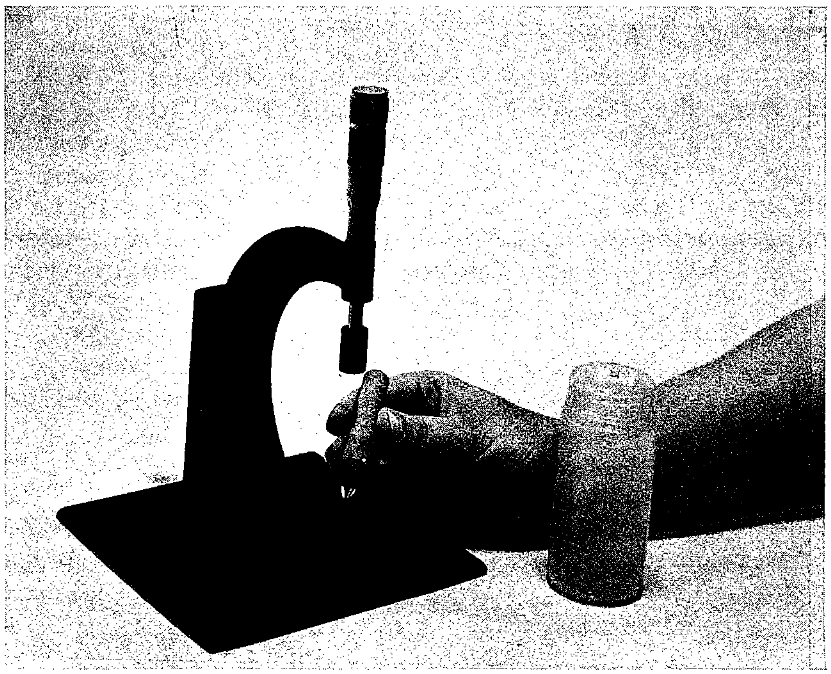
ASTM Designation: C 295, Petrographic Examination of Aggregates for Concrete, offers the opportunity to detect the characteristic texture of the expansive rocks, which has been mentioned. The rocks which form negative rims are habitually enormously fine-grained highly calcitic rocks. Both kinds of texture can be easily picked up in petrographic examination.

Howard Newlon has recently been chairman of a subcommittee which has drafted a rock cylinder test accepted by Committee C-9 on Concrete and Concrete Aggregates and now published as ASTM Designation: C 586. A number of us have used it, and I am sure that the published method will prove valuable.

<u>Slide 2</u>

This shows the small length comparator in which is seated a small drilled core. The ends have been ground down so that you can mount the core in the comparator. The length of the test specimen is 1-1/2 in. This is, I think, what most of us have been using. It is a very simple and rather neat procedure.

* Slide not available for publication.



SLIDE 2

We have a small core drill, and when we are screening rocks for carbonate reactivity, we usually decide which rocks to test after we have determined the calcite-dolomite ratio by X-ray diffraction using the method described a few years ago in the American Mineralogist by Tennant and Berger. We usually drill two cores, companion specimens, of each rock type. We happen to put flat ends on our specimens. We then soak them 72 hours or so in distilled water until the length has equilibrated. The cylinders are then stored in one normal sodium hydroxide solution in small polyethylene bottles. We use about 40 milliliters of one normal hydroxide for each prism. The length changes, if any, are subsequently determined at selected intervals with a comparator. Almost everybody's comparator is similar to the one which you see in the picture, although they differ in detail.

This completes my mission of describing the test methods used.

CONCRETE PERFORMANCE AS RELATED TO THE BEHAVIOR OF CARBONATE AGGREGATES

by J. E. Gillott

The majority of carbonate rocks, if physically sound, make excellent aggregate for concrete. There are certain dolomitic limestones, however, which cause excessive expansion and cracking of concrete when used as coarse aggregate together with high alkali cement. This is an alkali-aggregate reaction quite different from the alkali-silica type. Studies of this problem started in Canada in the fall of 1955 when the Division of Building Research of the National Research Council was asked to investigate cases of expansion and cracking in recently placed concrete in the vicinity of Kingston, Ontario.

Field investigation at Kingston showed widespread evidence of expansion in concrete which took the form of pattern or map-cracking (Figure 1), buckling and closing of joints, and extrusion of joint filler in sidewalks. It was estimated that about 70 per cent of the concrete at Kingston was affected. Examination of recently placed concrete showed that these symptoms developed during the frost-free season. Some form of cement-aggregate reaction was suspected from the start.

First laboratory tests consisted of making concrete prisms using job materials and mix designs and exposing these to continuous fog-room conditions. The prisms employed measured 3 by 4 by 16 in. and contained reference studs inset at each end during fabrication. Measurements of length changes were made on a comparator which contained a micrometer at the top which measured to the nearest 0.0001 in. Expansions of the order of 0.1 per cent or more were measured after a period of a few months. Beams were also fabricated in which low-alkali cements were employed and the rate and degree of expansion were found to decrease with decreasing content of alkali (Figure 2).

Extensive tests with various combinations of different coarse and fine aggregates revealed that dolomitic limestone coarse aggregate was the expansive component. It was found that prisms cut from the rock and immersed in a 2 Molar alkaline solution produced expansions of the same order as was measured on the concrete beams.

The largest expansion was measured on the prism with the bedding planes normal to the length. In solutions with a reduced content of alkali there was a big decrease in expansion and none in water.

Studies showed that there were different degrees of reactivity between

* Inorganic Materials Section, Division of Building Research, National Research Council, Ottawa, Canada.

Figure 1 Pattern cracking in concrete sidewalk with extrusion of joint filler (Kingston, Ontario)

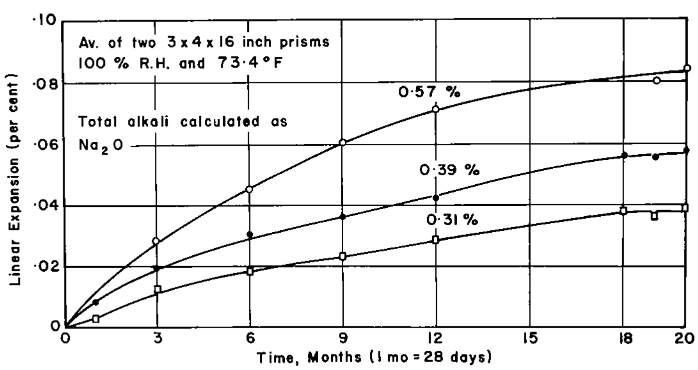


Figure 2 Decrease in expansion of concrete beams with reduction in alkali content of cement

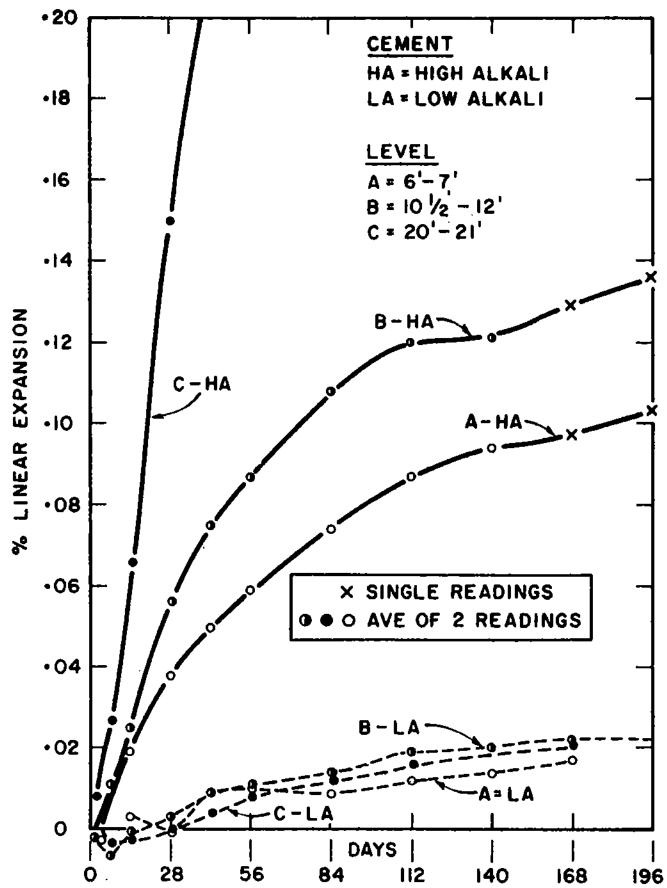


FIGURE 3

Differences in Rate and Degree of Expansion of Concrete using Aggregate from Different Beds of Rock in the Same Quarry the beds of rock within a single quarry (Figure 3) and between different parts of the same bed.

On the basis of this work it is concluded that the limit of 0.60 per cent on total cement alkali, where an alkali reactive aggregate is used, is too high for the Kingston rock. It is suggested that a limit of 0.4 per cent be accepted for this case (total alkali calculated as Na_2O).

Three field cases in which reactive aggregate and a low-alkali cement were used have been under observation by the Division of Building Research for about 8 years. In each case the concrete has shown no visible signs of distress.

The durability of concretes made with special low-alkali cement was investigated in the laboratory using aggregate from various beds in the quarries near Kingston. A freeze-thaw test extending over a 6-year period and a wetting and drying cycling test similar to the Scholer test were employed. The beams made with the low-alkali cement gave as good a performance as did companion beams made with aggregate of good durability history.

The alkali-silica reaction can often be controlled by use of pozzolanic material. Ten types of pozzolan were used in replacement for 25 per cent of the cement together with Kingston aggregate. Some pozzolans had an initial retarding influence but none had much effect at later stages of the reaction (Figure 4).

The main conclusions from this phase of the work were:

(a) the alkali-carbonate rock reaction is different from the alkali-silica reaction;

(b) it results from reaction between a special type of dolomitic limestone and the alkalis derived from high-alkali cement;

(c) moisture is necessary for the reaction; concrete beams held at a relative humidity of 50 per cent or lower showed practically no expansion. These beams showed expansion as soon as moisture was made available.

(d) Pozzolanic materials which control the silica reactions are not effective in this case;

(e) the reaction may be controlled by use of low-alkali cement but the total alkali content should be limited to about 0.40 to 0.45 per cent (calculated as Na_2O).

Regarding tests and recognition of potentially expansive carbonate rocks the main conclusions from these studies are that three procedures are most satisfactory for field use:

(a) Petrographic methods provide a rapid means whereby suspicious rocks can be recognized. The limitation is that quantitative predictions concerning the amount of expansion cannot be made. Furthermore some petrographic training is essential.

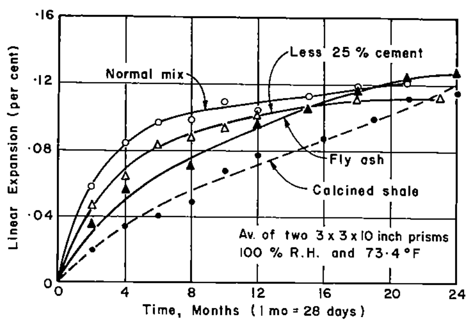


Figure 4 Failure of pozzolans to control expansion of concrete made with expansive carbonate aggregate

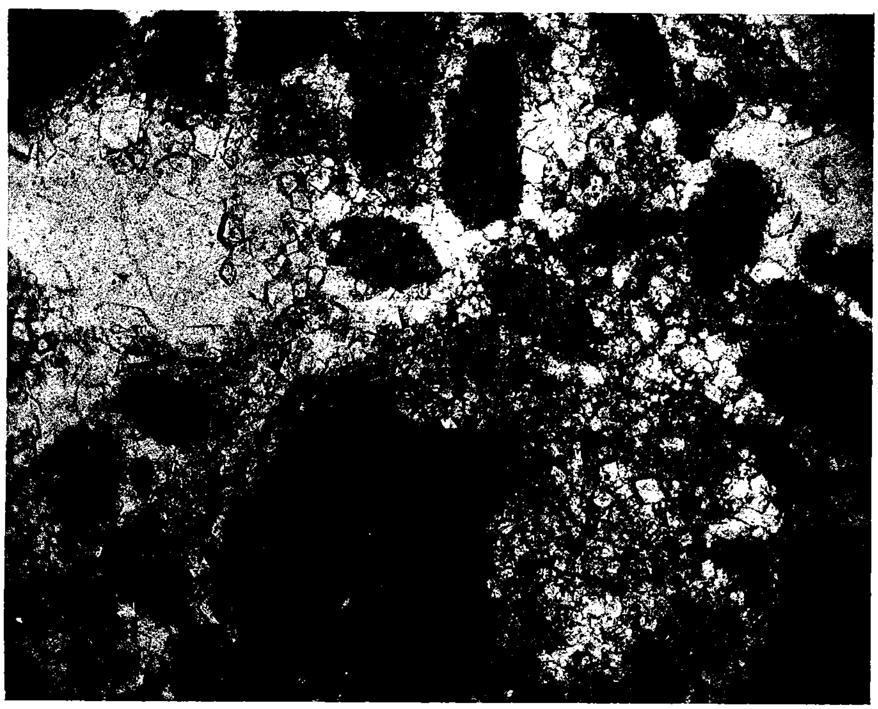


Figure 5 Photomicrograph of alkali-expansive dolomitic limestone, Kingston, Ontario

(b) The rock prism or rock cylinder test is fairly rapid, is easy to carry out, and may be set up with fairly simple laboratory facilities. Sampling must be adequate as expansivity varies from bed to bed of rock and between different parts of the same bed.

(c) The concrete beam test is the most direct method of evaluation. It takes somewhat longer than the rock prism test but sampling can be made representative. Moist conditions are required.

Other tests such as that employing a type of dilatometer in which powdered rock may be investigated and that employing a high precision optical extensometer have been used mainly as research tools in fundamental studies of the mechanism of the reaction.

Detailed petrographic studies have shown that most reactive rocks are very fine-grained dolomitic limestones which weather pale to dark grey. The dolomite is present as rhombic crystals which reach a maximum size of about 0.05 mm (50 microns). Most of the calcite is much finer and has a size of about 4 microns. Clay is invariably present in the acid insoluble residue (Figure 5).

In considering the recognition of potentially expansive rock and the mechanism of reaction most workers have assigned considerable importance to the dolomite. In an attempt to place the petrographic criteria for recognition of potentially expansive rocks on a firmer foundation, quantitative size analyses of dolomite crystals have been carried out on 40 rocks of varying reactivity. Crystal size distribution has been obtained microscopically by actual count. Wherever possible 300 crystals were measured in each slide. A graph was then constructed for each rock with crystal count plotted against crystal size in microns. From this data and the dolomite content of the rock the approximate dolomite specific surface was calculated.

The main conclusions from this work were that rocks with a specific surface greater than 100 sq. cm. per gram of rock nearly all showed excessive expansion. Rocks with a dolomite specific surface below 30 sq. cm. per gram of rock did not show abnormal expansion. Those with values between 30 and 100 sq. cm. per gram appeared to be borderline, some being expansive and others not.

The most important mineralogical change which takes place when fine-grained dolomitic limestone is placed in strong alkali is known as dedolomitization. The mineral dolomite is replaced by magnesium hydroxide (brucite), calcium carbonate and alkali carbonate. The solid products of the dedolomitization reaction occupy a smaller volume than the reactants. This reaction therefore does not account directly for the observed expansion. We have proposed that the dedolomitization reaction exposes clay and other fine-grained constituents which were included within the carbonates. This newly exposed material of very high specific surface takes up water which causes swelling. Cracks open up in the matrix and more clay and fine-grained constituents contribute to swelling in the same manner.

Strong evidence to support this hypothesis has been obtained by measuring the dimensional change of thin rock wafers, and powder compacts of rock and clay. Measurements were made by means of small optical extensometers which measure length change to 0.00005 inches. Each sample was measured directly in a special conditioning cell in which the environment ambient to the sample could be varied. The technique permitted much better control than is possible in the usual rock prism test. Samples were investigated under different relative humidity conditions, and when immersed in 2M NaOH. All experiments were carried out in a conditioned room at 21°C.

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2. J. E. Gillott, 1964, Mechanism and kinetics of expansion in the alkali-carbonate rock reaction, Canadian Journ. Earth Sciences, 1, No. 2, 121-145.

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CHEMICAL AND PHYSICAL REACTIONS OF CARBONATE AGGREGATES IN CONCRETE

by Howard H. Newlon, Highway Research Engineer

Last year (1965) Virginia was the 7th largest aggregate producing state in the United States and the overall quality level of our aggregate, of course, is quite good. This is attested by the fact that until two years ago the Virginia Department of Highways limit for loss in the Magnesium Sulfate Soundness Test was 8%. This maximum was increased, not because we had to, but just to bring ourselves into step with people around us. I emphasize this at the outset of this discussion for two reasons. First of all, in keeping with what Dave Hadley has just said, I think that you might get the perspective from what I am going to show you, that Virginia is underlain by a large quantity of very poor aggregates. This is not true. Thus, I am talking about the exception rather than the rule; although in certain cases, the exception represents a considerable volume of the aggregate used in a local area. The second reason for making this general statement is that the quality level of the aggregates that I will be talking about, even though they are chemically reactive, is quite good when judged by the usual criteria. The thin section of rock from Virginia that Dave Hadley showed is an excellent quality rock from a physical standpoint. The sulphate soundness loss is about 4 to 5 percent and the Los Angeles Abrasion Loss is about 15 - 17 percent. This certainly represents a good, high quality rock.

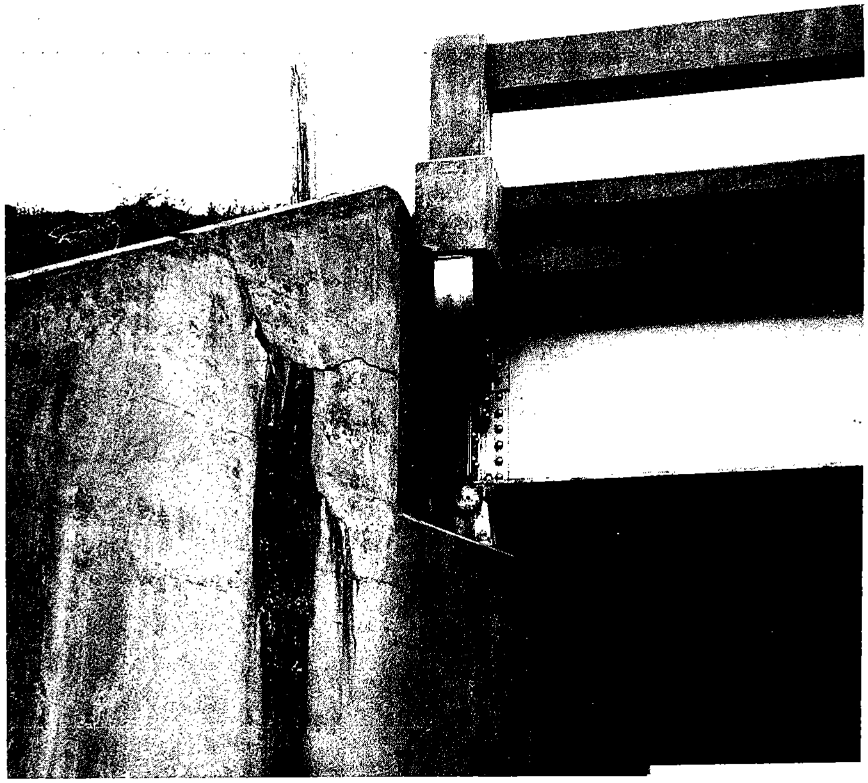
Slide 1

In discussing our work, I want to give credit to my associate Dr. W. Cullen Sherwood, a geologist, who in the past has participated in your meetings.

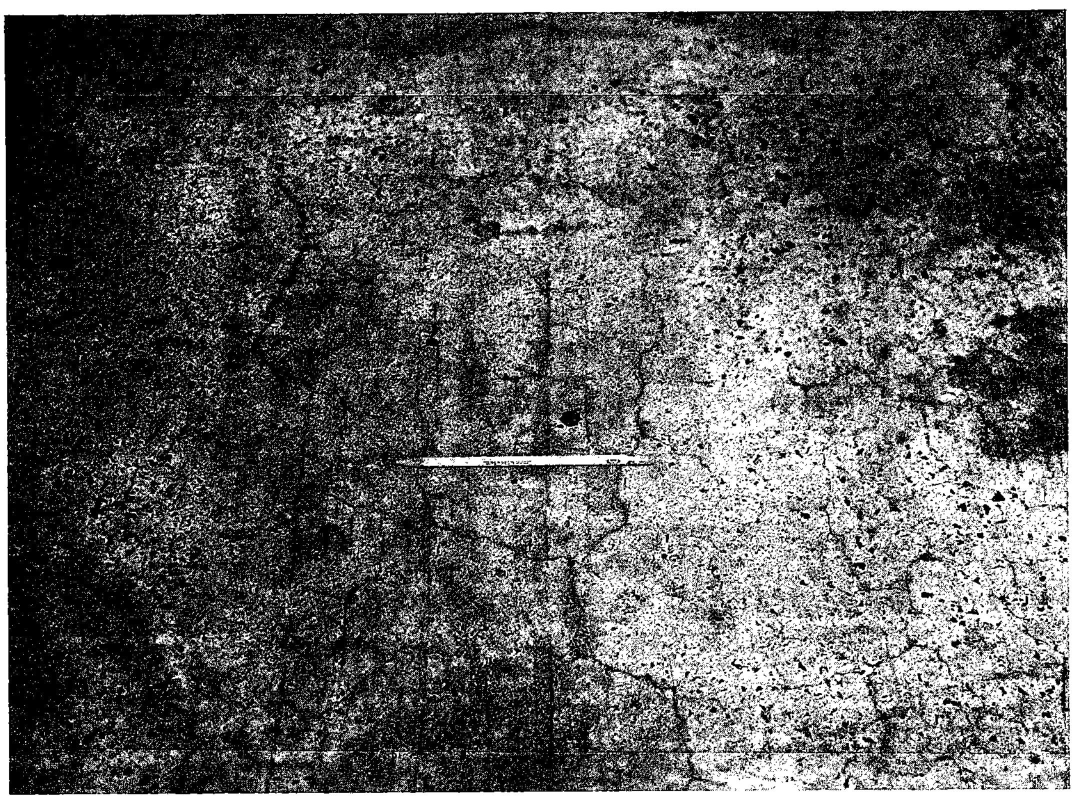
Our concern began about 5 years ago when we observed unusual behavior in some of our structures. The first slide will indicate one of these structures in which expansion is apparent. This structure is 15 or 20 years old and the deck expansion has proceeded to the extent that all the joints were closed so that the deck ultimately had no place to go to. It thus sheared off the back wall. This kind of expansion is accompanied by the typical pattern cracking shown in Slide 2.

Slide 2

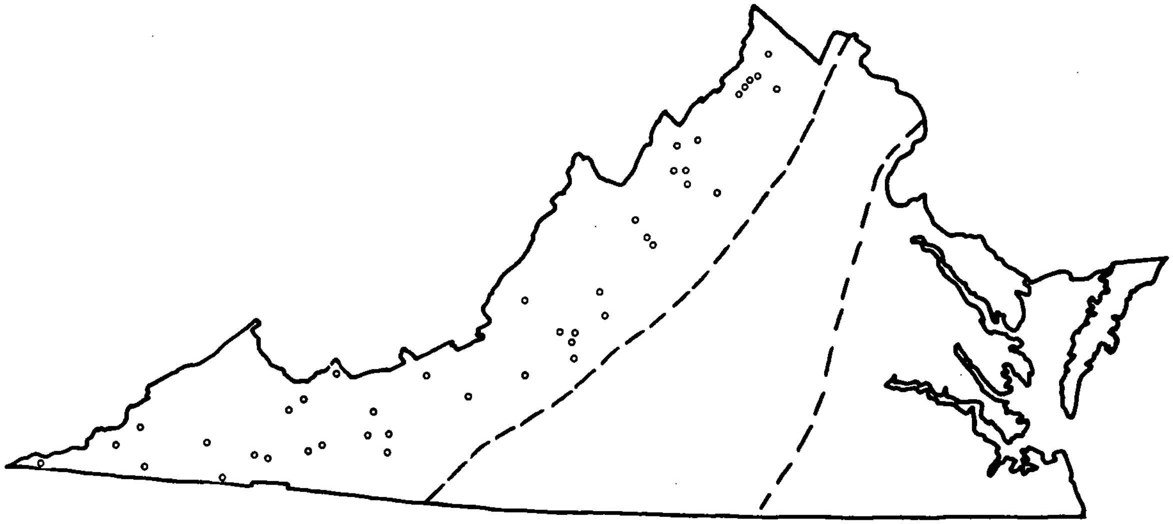
The structure shown in Slide 2 was only about 4 or 5 years old when this picture was taken. I might point out that we don't have any cracking as severe as that which has been observed in Kingston, Ontario. This is true in large measure, I think, because of the fact that we have a low alkali cement in the area where these aggregates occur. Our study of this behavior has been directed toward a number of facets which are very similar to those being studied in other places. First, we wanted to establish the reactive rock locations. We then wanted to determine the behavior of these rocks in concrete to develop some possible remedial



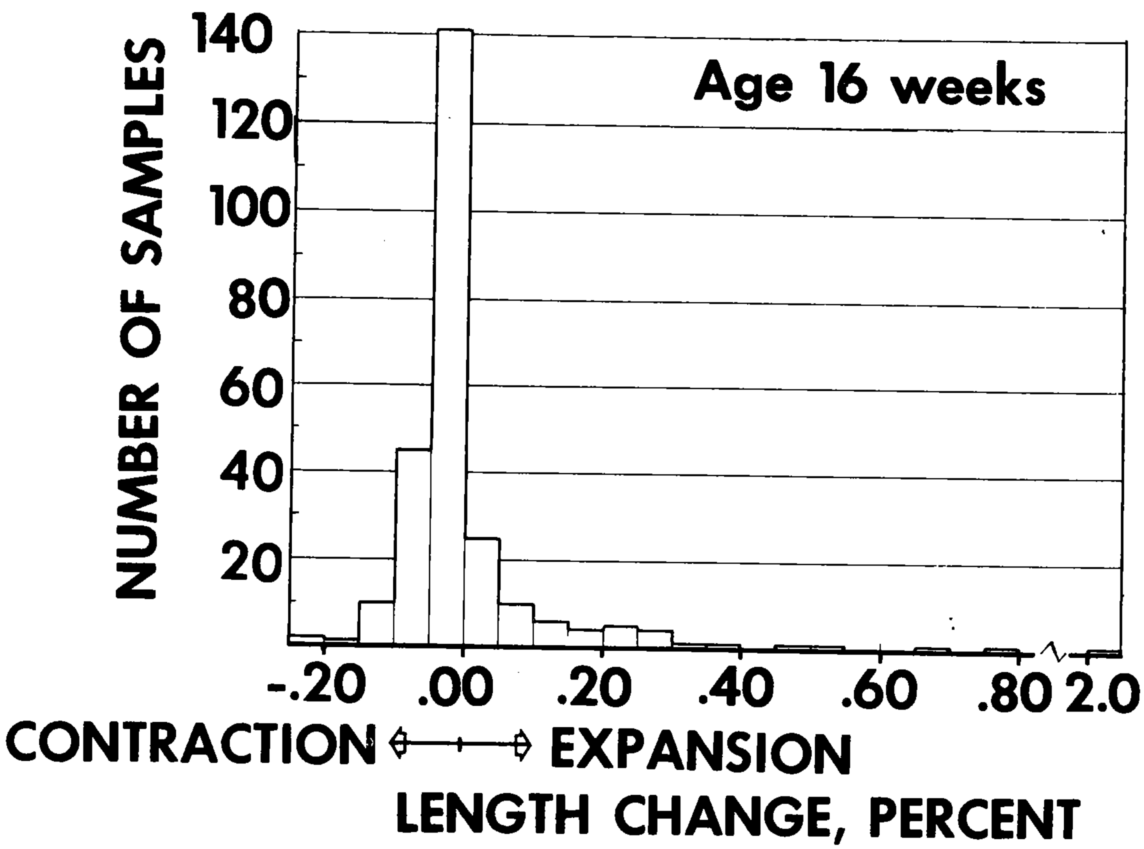
Slide 1. Failure of backwall, tilting of rocker, and pattern cracking of end post for structure containing expansive aggregate.



Slide 2. Pattern cracking of deck accompanying expansion of deck as a whole.



Slide 3. Carbonate aggregate sources in Virginia.



Slide 4. Distribution of prism expansions.

measures. In addition, we wanted to develop or evaluate some testing procedures in order to be able to evaluate specific materials and to understand the mechanism of the reaction. Obviously we wanted to relate all this work to the field performance. Because of the time available, I won't be able to talk about all of our work. A bibliography is attached.

Slide 3

Initially, the producing carbonate aggregate sources in Virginia were sampled. Slide 3 shows three major provinces and the distribution of carbonate quarries. There are in Virginia three geologic provinces and, of course, our carbonate quarries are in the Ridge and Valley area to the west. We sampled 45 quarries in the initial phase and obtained about 230 samples representing these quarries. Our preliminary screening was done on the bases of the rock prism test, chemical analyses, and petrographic examination. The distribution of the 230 prism expansions after 16 weeks of measurement is shown in Slide 4.

Slide 4

It will be noted that the majority of the specimens contracted just a little bit. The vast majority showed no expansion but there are a few which gave values that would be considered definitely expansive, and a few in an area which might be border line. After this initial screening, additional samples were obtained to represent a suite of 22 rocks which in the initial screening showed varying expansions and various textures. These rocks were more extensively studied, in thin sections, as rock prisms, in Mortar Bars (ASTM C 227), and in concrete bars.

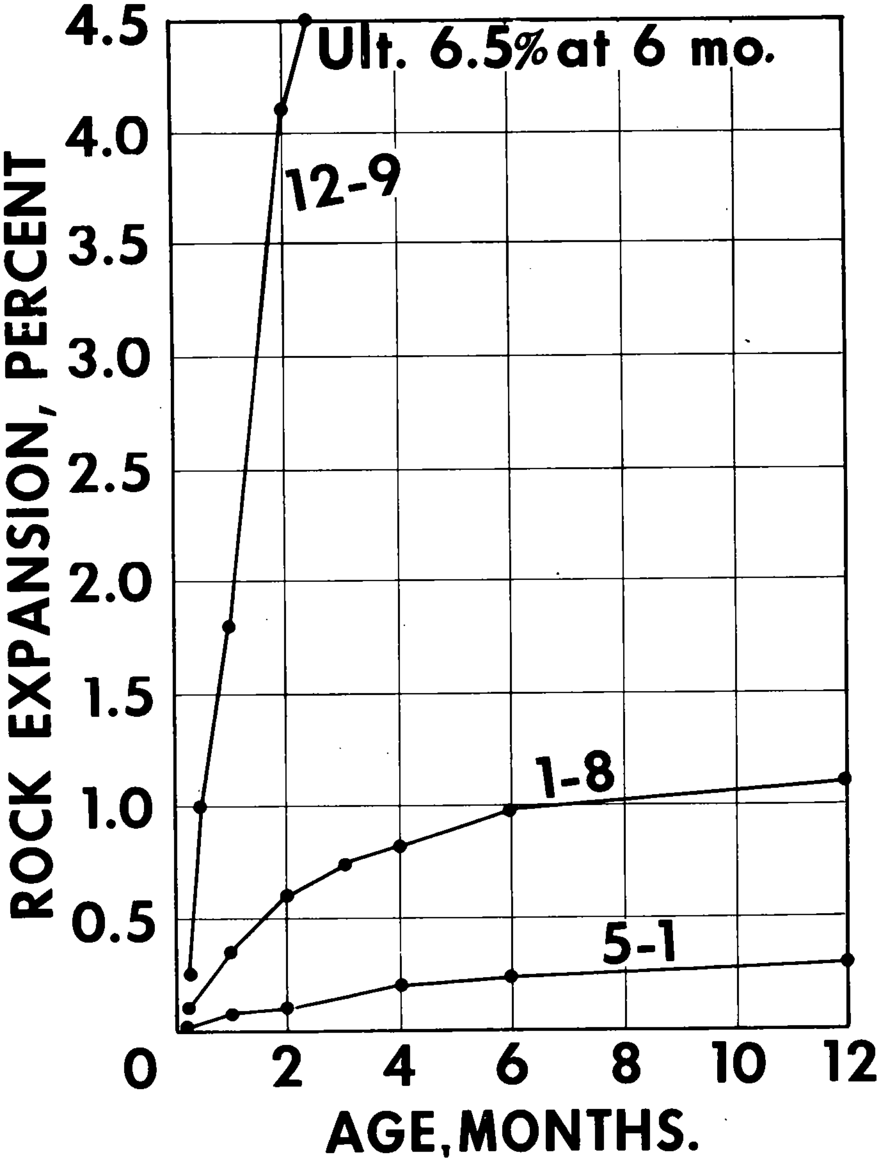
In addition, aggregate from one quarry which we identify as "1-8" was extensively studied in concrete to evaluate various remedial measures which might be employed if reactive carbonate rocks must be used. The important variables included in this phase of the study were:

1. Degree of aggregate reactivity
2. alkali content of cement
3. dilution of test aggregate with non-reactive material, and
4. storage conditions.

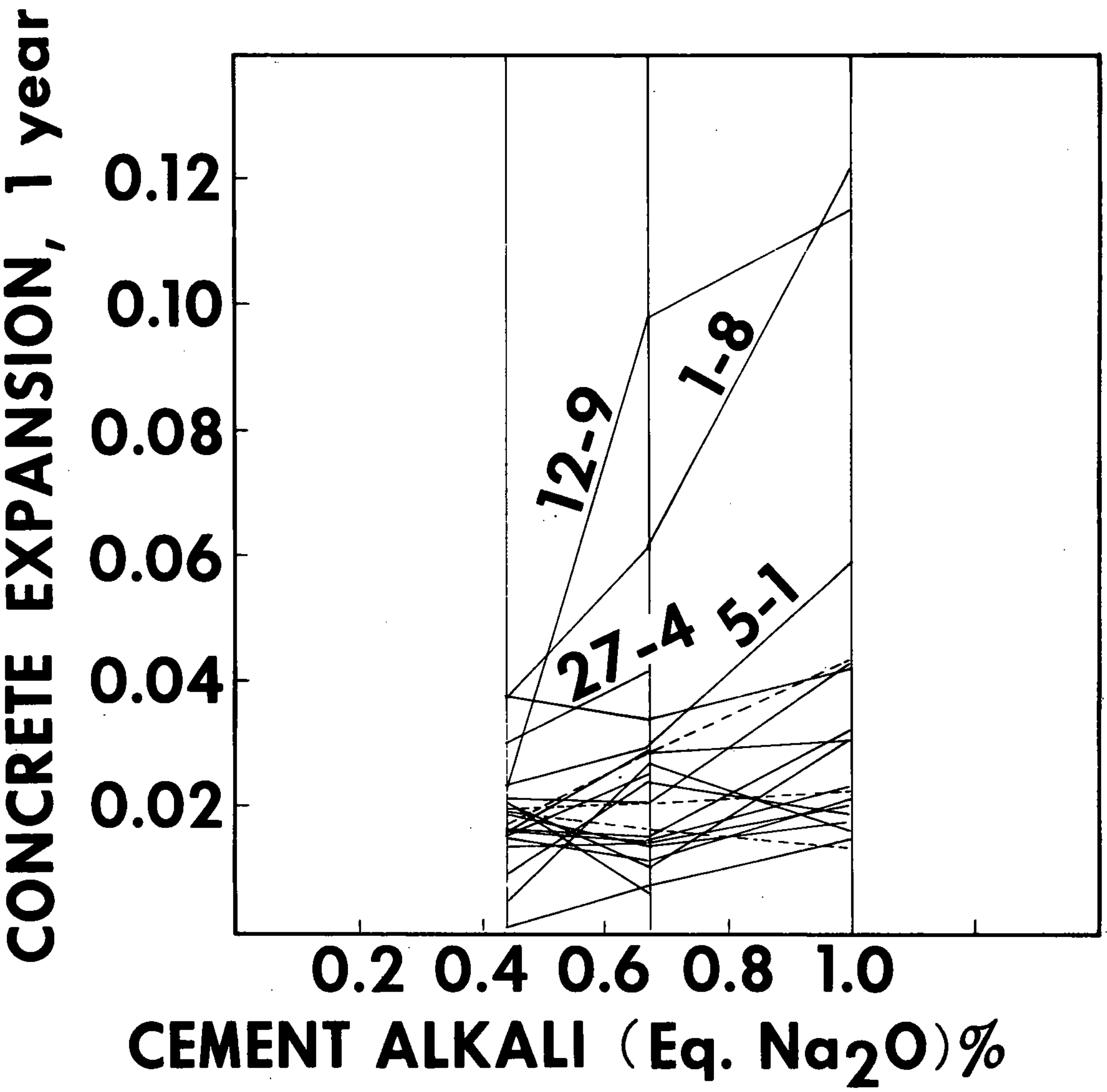
I will not cover all phases of these various studies, but will illustrate with typical results. The results of the studies of remedial methods are given in an earlier publication (Newlon and Sherwood - 1964). I will concentrate on the study which involved the suite of 22 rocks. As previously noted, these aggregates were studied mineralogically as well as in rock prisms, mortar bars, and concrete beams. In the mortar bars and concrete beams three cements were used with alkali contents of 0.95, 0.67, and 0.43 percent, expressed as equivalent Na_2O.

Slide 5

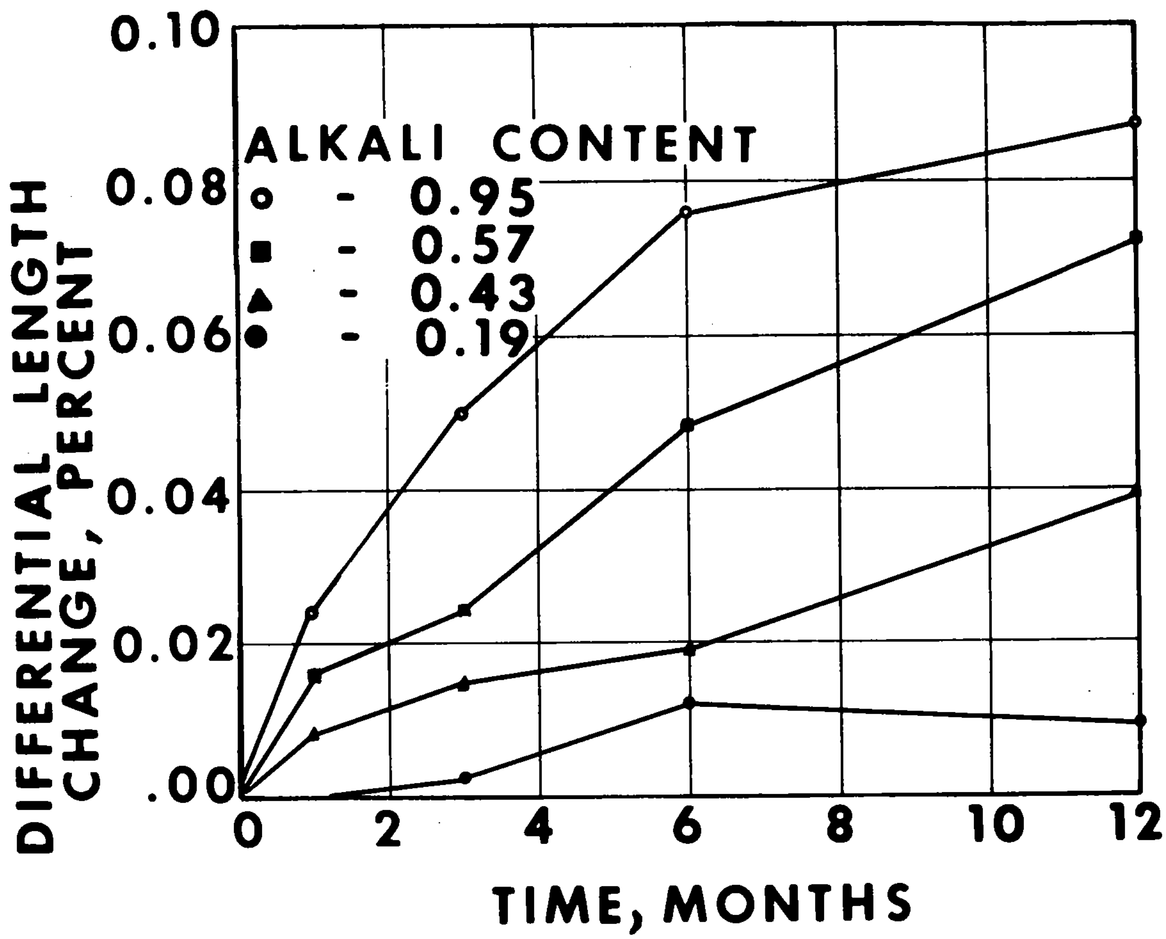
Let us first consider some typical rock prism expansions. Slide 5 shows the expansion of three aggregates as a function of time. Aggregate (1-8) is that which was used in the structures shown earlier and the one



Slide 5. Rock expansion versus time.



Slide 6. Concrete expansion versus alkali content.



Slide 7. Concrete differential length change with time, Aggregate 1-8.

with which we have worked most. It can be seen that it reaches a level of expansion of about 1 percent. The designations used throughout this presentation are devised so that the first number designates a quarry, the second a specific lithology within that quarry. The second aggregate, 5-1 expands about 0.4 percent. Aggregate 12-9 is extremely reactive. It takes off right at the beginning and expands to about 7% at 6 months. Even with this degree of expansion, it is not cracked at all. You can drop it from as high as you can reach on a concrete floor and the prism will not break. That it has sufficient elasticity to accommodate such an expansion is somewhat amazing in itself. That the behavior of the aggregate in the rock prism test is reflected in concrete, and that the expansion is a function of the cement alkalies (as was the case with the Kingston rock just discussed by Dr. Gillott) is shown in Slide 6. In Slide 6, expansion of concrete continuously moist stored for one year is given as a function of cement alkali content. The concretes* were made with the 22 different coarse aggregates.

All of the unlabeled concretes didn't expand significantly so that you can just group all of them as relatively non-expansive. But even for these concretes, the general trend of the expansion is an increase with increasing alkali content. Remember from Slide 4, that Aggregate 5-1 was one of the prism tests that expanded rather little. Aggregates 1-8 and 12-9 both expand increasingly with higher alkali content. The reason why 12-9 doesn't expand as much as 1-8 may be explained by some work we have been doing with restrained expansion, which I won't go into (Hilton - 1966). But note that the order of expansion is the same.

Slide 6

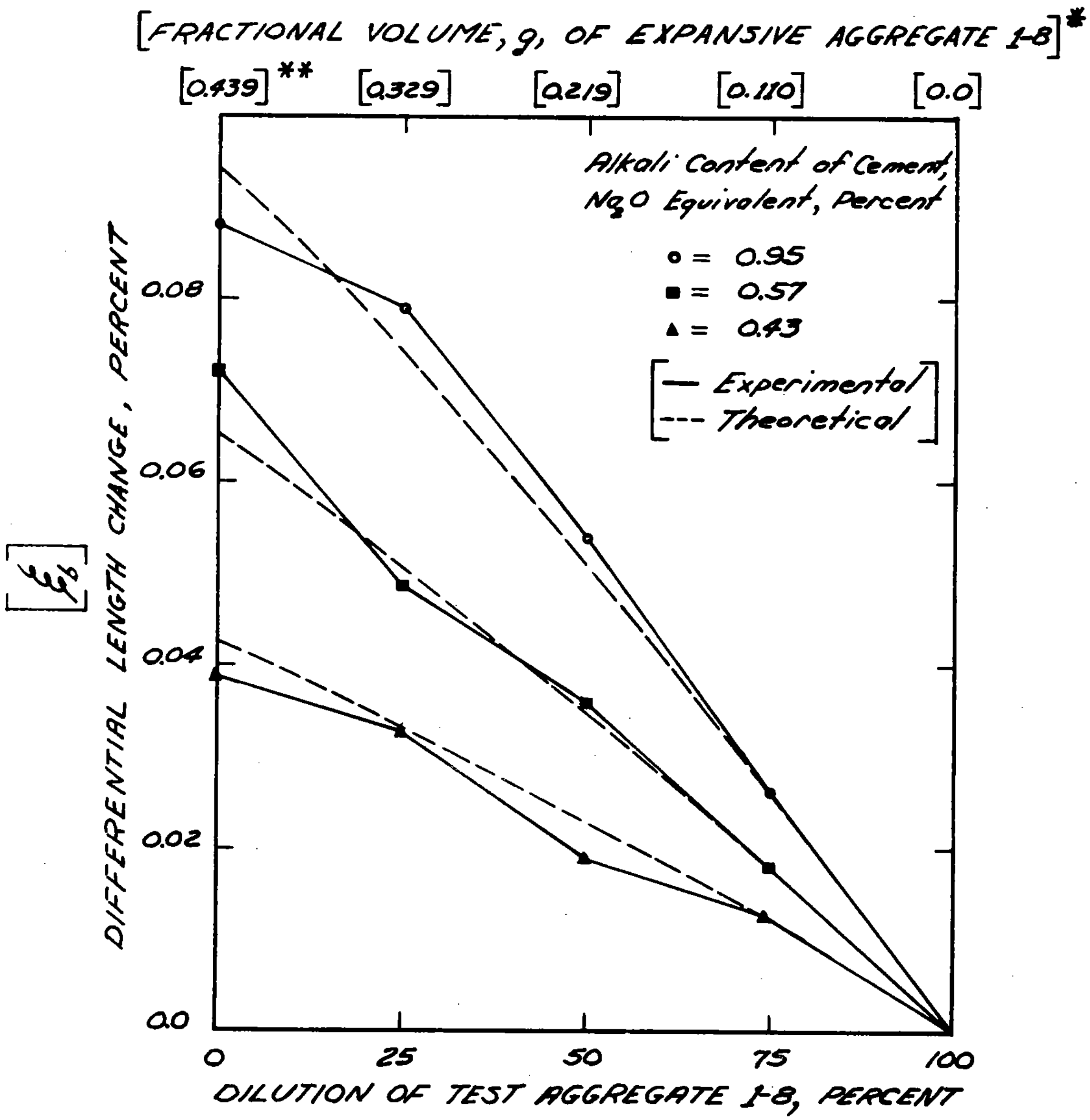
Slide 7

Now we will consider in somewhat more detail concrete containing Aggregate 1-8. Slide 7 shows the same kind of data that was shown by Dr. Gillott for the Kingston rocks; i.e., the length change in percent as a function of time, for concretes made with cements having four different levels of alkali content. These data indicate the obvious alkali-reactive nature of the reaction. The expansion shown is not the absolute expansion, but the difference between that containing the expansive aggregate and a corresponding concrete containing a non-expansive granite.

Slide 8

*Note: In all concrete referred to in this discussion the important nominal characteristics were as follows:

Cement Content, Bags/cu. yd.	6.25
W/C, By Wt.	0.50
Air Content, %	5
Slump, In.	2
Cement-Aggregate Ratio by Wt.	1.0:2.0:3.0
Maximum Aggregate Size, In.	1



Slide 8. Effect of dilution of Aggregate 1-8 on expansion at one year.

Slide 8 gives us, I think, some data that are encouraging. It shows that the length change is essentially a function of the fraction of the total volume of coarse aggregate which is the reactive Aggregate 1-8. The lines designated as theoretical were developed by an associate, Marvin Hilton, (1966) from an equation:

$$E_b = \frac{E_g}{E_g(1.5\frac{E_m}{E_g} \quad \frac{1-g}{3g}) + \frac{1-g}{3g}(1.5\frac{E_m}{E_g} + 0.5) + \frac{1+2g}{3g}}$$

E_m = Modulus of Matrix

E_g = Modulus of Aggregate

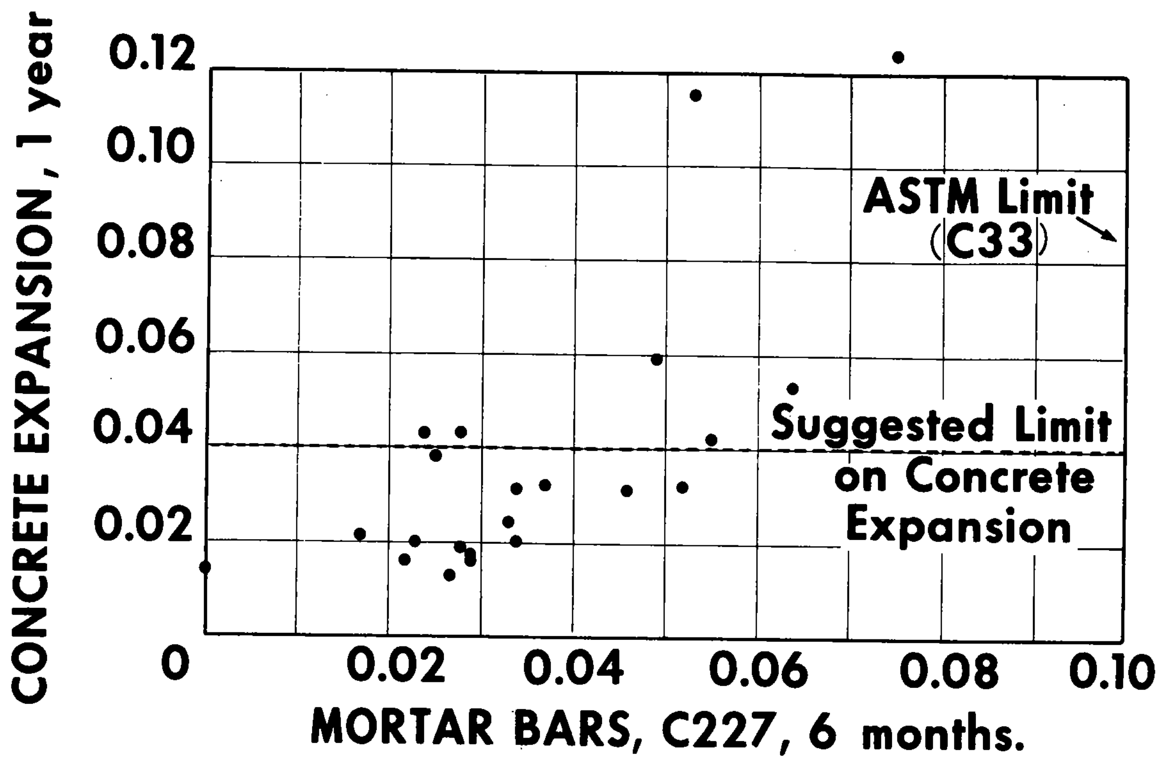
g = Fractional Volume of Aggregate

He predicted on the basis of this complex equation that the relationship would be as shown in Slide 8 and asked if I had any data. I gave it to him and behold it fits so well that I had to show it to you. In the concretes of Slide 8 the reactive aggregate 1-8 was diluted with a non-expansive reference aggregate. A void fraction of 0 would be entirely reference aggregate. The four curves represent the four different alkali contents shown previously in Slide 5. The important thing is that where expansion occurs, the reaction is related to the amount of the reactive material in the concrete. This is different, you will recognize, from what happens in the alkali-silica reaction where there is a "pessimum" amount of aggregate. It does point out the practical remedy that if you have a part of a quarry which contains reactive material and a part which is not reactive, you can estimate on a linear basis, we feel, how expansive the end product will be on the basis of how reactive and how variable the various ledges are.

Obviously one thing we must deal with is testing procedures to evaluate potential aggregate sources. In this brief presentation, I'll use as a basis the concrete expansion after one year of continuous moist storage and show the ability of these other test methods to predict this expansion in one year. We are of course attempting to relate this expansion to performance of concrete in field structures.

Slide 9

Slide 9 shows the relationship between the concrete expansion at one year and the mortar bar (ASTM C 227) expansion at 6 months. The alkali content of the cement in both the mortar and concrete was 0.95 percent. An expansion of 0.04 percent is what we have come to consider as the objectionable level that indicates laboratory expansion which we feel will lidely give us difficulty in the field. The mortar bar expansion of 0.10 percent is that which ASTM C-33 gives as the criterion for reactive aggregate based on alkali-silica reactivity. While there is a definite relationship between the concrete expansion and that of the mortar bars, the mortar bar expansions are all below the limits stated in ASTM C-33, even for concrete showing excessive expansion. I think that this has been borne out by other studies, so that while we perhaps could use the mor-



Slide 9. Concrete versus mortar bar expansion.

tar bar method, there are too many anomalies and the expansions are too small to make it very meaningful.

Slide 10

For the suite of 22 rocks discussed earlier, 300 lb. ledge rock samples were crushed to a maximum size of 1" for use in the concretes with varying alkali contents. Slide 10 presents the data relating the average expansions of 3 concrete beams made with high alkali cement to the average expansion of 6 prisms made from pieces randomly selected from the 300 lb. sample.

As you can see, there is a definite relationship between the expansions. However we're dealing with a fairly steep slope so that a small increase in prism expansion apparently indicates a correspondingly larger expansion as far as concrete is concerned.

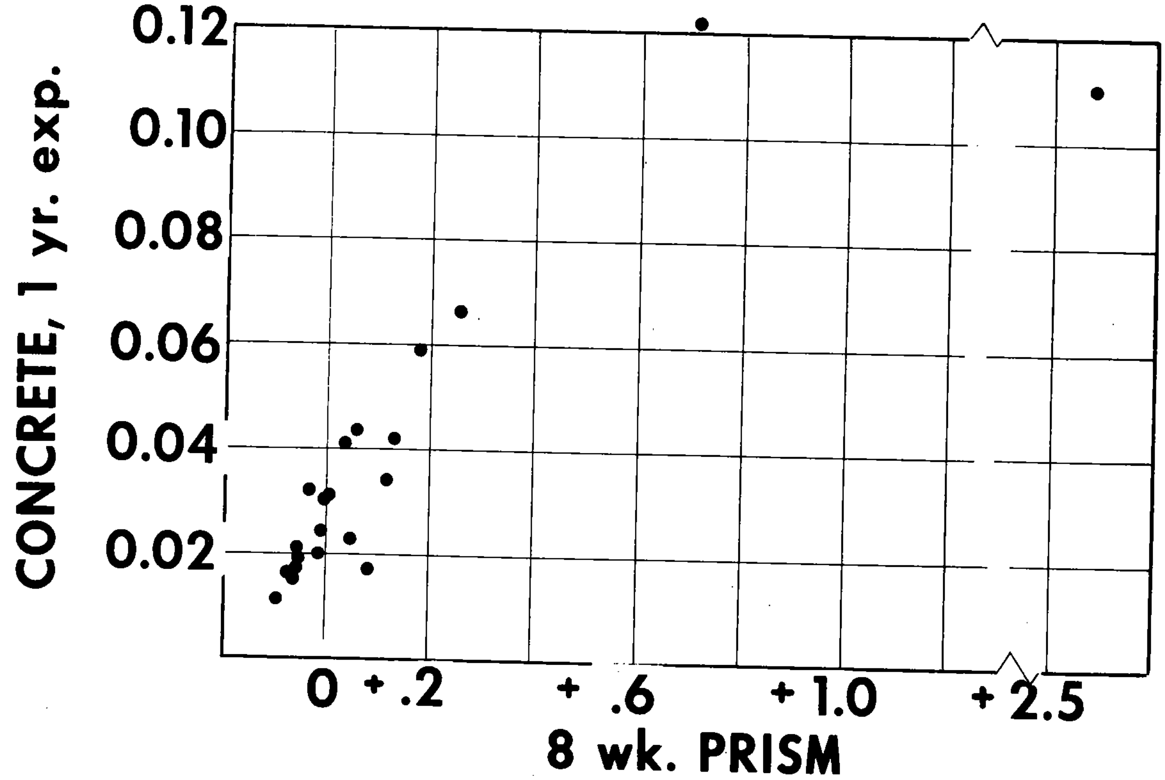
Slide 11

I would not want to lead you to believe that the expansions of the 6 prisms were the same. Actually, as shown in Slide 11, the range of the 6 prisms from the 300 lb. sample represents a significant spread but still the trend is up and even with the overlap, the tendency is evident. The highly reactive aggregates are also separated in a place by themselves.

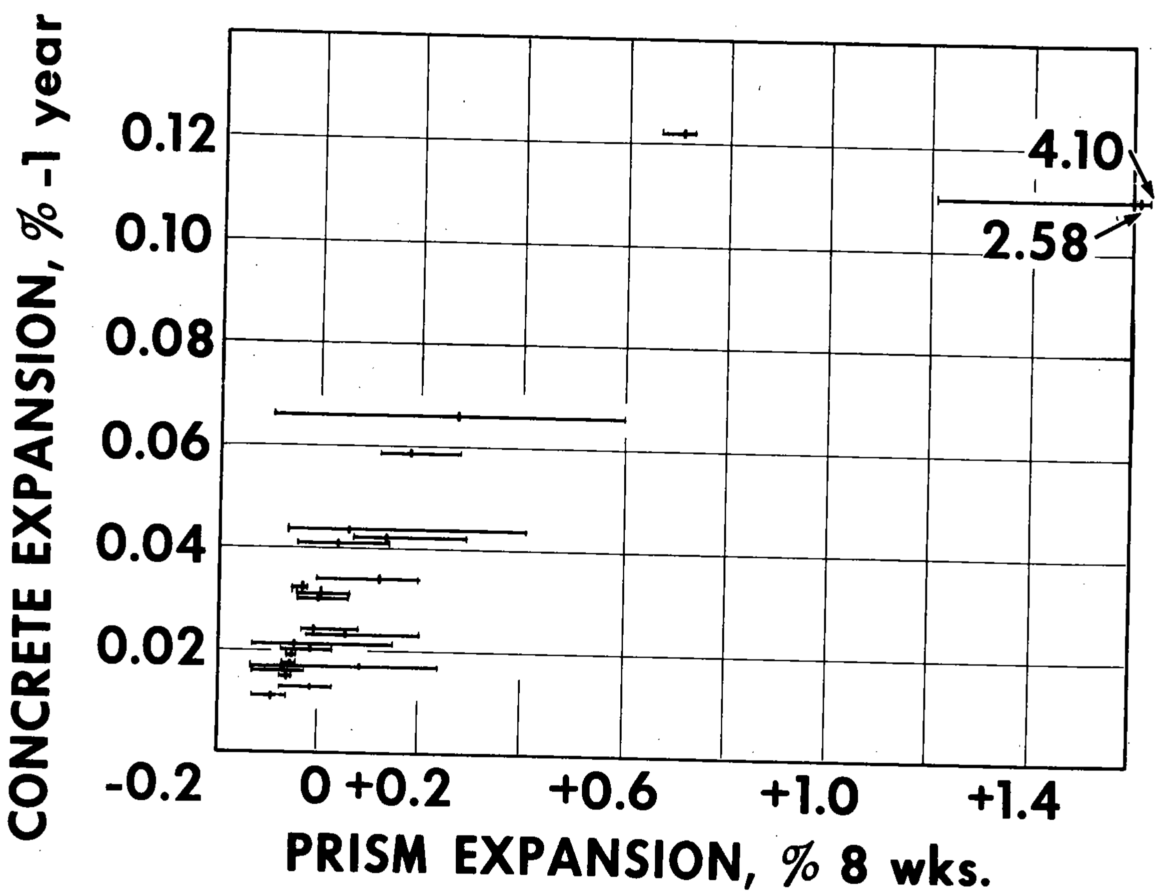
Slide 12

This then brings us to the last slide which I think relates to what I have discussed and also to the fact that I am speaking in the presence of so many geologists. What is the role of the geologist in all this? As far as the engineer is concerned, the role of the geologist, it seems to me, is to help us (1) get the proper sample and (2) relate the behavior of our samples to the rock structure that we find in place. All of the rock prisms shown in Slide 11 came from the same quarry. They came from a single core taken down the quarry face. As you see, they vary in color and are completely different rocks. The composition varies from a pure dolomite to a pure calcite. Obviously, to find our way out of this dilemma we've got to have a geologist to relate what we find in concrete to what we find in the quarry. That's why, I think, this common problem is the legitimate concern of this panel. This is where we need to come together because the engineer cannot face this problem alone. Obviously, the geologist has a large role. This, then, summarizes our experience in Virginia.

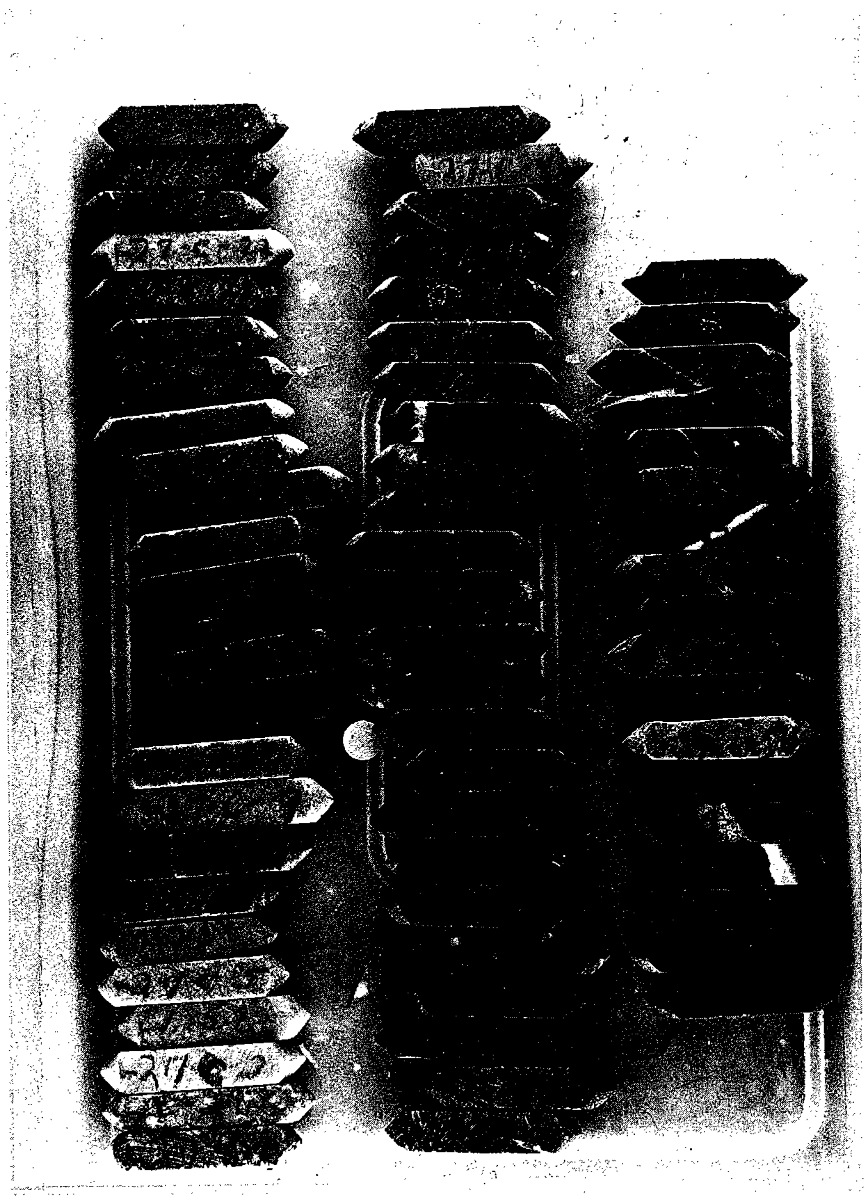
We now then come to consider a transition from a purely expansive reaction to one which combines expansion and other manifestations and bringing us that is Mr. Axon to discuss the work which he has been doing and the experience that he has had in Missouri.



Slide 10. Concrete expansion versus average expansion of six prisms.



Slide 11. Concrete expansion versus prism expansion.



Slide 12. Prisms cut from core taken from a single quarry. Note variation from top of quarry (upper left) to bottom (lower right).

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CONCRETE PERFORMANCE AS RELATED TO THE BEHAVIOR OF CARBONATE AGGREGATES

by E. O. Axon
Materials Research Director, Missouri State Highway Department

Mr. Chairman, ladies and gentlemen; the service performance of concretes, made with Missouri approved carbonate rocks, has been rated, with very few exceptions, from fair to excellent. Consequently, this discussion could concern either (1) the differences in the carbonate rocks which apparently caused the observed variation in service performance of concretes, or (2) the problem created by factors which appear to be acting to decrease the service performance of concretes made with previously approved carbonate rocks. Although discussion of the first topic could be interesting, discussion of the second would appear to be more timely.

For those unfamiliar with conditions in Missouri, some background information is desirable. First it should be stated that carbonate rocks, having a fair to excellent service performance in concrete, are readily available in a fairly large portion of Missouri. Although many of these carbonate aggregates contain chert or shale as contaminants, the acceptable amount of the contaminant has been fairly well established by service performance of concretes.

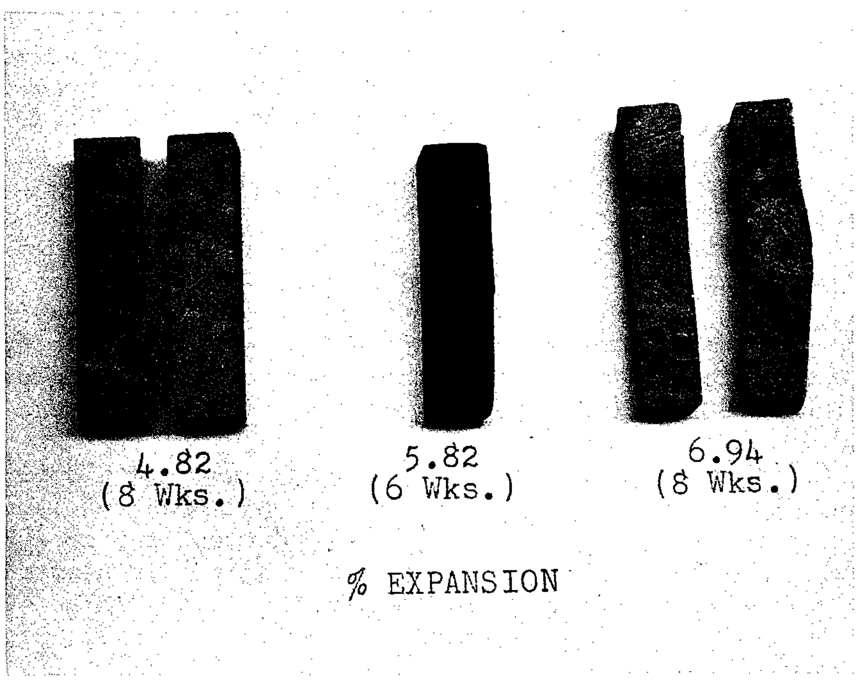
Secondly, there has been an increasing trend to produce coarse aggregate from local and previously unused sources, using portable equipment. As most of this locally produced stone comes from previously approved geological formations, it is usually not too difficult to determine whether the stone is comparable to that previously used. However, it is still extremely difficult to determine whether stone from previously unused geological formations is of adequate quality.

The preceding indicates that problems concerning approval of carbonate rock for use in concrete in Missouri are not too serious. Unfortunately, however, some factors now appear to be acting to decrease the service performance of approved carbonate rocks in concrete. These factors appear to be associated with the steadily increasing use of deicers. Not only does the bare pavement policy result in more frequent and more severe cycles of freezing and thawing, but increasing evidence indicates that one de-icer increases the rate of alkali carbonate reactions.

Although some cracking of concrete, which would not be attributed to alkali carbonate reaction, has always been observed; such cracking was rare, prior to the mid 1950's, and it did not appear to progress appreciably. Since general use of de-icing salts was started in the mid 1950's an increasing amount of cracking, which appears to be similar to that attributed to alkali-carbonate reactivity, has been observed in PCC pavements.

<u>Slide 1</u>

SLIDE 1



SLIDE 2

Slide No. 1 shows map cracking in a pavement approximately 8 years old. I would attribute this map cracking to alkali-carbonate reactivity. Note that the longitudinal cracks are more predominant than the transverse cracks. This may indicate that the pavement is under compression in a longitudinal direction and is only free to move transversely.

Although this type of cracking appears to be increasing and evidence indicates that use of sodium chloride does increase alkali-carbonate reactivity, this increased cracking has not as yet resulted in serious concrete deterioration.

In our paper in Highway Research Board Record No. 45, evidence was presented showing that some Missouri carbonate aggregates expand appreciably when exposed to alkali. This is evident by the appearance of some of the small rock prisms shown in slide No. 2.

You will note the obvious warping and cracking in some of these small rock prisms.

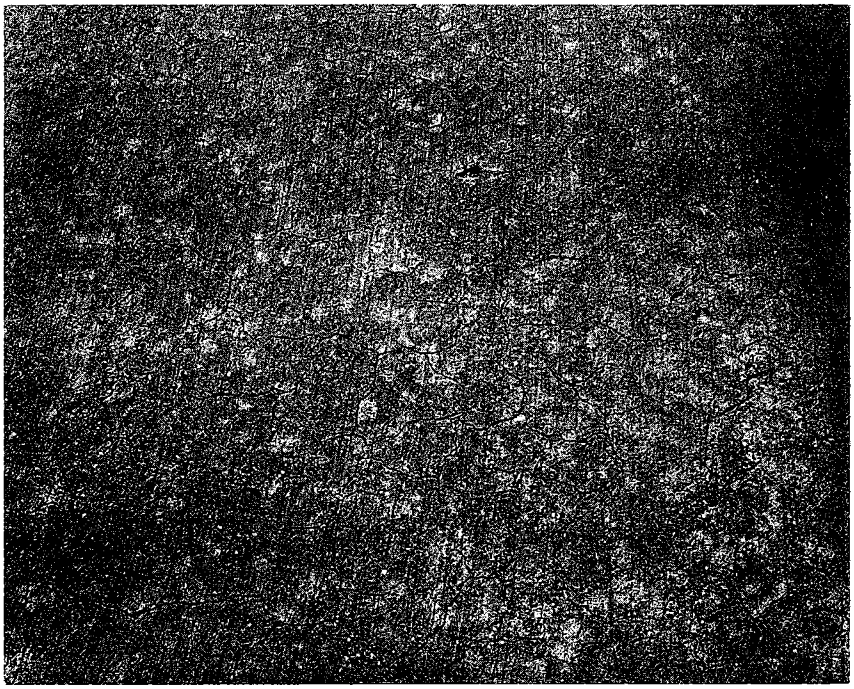
Slide 2.

Evidence was also presented in this paper of excessive expansion of concrete containing an unapproved coarse aggregate. The center prism in slide No. 2 is from this unapproved stone, and the effect of this aggregate upon concrete is shown in slide No. 3.

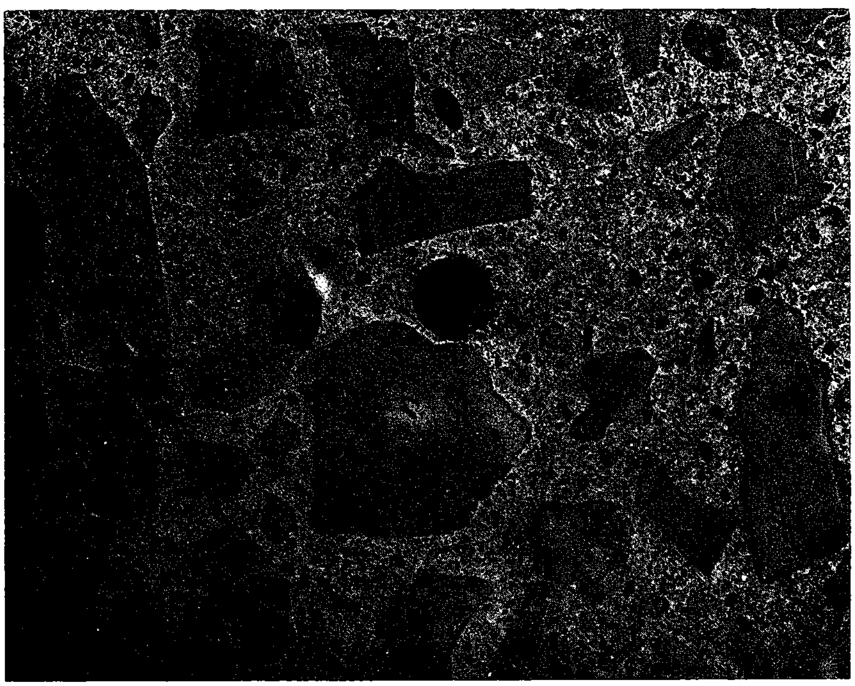
Slide 3

This is a picture of a sidewalk containing this unapproved aggregate. Our information indicates that this sidewalk was approximately five years old. So far this is the only source of Missouri stone where positive evidence of detrimental early expansion of concrete has occurred. The probable reasons, for lack of detrimental early expansion not occurring with other carbonate rocks, is that these other rocks have had a lower maximum reactivity or the stone as used contained an appreciably lower percentage of reactive rock. Very definitely some approved carbonate rocks contain some percentage of fairly expansive material. Of 857 ledges of carbonate rock tested for use in PCC pavements 41 have shown expansions of 0.20 percent or greater in the eight-week prism test. The rock in another 43 ledges exhibited expansions of 0.050 to 0.199 percent. Although it is most unlikely that stone from all of the reactive ledges have been used in PCC pavement, stone from a considerable number has been used. Actually many of these stones were not approved for use because of failure to pass normal acceptance tests.

The above indicates that nearly ten percent of the tested carbonate rocks showed 0.050 or greater percent expansion in the eight-week prism test. Whether this is a realistic estimate of the amount of reactive stone is difficult to determine; because other tests indicate that the reactivity of the rock in a ledge can be highly variable. In fact it is now known that the ledge (not approved), from which our most reactive sample came, also contains non-reactive rock. With such variability in reactivity within one ledge, it is most difficult to predict, without a more exhaustive sampling of ledges, the exact amount of reactive stone.



SLIDE 3



SLIDE 4

It is believed, however, that the amount of reactive stone used in PCC pavements in Missouri is fairly small. It is also believed that use of this small percentage of reactive carbonate rock would not seriously impair the service performance of concrete, if the concrete was not being subjected to an increasing number of applications of sodium chloride. As use of sodium chloride is not expected to decrease, there appears to be reason to be concerned about the ultimate effect of the use of reactive carbonate rock upon the service life of concrete in pavements.

In the preceding discussion concerning the reactivity of carbonate rock to alkali reference has only been made to the type of reaction resulting in expansion. Although this type of reactivity is the most spectacular in that it results in growth and cracking of the aggregate and concrete, it may or may not be the most damaging. The other recognized type of reactivity associated with carbonate rock is development of rims around the periphery of stone particles. Although the exact mechanism by which use of rim-developing rocks would adversely affect concrete performance is presently unknown, available information indicates that use of such rocks may have in some manner adversely affected the performance. If rim-developing rocks do adversely affect concrete performance, it does so in an insidious manner.

In Missouri little has been done in studying the effect of use of rim-developing rocks, except to observe that rims have developed on rock particles in some deteriorated concretes. These observations are illustrated in the next two slides.

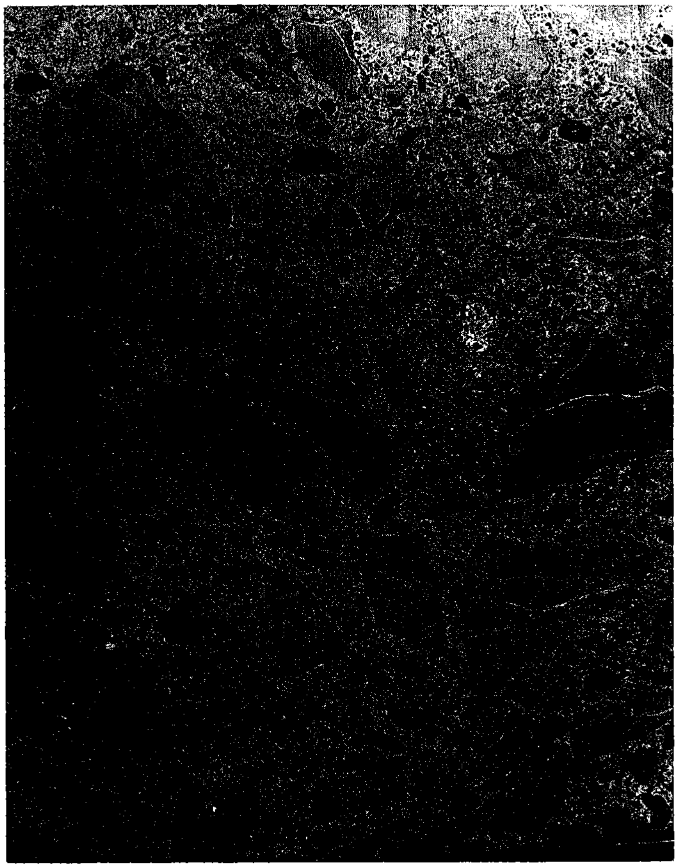
Slide 4

Slide No. 4 shows a slab from a core that we recently drilled from a bridge deck. Note that the dynamic modulus of this core was 5.7 million which is indicative of fairly good concrete. Yet you can see evidence of rims starting to develop on some aggregates. You can also probably see some faint white lines on some aggregate particles which indicates microscopic cracking.

Slide 5

Slide No. 5 shows a slab from another core from the same bridge deck. The dynamic modulus of this core was near 1.3 million which is indicative of badly deteriorated concrete. You can see rims on most of the coarse aggregate particles and the evidence of microscopic cracking is more pronounced. We do not know whether the rims caused the deterioration or the rims developed after the deterioration.

If use of rim-developing rocks is proven to be detrimental and the rate of formation of rims is proven to be accelerated by use of some deicers, this type of reaction could be more serious in Missouri than the expansive type of reaction. This is true because rim-developing rocks appear to occur in formations where expansive rocks do and do not occur. Consequently, the carbonate rock being used could contain varying amounts of both rim-developing and expansive material.



SLIDE 5

Presently, no effort is being made to control the use of either previously approved expansive or rim-developing rocks, because available information does not indicate that such control is necessary. On the other hand it is recognized that both expansive and rim-developing carbonate rocks are being used, and that their performance in concrete should be closely watched for evidence of increased rate of deterioration. In addition considerable work is being done to determine (1) the source and amount of expansive rock being used, and (2) the problems that would be encountered in controlling their use.

In conclusion the situation in Missouri can be summarized as follows: Both expansive and rim-developing rocks occur and are being used. Although evidence of an increasing amount of cracking in PCC pavements has been observed (possibly due to the expansive type of alkali-carbonate reactivity), no positive evidence of serious reduction in service life has been obtained. Nor has positive evidence been obtained that use of rim-developing rocks has seriously reduced the service performance of concrete. Recognizing, however, that changes in other factors may be acting to increase the detrimental effect of both expansive and rim-developing rock, considerable effort is being expended to prevent being caught entirely unprepared if control of use of reactive carbonate rock becomes necessary.

WATERWAYS EXPERIMENT STATION EXPERIENCE WITH ALKALI-CARBONATE REACTION

by Katharine Mather

The Corps of Engineers has used a great deal of carbonate coarse aggregate ordinarily with entirely satisfactory results. We have, however, considered that we owed it to the taxpayers to pay attention to deleterious reactions. Therefore, the Concrete Division, Waterways Experiment Station, in the past four or five years has been monitoring carbonate rocks, trying to develop some guidelines for the use of possibly expansive materials as a portion at least of the aggregate. Now to summarize our experience. We have been making prisms or cylinders of every carbonate rock that comes into the laboratory in which the composition of the carbonate portion is near to 50% calcite, 50% dolomite. We have painfully measured all of the cylinders. In our experience, which deals principally with the limestones of southern and central Illinois, Missouri, and Tennessee, we have not found an aggregate in this total collection which contained enough expansive rock to make us believe that there was any reason to avoid using it.

To show you the sort of thing that we have been doing in the recent past, we can look at several slides.

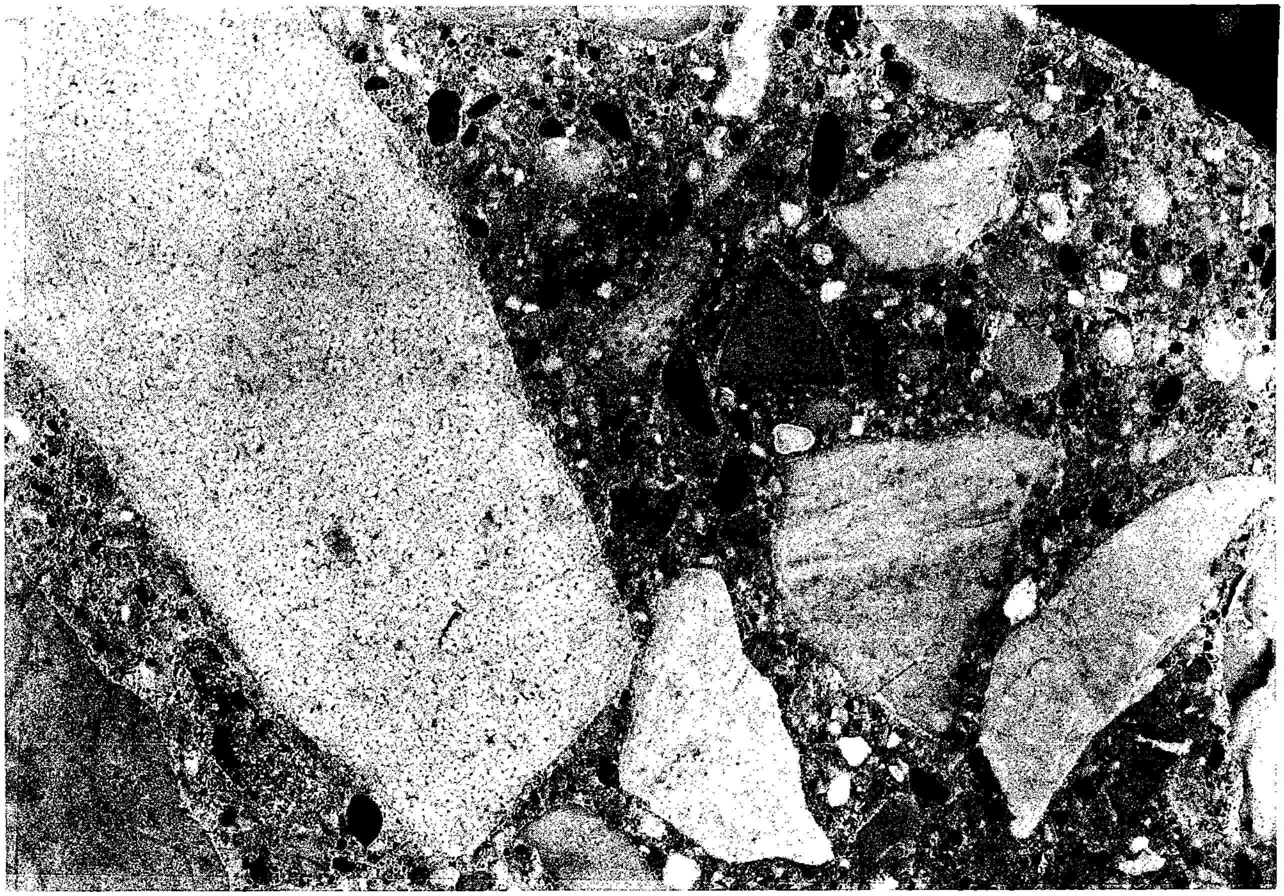
<u>Slide 1</u> - (Carlyle Dam. Narrow rim on dolomite particle.)

This is a slide of a core from Carlyle Dam. When the dam was about three years old, the structure was cored purely for the record, to see how good the concrete control had been. The cores were sent to us to do dynamic modulus and strength tests on the major portion of the cores and to determine the air content of hardened concrete on core ends. We also looked at the concrete as we did the air contents. This is a perfectly respectable air-entrained concrete with a carbonate coarse aggregate. The triangular particle here has developed a dark rim. That rimmed particle, based on what we know about the composition of this coarse aggregate, is a fine-grained, essentially pure dolomite. There was one variety of rock, not this one, in the aggregate when we examined it which expanded 0.06% in two years. This was insignificant expansion over a long time, and this lithologic type made up 6% of the whole aggregate. We regarded it as trivial. However, when we looked at our air-content specimens, we did observe that we were getting rims. Mind you, so far as we know, this is good concrete.

<u>Slide 2</u>

Another lithologic variety in the same concrete developed a double rim, shown here as a dark rim at the edge of the particle, and narrow white rim in the paste outside.

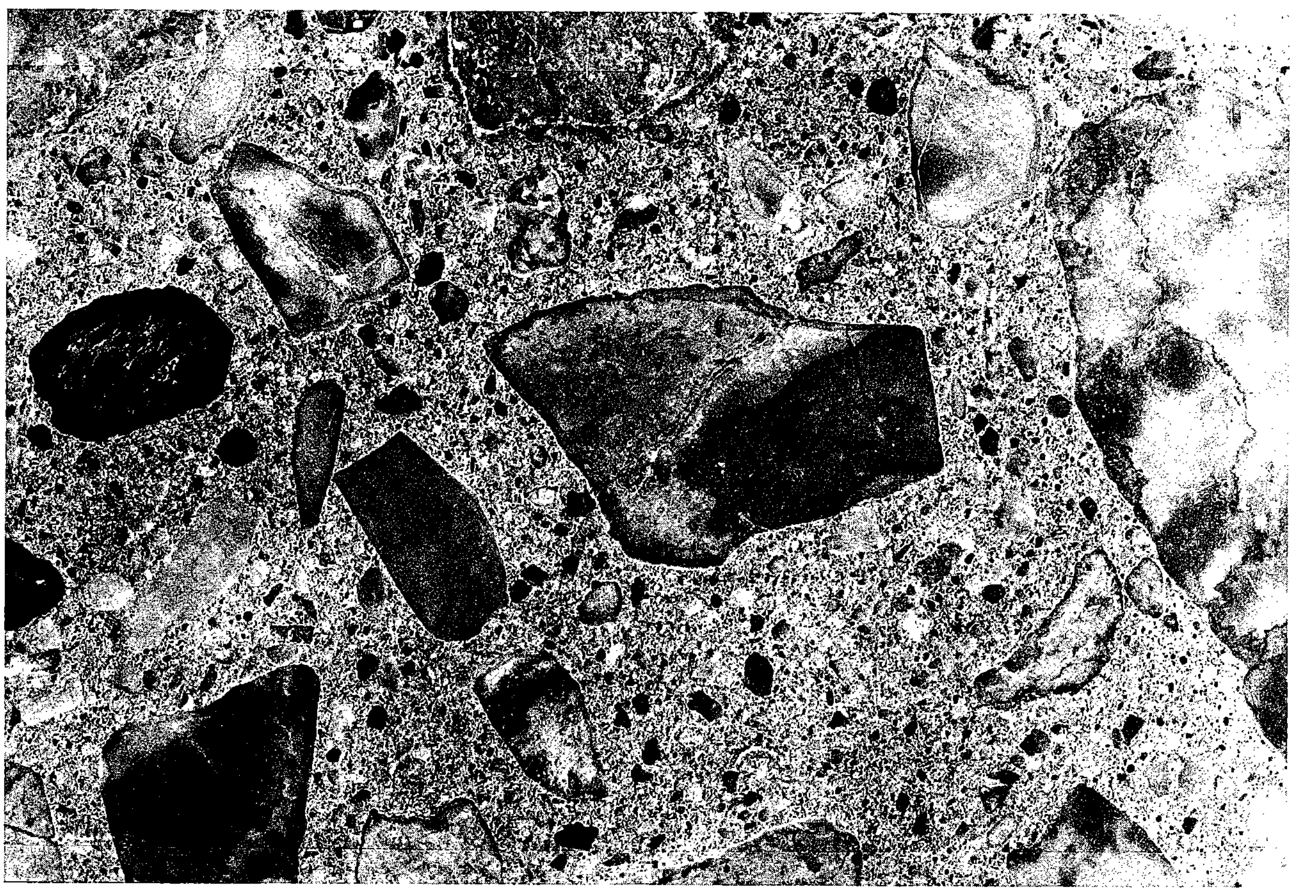
You can see that we have taken two cores with our little drill, used in this case to sample the rock for an X-ray diffraction specimen. It was a very slightly dolomitic limestone. It grew a rim.



SLIDE 1

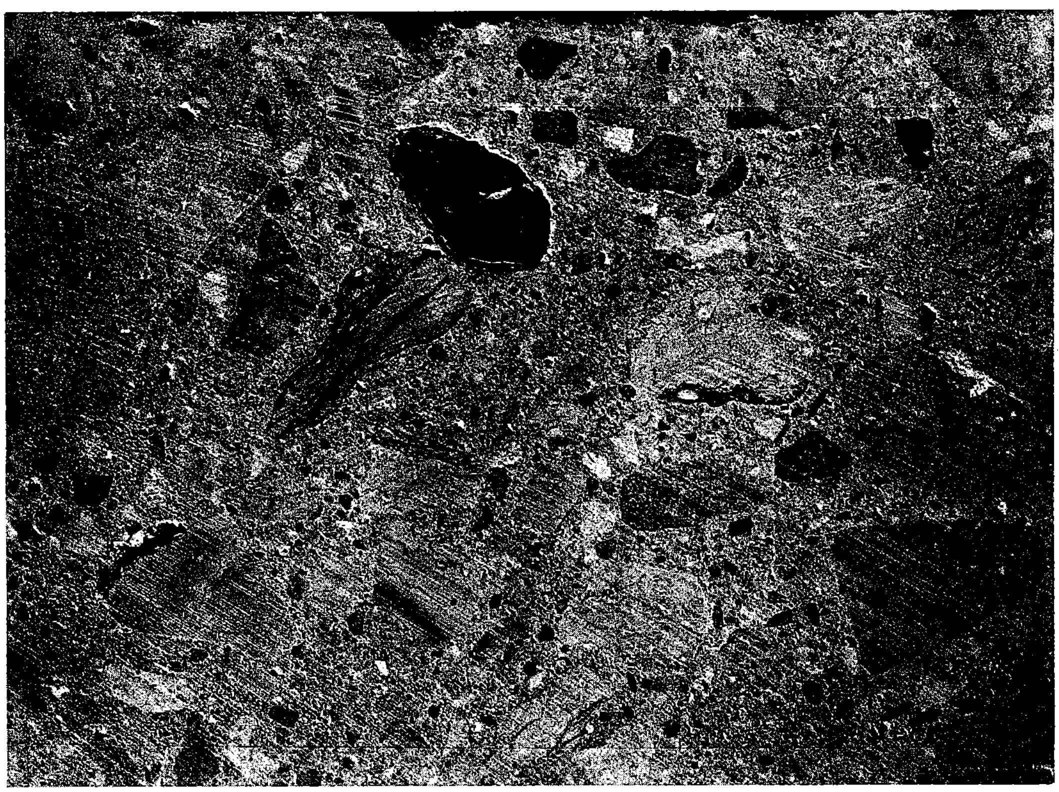
SLIDE 2

SLIDE 3

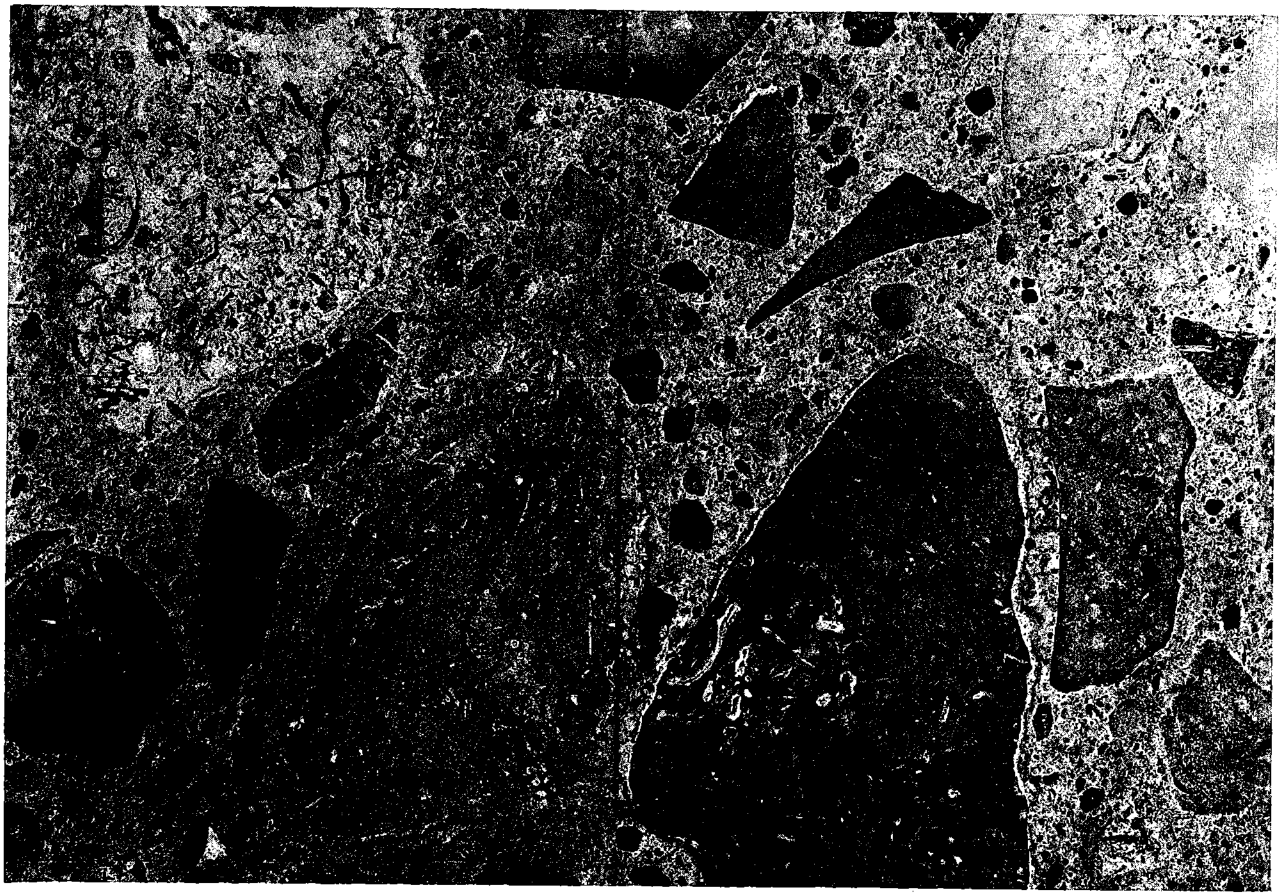


SLIDE 4

SLIDE 5



SLIDE 6



SLIDE 7

Slide 3

A third example from the same Carlyle Dam concrete but a different core. Rimming has taken place on two fine-grained essentially calcitic limestone particles with not very much insoluble residue and a little bit of dolomite. The concrete is in fine condition.

Slide 4

This concrete is a somewhat different story, related to what Dave Hadley was saying about D-cracking. There is an airfield pavement in the Middle West where D-cracking is developing along the joints and along some structural cracks. We went out there to get specimens to investigate the D-cracking phenomenon, and we discovered that we had also growth of rims on aggregate. The aggregate is the Bethany Falls limestone, which seems to form rims freely. We have examples from three quarries - one in Missouri, one in Kansas, and one in western Iowa. This particular core shows a small failure in the black shale particle at the far left. Probably it was wet when the concrete was placed, and when the shale dried, it contracted in the center.

The slide was 2X on the original print.

Slide 5

Here is a sawed surface less than an hour after sawing. To remind you that there are other problems in aggregates than those of carbonate aggregates, look at the black shale particle. As the specimen dried after the surface was ground, the shale shrank more and now is loose and rattles. The pavement surface is at the top of the photograph. This core came from an undeteriorated area. If the shale particle had been located nearer the pavement surface, it would have made what is called a "weather out" - a hole.

Slide 7

The slide shows additional rimming in more of the concrete, which is particularly apparent in the right side of the photograph.

I have shown you a few photographs of concrete from two structures, a dam and an airfield. The dam is, as far as we are aware, in mint condition. There is no reason at the present moment to anticipate that its future will be other than happy. The airfield is distinctly D-cracked. We are investigating that at some length.

I think that all of the rims I have shown you, except that on the dolomite particle in slide 1, are negative rims, more soluble than the central portion of the rock. Like Dave Hadley, I do not know their significance. I am unwilling at the present moment to conclude that this particular kind of rim is anything beyond a chemical phenomenon which takes place. I cannot currently think of a mechanism whereby this kind of rimming can become deleterious unless it facilitates the formation in the surrounding paste of some compound which is not cementitious at the expense of some compound which is cementitious. I have not thought of this kind of mechanism yet.

PHYSICAL TEST METHODS USED IN INVESTIGATING AGGREGATES

by Howard Newlon

We've talked about aggregates with particular emphasis more or less on chemical phenomenon. Obviously, since relatively few areas exist where we can build concrete completely free from freezing and thawing, it is sometimes very difficult to separate the things which are chemical phenomenon from those which are associated with freezing. A considerable research effort has been directed for many years towards being able to evaluate the frost resistance of aggregates when used in concrete and out of this have come a number of testing procedures. I know that you are familiar with these, but just as it was necessary for Mrs. Mather to enumerate some of the methods used in studies of chemical reactivity, I think it is appropriate here to list some of the methods we use in studying first resistances. Sub-committee III0 of ASTM committee C-9 on concrete and concrete aggregates, after considering this problem of aggregate evaluation several years ago, and for some period of time before, declined to recommend a standard testing procedure for evaluation of freezing and thawing aggregates in concrete. They declined for a number of reasons, primarily recognizing the inability of any of the existing methods to apply to the whole country. Although it was recognized that in certain local areas a number of different methods have been used and correlated with apparent satisfaction, the ASTM declined to adopt a standard test procedure for evaluation of aggregates. The report of the task group indicated that in the absence of service record data (we all recognize that this is the thing we need) and until a better procedure was developed perhaps the best thing to do was to freeze and thaw the aggregate in question in laboratory concrete. This procedure certainly has its shortcomings and has been a matter of considerable controversy related primarily to the severity of the freezing and thawing conditions or compared with natural environments. The most common methods which are employed can be classified under three broad headings:

1. freezing and thawing of concrete containing the aggregates.
2. unconfined freezing and thawing of the aggregate itself.
3. simulation of freezing and thawing through alternate drying and immersion in some sort of solution.

A variety of freezing and thawing methods for concrete are employed including ASTM C-290 which is a rapid freezing and thawing in water. ASTM C-291 is the equivalent method in which freezing in air and thawing in water, by rapid procedures. These methods usually permit about 8 cycles in 24 hours. The corresponding slow methods (one cycle per day) are still used but they have been removed from ASTM procedures, primarily because those people that use slow freezing and thawing were not using the ASTM cycle.

The aggregates in concrete are evaluated usually on the basis of loss of dynamic modulus, loss of weight, or increase in volume. The unconfined freezing and thawing of aggregates is conducted as in water according to AASHOT-103. I'm not too familiar with the details of methods of unconfined freezing and thawing of aggregates in water. Solutions of water-alcohol may be used as in the Iowa "A-freeze". Also, I believe, a sodium chloride solution is being used in the New York Department of Public Works. And then, there is always the Sulfate Soundness Method which I won't discuss, because of lack of time. Opposed to these more traditional methods of selecting and testing aggregates some of you have heard of research that has been going on in New York. We are very fortunate that Dr. James Dunn who has been associated with this work which we have been reading about with great interest is here to discuss it with us. Dr. Dunn now will talk about his work related to freezing of concrete and concrete aggregate.

DISTRESS OF AGGREGATE BY ADSORBED WATER

by James R. Dunn ***

Deterioration of Aggregate by Water

Many rock types which are unsound under freezing conditions deteriorate also by simple wetting and drying. Wet-dry deterioration is anticipated, for instance, for shales or shaly rocks, because of the expansion and contraction on wetting and drying of shales and clays (Mielenz and King, 1955).

In research at Rensselaer, when three fresh, Paleozoic shales* were soaked in water at ambient temperature and pressure and dried at 110°C, disintegration occurred in one to five cycles. Such disintegration can be expected to occur in any rocks which contain unmetamorphosed shale, such as shaly limestone, shaly dolomite or shaly graywacke sandstone.

What we did find surprising in the Rensselaer research was that similar wetting and drying of unsound**, argillaceous (non-shaly) dolomitic limestones and dolomites also produced failures in each case tested (figure 1). Expansion on wetting was also observed in the four cases in which expansion was checked.

Seeking to answer questions resulting from this deterioration of aggregate by water has required several years of research, and has led to development of testing and recording apparatus, which has led in turn to refinements of questions and refinements of apparatus. This continuing research is being done for the New York State Department of Public Works and the Federal Bureau of Roads.

Other researchers, stressing pore characteristics, mineralogy, petrography, and chemical reactivity have made some brilliant analyses. But we feel that there is still much to be learned about the reasons for the parallel behavior (deterioration) of argillaceous dolomitic rocks on wetting and drying and on freezing and thawing.

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* From the Normanskill, Esopus and Rochester formations.

** Soundness is defined by the N.Y.S.D.P.W. freeze-thaw test in 10% NaCl solution. Unsound rocks, for the present purposes, would have a backscreening loss of over 20%. Such unsound materials deteriorate rapidly in outcrops.

*** James R. Dunn & Associates, Averill Park, N. Y.

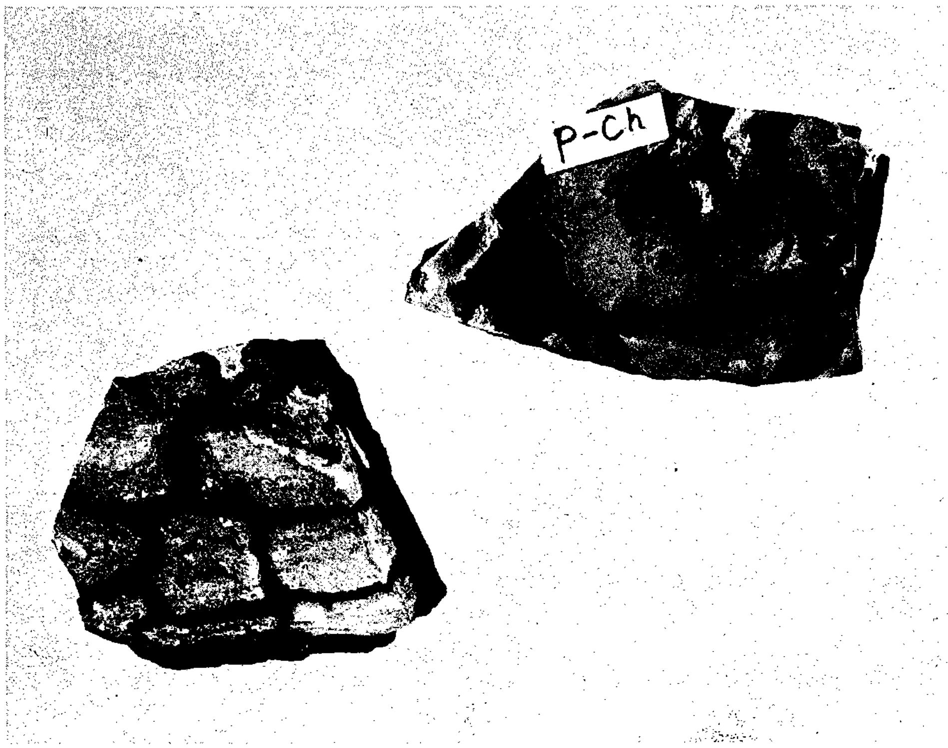
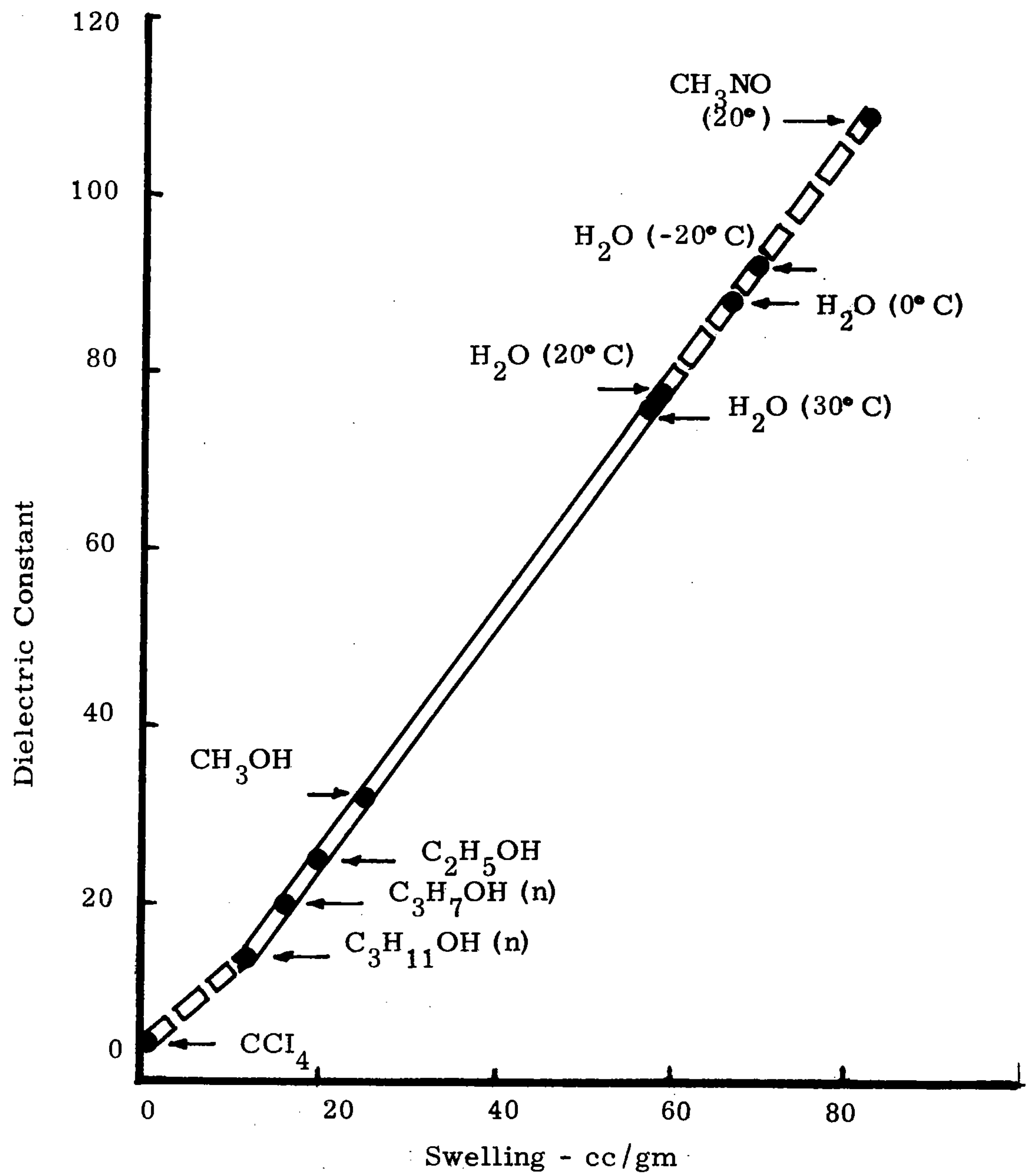


Fig. 1



SWELLING OF KAOLIN vs DIELECTRIC CONSTANT

Fig. 2

2 - C

FORMAMIDE

Fig. 3

Explanation of Wet-Dry Deterioration

The most generally accepted hypothesis to explain the expansion of clay on wetting is that the hydrogen ends of water molecules (being polar) hook on to the negative oxygen atoms at surfaces of clay particles and orient themselves (Hendricks and Jefferson, 1938). It has been postulated by clay mineralogists that such water has a higher specific volume than bulk water, is ordered, rigid, and, if confined, is capable of exerting considerable expansive force (for review see Low, 1961, and Martin, 1960).

Winterkorn and Baver (1934) demonstrated that the volume expansion is proportionate to the dielectric constant of the liquid which is taken onto-or adsorbed on-the clay surface; presumably the force of expansion is also proportionate to the dielectric constant. Expansions of kaolin-liquid systems versus dielectric constant for the liquids are plotted largely from the data of Winterkorn and Baver (figure 2). Because their points, connected by a solid line, show a clearly linear relationship, we have taken the liberty of extrapolating the data to water at 0°C and -20°C, and formamide at 20°C. The inference which we draw from this plot is that as a water-clay system cools and the dielectric constant of water increases, clay surfaces should be capable of taking on more rigid water. Therefore, presumably the bound water should be capable of exerting more expansive force.

If wetting by water alone is capable of distressing certain argillaceous, dolomitic carbonate rocks, then a more polar liquid, such as formamide, should cause more severe distress. Cores and slabs of unsound and sound carbonate rock - as defined by the N.Y.S.D.P.W. Freeze-thaw tests - were warmed for one-half hour in liquid formamide to 85°C and cooled for three and a half hours to ambient temperature at ambient pressure six cycles a day. Every frost-sensitive dolomitic rock deteriorated and in the same manner as it did in the freeze-thaw test (figure 3). Sound rock did not deteriorate. Two sets of control experiments on the same materials with non-polar carbon tetrachloride and dioxane caused no deterioration whatever.

The Rejection Texture

What is the reason for the wet-dry sensitivity of certain argillaceous dolomitic rocks?

We feel that the answer is in the petrology and petrography of dolomitic rocks. Dolomite crystals tend to be clear with the argillaceous impurities rejected to the grain boundaries. Excellent examples of this can be seen in Hadley (1964) writing in the H.R.B. Bulletin, and in Folk (1965) and Beales (1965) writing in the A.I.P.G. Special Bulletin Publication 13. Conversely the clays associated with calcite grains tend to be more evenly disseminated, less continuous and largely protected from wetting. The New Scotland limestone of the Devonian of New York, with up to 50% clay, is largely sound because the clays are apparently not wettable. The clay component is largely either completely enclosed in sparry calcite or evenly disseminated through an extremely fine grained micritic calcite.

The act of replacement of calcite by dolomite makes the clays more continuous and hence more available to water. We speculate that the force exerted against the pore walls by water around a clay particle is equal to the lithic head at the time of dolomitization provided the temperature is the same as that during the original dolomitization process (figure 4). At higher temperatures such force may become negligible because the dielectric constant of water decreases with higher temperature; conversely at lower temperatures the force should increase. Or, if a more polar liquid, such as formamide, is used the force may be great enough at higher temperatures to disrupt the rock.

Adsorption of Water

If water alone is capable of exerting force against pore walls, a large percentage of water in frost-sensitive carbonate rocks must be adsorbed, tightly held, surface controlled water. Adsorption measurements at 100% R.H. verified the hypothesis: in 17 of 19 frost sensitive carbonate rocks so tested, over 50% of their total possible water content was taken on in 100% relative humidity. Some took on as much water at 100% R.H. as they could under vacuum saturation conditions.

Most sound rocks, however, took on relatively less water by adsorption.

Water adsorbed on clay (or other) surfaces has one additional critical characteristic - much is not freezable, as shown by thermodynamic calculations, -from vapor pressures, -by heat of thawing - and by magnetic resonance measurements (Martin, 1960; Low, 1961; Sussman & Chin, 1966, among others). An immediate paradox is evident: The carbonate rocks with the most non-freezable water (adsorbed water) tend to be the most frost-sensitive!

Is Ice Necessary?

The following question quite naturally arises: Since ice is not a requisite for deterioration of shaly rocks or certain argillaceous, dolomite carbonate rocks, is it even critical? Since such rocks can be destroyed by polar molecules at temperatures above freezing, and the destructive water is adsorbed, rigid, tightly-held, low-vapor-pressure water, perhaps ice not only is not necessary for destructive forces, but perhaps in many cases it does not form at all.

To test this speculation cold quantitative differential thermal analysis equipment was constructed. The sensitivity is such that it can detect the freezing of 0.001% of water per minute in a 40 gram 3/4 inch core.

After about 5°C of supercooling, water froze suddenly in cores of all sound carbonate rocks. Nearly all of the water which could freeze froze at once in sound rocks. In unsound rocks a lower portion of the water froze. In 8 of 19 unsound rocks no freezing whatever was observed. Thawing - a more standard technique - also failed to reveal freezing in these rocks.

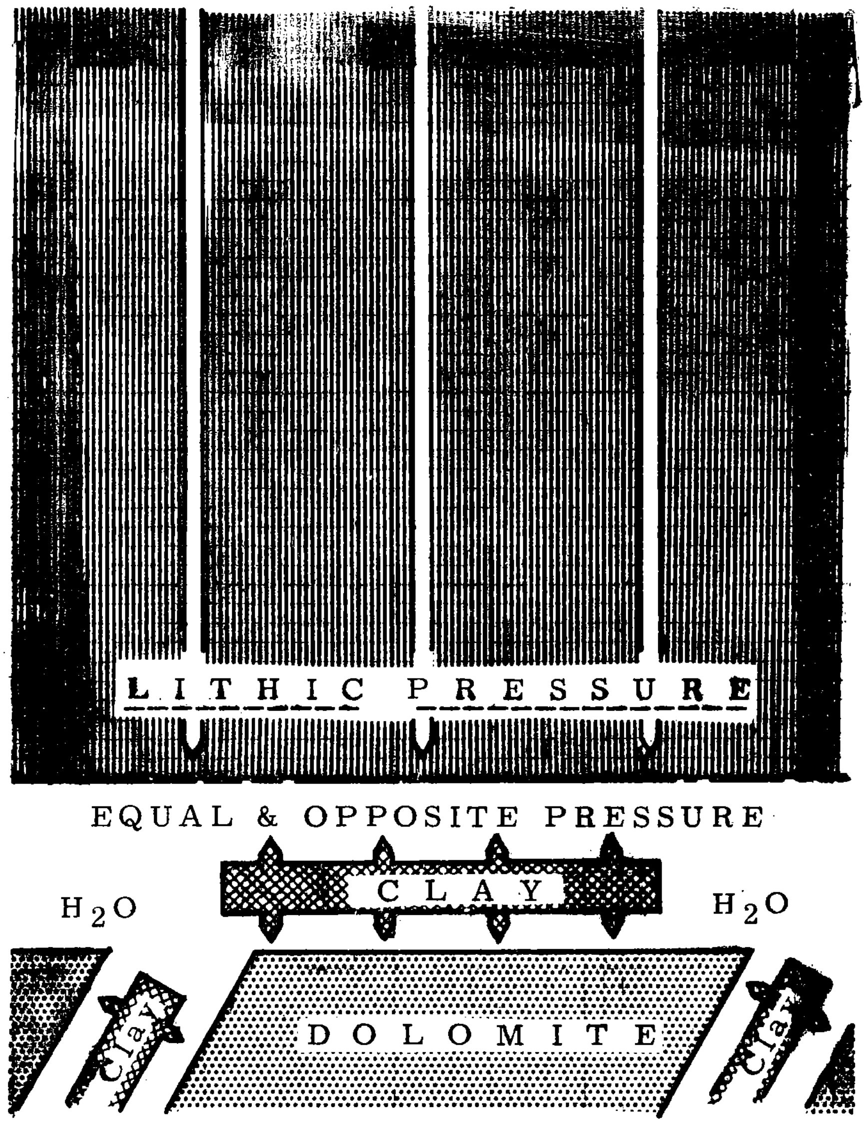

LITHIC PRESSURE
EQUAL & OPPOSITE PRESSURE
CLAY
H_2O
H_2O
Clay
DOLOMITE
Clay

Fig. 4

The average ratio of the per cent of water adsorbed at 100% R.H. at 30°C, to the per cent which froze, was 1:1 for all 30 rocks and for 8 porous porcelain cylinders, i.e. the adsorbed water did not freeze.

Sorption Versus Soundness

If frost sensitivity is often inversely related to the quantity of ice which forms, can unsoundness be directly related to sorption characteristics?

Figure 5 is a plot in which per cent void space after 24 hour absorption is plotted against per cent adsorbed after three days in 100% R.H. at 30°C. One hundred per cent saturation is defined as that quantity of water taken on after six hours of vacuum saturation in water. Note that most of the highly unsound rocks plot in a cluster about the high adsorption - high saturation part of the graph. These are rocks under sorption stress. All unsound rocks tend to fall below the dashed "critical saturation" line. Rocks at the left end of this area are truly frost-sensitive - ice sensitive. Four exceptions or near exceptions are noted. These are the four purest, low clay, calcitic limestones studied. They have vacuum absorptions of from 0.4% to 0.1% and are very densely packed, cryptocrystalline rocks. The water taken on is probably not clay-associated water but is intergranular water, largely sorbed on calcite surfaces.

This diagram is a further check of the hypothesis that "frost" sensitivity can often be related not to the ice which forms but to adsorbed water.

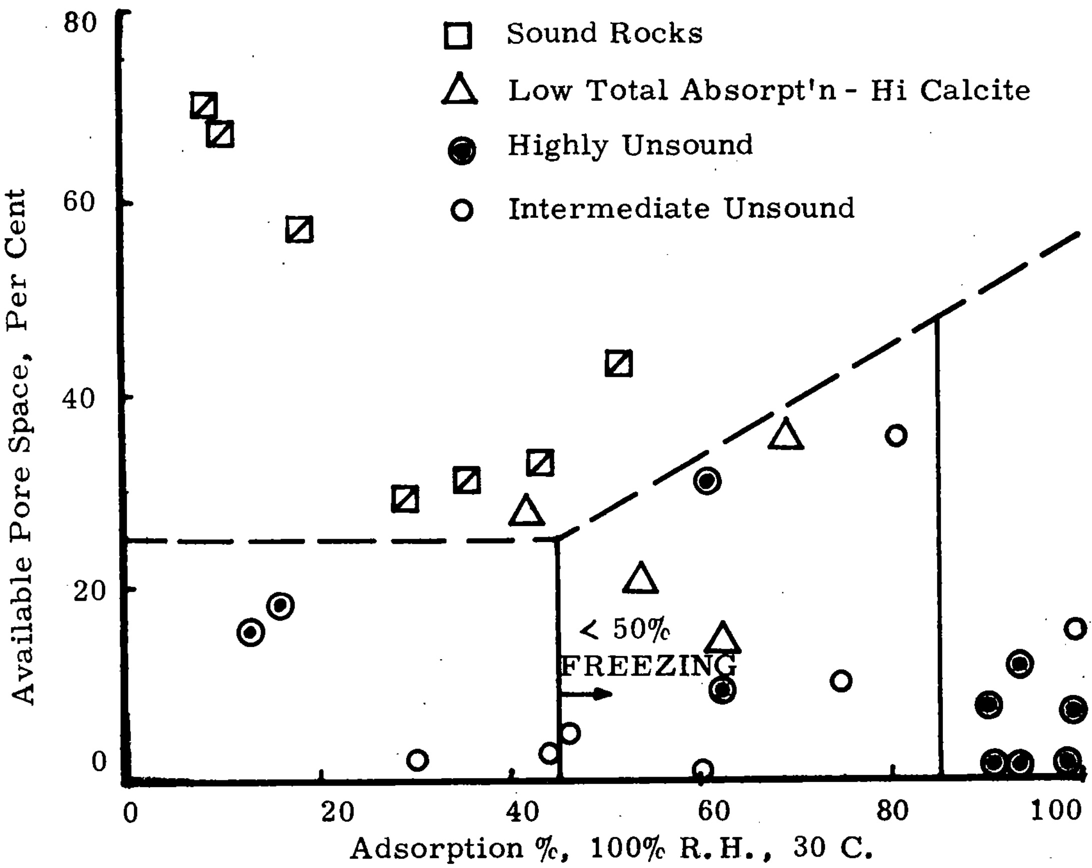
Applicability of Sorption Evaluation Procedures

A plot of the type shown in figure 5 can be used for evaluation of carbonate aggregate. If rocks with low absorptions are eliminated, the plot gives us a 100% predictability of soundness as defined by the N.Y.S.D.P.W. freeze-thaw test. Such a plot may also be worthwhile in evaluating cherts and any shaly rocks.

But is it enough merely to check a test in which back screening is used? A screen does not know the difference between a minor quantity of deleterious material, causing a piece of aggregate to split, or a major deterioration involving significant gross expansion of the whole aggregate.

We feel that in these adsorption-absorption relationships we may have a key to the measurement of bulk expansion of aggregate. The technique can be used in such a way as to completely ignore minor planes of weakness which may cause pieces to split but which would not cause significant distress in concrete.

With sorption measurements the kinds of water are sorted out; further, the quantity of each kind of water can readily be determined. We feel that it may soon be possible to match quantities of critical types of water with expansion of aggregate and ultimately correlate this with the deleterious expansion of concrete under winter conditions.



SOUNDNESS, SORPTION AND FREEZING

Fig. 5

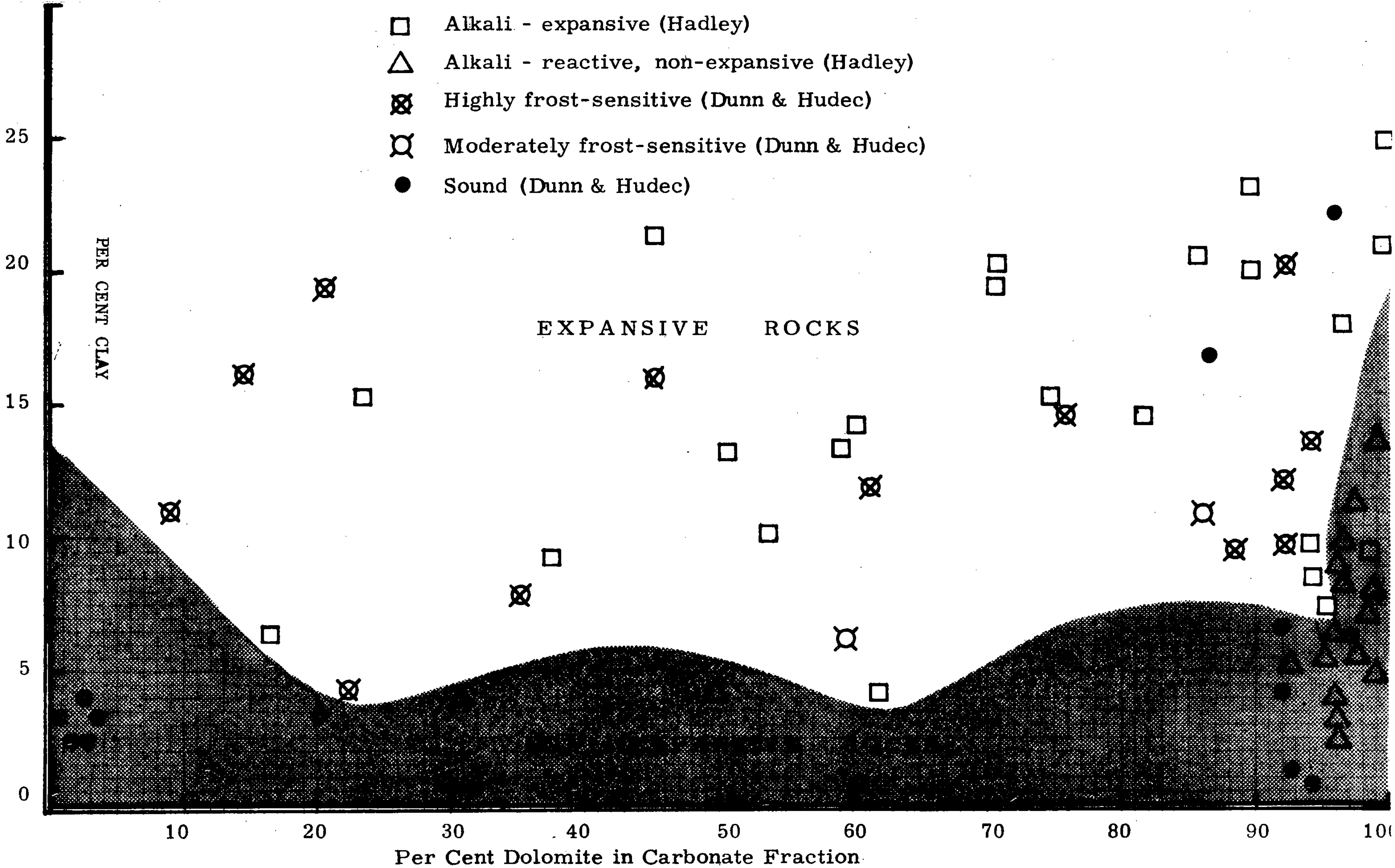


Fig. 6 ALKALI REACTIVITY & FROST SOUNDNESS RELATED TO CLAY & DOLOMITE

SORPTION, FROST SENSITIVITY AND ALKALI REACTIVITY

In working over the measurements made for the order-disorder research and those observations involving reaction rims, I have discovered a seeming paradox: "The New York State Department of Public Works freeze-thaw test may predict - at least partially - the presence of expansive chemical reactions". Surficially this is an absurdity.

Hadley (1964, figure 13) plotted per cent clay against per cent dolomite in the carbonate fraction. I have superimposed on his plot similar data for the 29 rocks studied in Dunn and Hudec (1965) for which we have chemical analyses (figure 6). I find that it is possible to draw a line, without too much fudging, which encloses all frost-sensitive rocks, as indicated by the N.Y.S.D.P.W. test, and all but one expansive, reactive rock on Hadley's diagram. (Two sound rocks fall, incorrectly, within the enclosure because they actually contain little or no clay. These rocks are sandy dolomites containing abundant feldspar, a mineral which calculates out as clay in the computer program used in the normative mineralogic determinations).

It seems likely that both frost sensitivity and alkali reactivity are functions of clay-dolomite content. It is also likely that any cause of expansion, whether it be by the formation of ice, high volume adsorbed water, or new, high volume mineral assemblages in carbonate rocks, can be related to their tendency to saturate critically and to adsorb most of their potential water in 100% R.H.. In highly adsorbent rocks the interior pore walls are probably under stress due to adsorbed water alone. For example, a sample of Black River argillaceous dolomite expanded 0.34% after 72 hours soaking in water, 0.95% after soaking in formamide. Any expansive products formed, whether chemical or physical, increase such expansive stress.

Rocks which fall outside of the expansive area are rocks which do not saturate readily, which contain must entrapped air. Expansive forces from physical or chemical reactions can thus be relieved in a manner similar to the relief of forces in air-entrained concrete.

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CONCRETE WEATHERING STUDIES

by John Lemish
Iowa State University

As my contribution to the symposium, I would like first to review the carbonate aggregate problem in Iowa. This will place what you have observed on our field trip into a better perspective and provide a better idea of the basis and approach to our research. Following this I will discuss our recent work on concrete aging studies. I would also reiterate a strong point cmphasized by previous speakers that most carbonate aggregates make good concrete highways but because poor concrete is self-evident, the relationship of concrete service records with the source of carbonate aggregates is our basis for recognition of our aggregate problem.

The history of aggregate research in Iowa centers on the key role played by the late Mr. Bert Myers who was materials engineer at the Iowa State Highway Commission for many years. It was because of his familiarity and interest in the source of carbonate aggregate as related to the condition of highway concrete that he became aware of the carbonate aggregate problem many years ago, long before any of us currently working on the problem were ever aware it existed. Mr. Myers' major problem was to get people interested enough in deleterious carbonate aggregates to do some research. In time he managed to interest Mr. Fred Dorheim, currently with the Iowa Geological Survey, to do a masters thesis problem at Iowa State University on aggregates. Dorheim's initial work in 1949 (1) led to the study of Dean Chalmer Roy and Dr. Leo A. Thomas at Iowa State University in 1951 on the problem of deleterious Mississippian aggregates from Le Grand, Iowa (2). My efforts in carbonate aggregate behavior and those of many hardworking graduate students commenced in 1955 and has continued with their help and sponsorship of the Iowa State Highway Commission to the present. As a result of Myers' early interest in aggregate research, we owe much to the Iowa State Highway Commission for their interest and support.

Our early research was of necessity aggregate oriented and concentrated on the behavior of carbonate aggregates in concrete. The initial work focused on aggregates with poor service records. Dean Roy started the petrographic study of Le Grand aggregates and I followed with a physical-chemical study of Devonian-aged aggregates obtained from the Rapid member of the Cedar Valley Formation at the Glory Quarry. We observed peripheral reaction rims on the Glory aggregate in distressed highway concrete and concentrated much of our early work in studying the nature of the reaction. We have since found out that the Glory Rock and most argillaceous dolomitic rocks of similar lithology dedolomitize, move silica about, form siliceous positive rims, and fulfill the role of a poor aggregate. The Glory Rock was the only example in Iowa of an expansive rock used in concrete. Rock with such a lithology is no longer used nor does it pass our current acceptance tests. Hence, expansive type rocks are not a problem in Iowa.

Continued investigation by Welp regarding the source of carbonate aggregate as related to service performance in highway concrete has shown that our problem aggregates in Iowa included other types of rock giving us trouble. These problem rocks have lithologies characteristic of those from the Le Grand and Otis Quarries. Such rocks are calcitic dolomites. They contrast Glory Rock in that they have very low insoluble residue contents, they do not expand, and they pass all of our acceptance tests. Because of their low silica content they do not have a silicified rim but have undergone the alkali aggregate reaction and subsequent dedolomitization forming a negative type rim. Service records indicate that we have a progressive deterioration in highway concrete made from and related to the source of such aggregates (3).

The question is, how do these non-expansive rocks cause distress in concrete? This is the crux of the problem in Iowa which is different from the expansive alkali reactive carbonate aggregates directly associated with distress in highway concretes elsewhere. I doubt our problem is unique to Iowa and in time with more study of service records and aggregate source relationship, similar associations of non-expansive aggregates to concrete deterioration will be recognized elsewhere.

Since no direct expansion behavior can be attributed as the cause of distress in concrete, our approach to the problem has been to look for other ways in which aggregates may contribute to failure of concrete. In order to do this, it became apparent we had to know how the entire system called concrete behaved. That is, how do fine aggregate - hydrated cement paste - coarse carbonate aggregates interact over a period of time. This interaction must be known before we can evaluate how coarse aggregate can contribute to distress. Research on this problem of the interaction of all parts of the concrete system will give us the type of information needed to eventually design concretes for the type and length of service we desire. This then has been our philosophy behind the concrete aging studies we have been conducting for the past five years and is the main basis of our research approach (4).

Our concrete aging study was undertaken to review in a systematic way the physical and chemical changes which occur with time in highway concretes. Concretes of different ages made from carbonate aggregates with good and poor service records have been systematically cored and studied. The concretes cored cover a time span of over forty years of service. For any concrete made with a given aggregate, an attempt was made to sample this system over as broad a time span as possible and at five year intervals. This was not always possible since concretes with a given aggregate were not placed with our study in mind. The concretes using Otis aggregate represent our best series of samples in terms of length of time span and intervals. For this reason, they have received most attention. The areas which have been investigated are the changes in physical properties such as compressive strength, chemical changes of the matrix and coarse aggregate, petrographic study of the whole system, and study of vertical gradients between the top and bottom of a concrete slab which may develop with time. It should be stated that regardless whether the concretes have a good or poor service record, all the cores studied were taken from the center of slabs where no deterioration was present (1). This was done in order to standardize our sampling procedure and provide a uniform basis of comparison. A brief summary of our observations and pertinent conclusions are now presented.

Slide 1

After the initial compressive strength data had been obtained, an initial plot of strength versus age of highway concretes made from any given aggregate are shown in slide one and it can be observed that no relationship between strength, age, and source of aggregate is evident (5). Instead a scatter showing a gradual increase in strength with time appears to be present which is typical of similar studies conducted elsewhere. This was true for concretes with good and poor service records. However, when concretes are separated according to the sources of cement used in them and plotted in the fashion shown in slide two it can be seen that there is a relationship between strength and the source of concrete. Also, by plotting the source of aggregate with time on a cement basis, we find that there is a decrease in strength of certain concretes such as those made from the Otis stone. This is, as far as we can see, the first evidence of change in strength with time which is also aggregate sensitive. In every case the decrease of strength does not appear until after ten years or more of service, suggesting that it takes time for strength dependence of aggregates to develop.

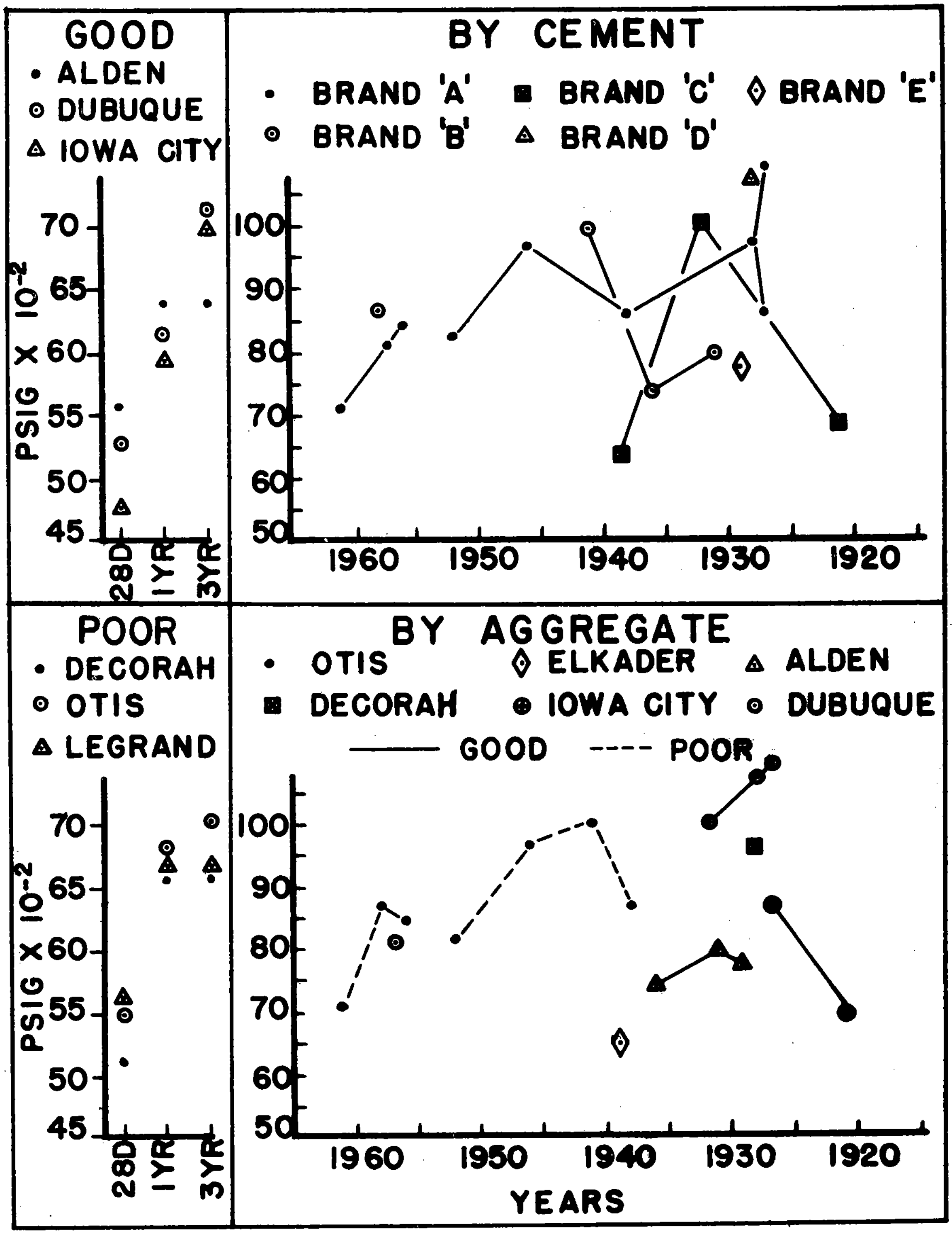
In the chemical studies of these concretes, especially on the matrix portion, time dependent changes which were found to occur are summarized in the next series of slides. A recalculation of the weight percentages in our raw analyses to a silica-free cement oxide basis was done for all of our analyses. This second slide shows what happens to the sodium content of the matrix (5). Between 1952 and 1947 there is an increase in

Slide 2

sodium content followed by a gradual decrease to an equilibrium concentration. This reversal was bothersome because it showed up in all of our plots of analyses. However, after studying the service record cards, it became evident that what we were seeing is what we call an air entrainment break. In 1952 we began using air entrainment in our Otis concretes. The span of time we are studying is represented by two different concrete systems, the air entrained since 1952 and non-air entrained concretes prior to 1952. Both are concretes then with different properties. This reversal in chemical data for the matrix of Otis concretes appears throughout our study and we have every reason to believe that it is one of the most significant observations.

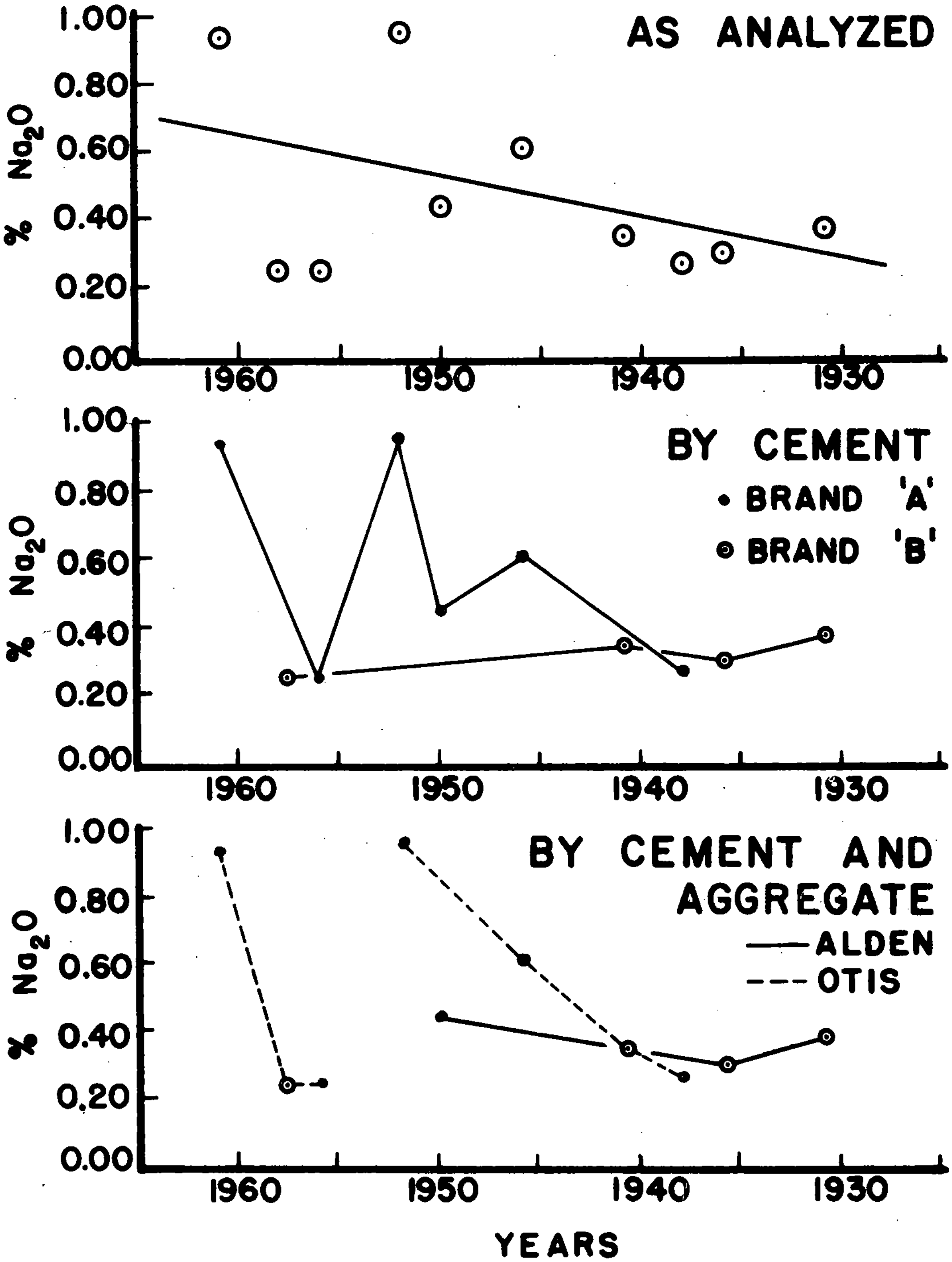
Slide 3

Slide 3 is a curve showing the changes in the water hydration and CO_2 content in Otis concrete matrix which are also quite significant. They both vary inversely with time. Water of hydration decreases as carbonation increases. This indicates that carbonation increases with time in our highway concrete and is readily verified by petrographic means.



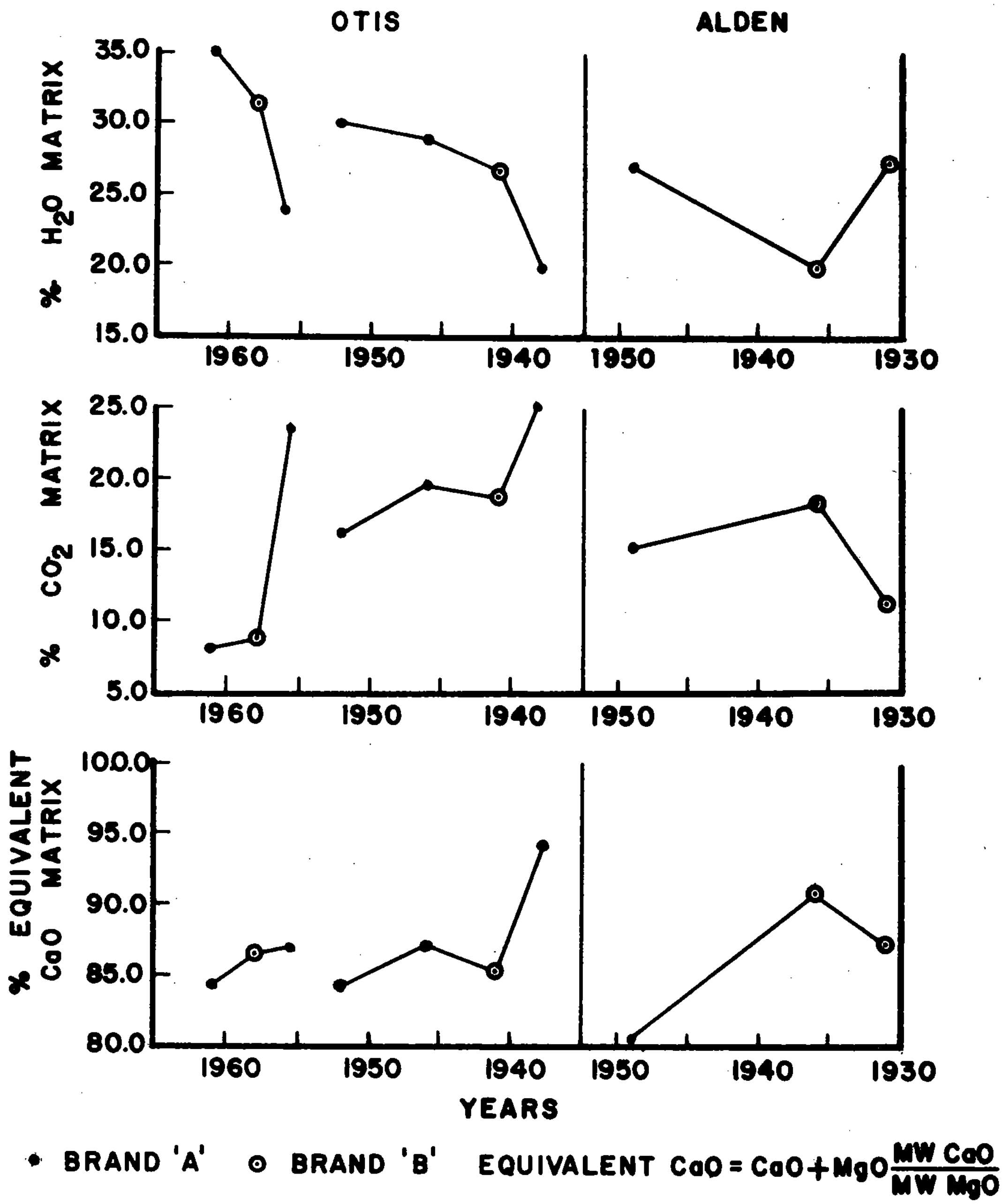
Compressive test results.

SLIDE 1



Classification of Na_2O results.

SLIDE 2



Associated matrix composition vs time curves showing H_2O, CO_2, and equivalent CaO changes.

SLIDE 3

During the petrographic phase of investigation of these Otis concretes, an attempt was made to pinpoint the behavior of various lithologies of Otis carbonate aggregates found in concrete by relating them to the actual ledges they came from in the Otis quarry (6). This was done with considerable success. Perhaps the most important observations during this study were the observations that changes in the breaking behavior of the bond zone of the coarse aggregate and matrix was dependent on time and lithology of the aggregate. It was found that the breaking characteristics of the matrix-aggregate bond area changed with time for certain lithologies (7). Young concretes break at the matrix-aggregate bond but with time we find that certain lithologies of Otis stone that have developed negative rims break within the aggregate. This phenomena occurs only after a ten to twelve year period and indicates that the negative rim zone within certain aggregates is weaker than the matrix. These data are the first direct evidence relating alkali reactivity of the aggregate with change in the strength of the bond.

At this stage of our aging studies it is apparent that all of the concrete studied to date undergoes chemical and physical change with time. The chemical changes are essentially similar for concretes of good and poor service record with exception that the rate and degree of change appears to be influenced by other factors. The study indicates that air entrained and non-air entrained concretes undergo similar chemical changes but at different rates. Matrix changes appear to occur at faster rates in air entrained concretes. Concrete strength appears to be dependent on lithologies only after enough time has elapsed to allow alkali-reaction to effect bond strength characteristics. Perhaps the most notable achievement to date is this indication that alkali reactions can in time affect the bond zone performance of aggregates. It also points out the great need for research on bond strength not only as it is related to aggregate lithology but to reaction induced changes which occur with time.

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DISCUSSION - QUESTION & ANSWER

Q: I would like to ask Mr. Axon, who inferred that he was noticing a difference in concrete service life when there was salt on the pavement. I wonder if this was more than just a casual observation or whether he had made some effort to measure this. Also he said that he thought there was a difference in the type of salt that was used. I wondered if he had investigated this to any extent.

Axon: Regarding casual observation, we have been observing our newer concrete pavements for almost 4 years. This last fall the researcher who has been doing most of this work told me he had seen this pattern cracking years ago when he was primarily studying D-cracking. But at that time this pattern cracking was very rare and didn't seem to progress. However, in some of our newer pavements we see this cracking more frequently and it seems to be progressive. As for the types of salt, I did point my finger at NaCl, and I would refer you to a statement on page 7 of HRB Special Report 31. According to this statement we would expect NaCl to react with some of the cement hydration products and provide free alkali, which would increase the alkali-carbonate or alkali-silica reaction. Consequently, the use of low-alkali cement is not necessarily a cure-all for alkali aggregate reactions, if you are going to use NaCl. This is the thing I am concerned about. We are now inspecting our newer pavements for evidence of early deterioration, and at this meeting I have been told of some early D-cracking. This D-cracking is apparently showing up in some of the pavements that contain rim developing rocks. We are not sure that rims have anything to do with the early D-cracking, but apparently the D-cracking is showing up at a much earlier age than we would normally expect. The early signs of concrete deterioration in pavements are causing us real concern.

Dunn: I would like to add another variable here. We also ran a differential thermal analysis curves on sodium chloride soaked samples. We did this because the New York State freeze-thaw test is a very severe test consisting of 25 cycles of freezing of unconfined aggregate in a sodium chloride solution. We found that in every single case the amount of ice which formed in this type of solution was far less than the amount of ice which formed by freezing in ordinary water. Though we don't have the full answer on this, we suspect that there is some sort of increase of ordering force, due to the presence of ionic sodium and ionic chlorine. This may be another factor.

Newlon: One certainly can accelerate the expansive alkali carbonate reaction either by immersing samples of concrete in a 1 to 30 sodium chloride solution as Peter Smith in Ontario has done or as we have on occasion done, by mixing sodium chloride in the concrete. In either case the expansion is substantially greater than it is in the absence of the NaCl.

Axon: Peter Smith used a saturated salt solution. We are testing companion concrete sepcimens in 4 percent solution of calcium and sodium chloride, and saturated limewater. The increased expansion we are obtaining in the 4 percent solution of sodium chloride is very similar to that obtained by Peter in the saturated solution.

Q: Does calcium chloride increase the amount of expansion?

Axon: I would say slightly greater expansion in the 4 percent solution of calcium chloride than in limewater, but not significantly higher.

Q: I just wanted to ask a question of Mr. Gillott, but he's left now, but maybe one of the other members of the panel could offer something. In his slide showing the paired relationship of increased size of the dolomite with lesser activity, would this be in his opinion, a dolomite that had uniform mosaic of dolomite euhedra or a larger amount of anhedral dirty calcite?

The size range that he talked about occurs in the range of 150 microns. In adding dolomite to these you get a more uniform texture and a relatively tight mosaic of these euhedral forms. In the finer grains you usually have more anhedral dirty calcite giving greater expansion.

Hadley: One factor that probably relates with what he said is, when you get the textures associated with dolomite rhombs of the size you mention, you are generally starting to develop a significant amount of interpenetration of interlocking dolomite rhombs. This undoubtedly would increase the strength of the rock in terms of resisting the internal force produced by whatever mechanism it is.

Mather: It would, I think, in addition, further reduce the specific surface of dolomite available for reaction. It is my recollection of the Kingston rocks that in most of those I've seen in thin section, the euhedral dolomite or groups of interpenetrating euhedral dolomite dominate over anhedral calcite. Is this agreed?

Dorheim (Iowa Geological Survey): Mrs. Mather answered the question, but I would like to make a comment. This is similar to our field trip the other day. We came home with the thought that Iowa had some very bad concrete although we knew that it has some good concrete. I think the various papers may have left the impression on those geologists who are not closely associated with this problem, that all dolomite is bad. I don't believe that the speakers mean that and I'd like to point out this fact.

Newlon: I certainly agree. We found for example, that if you establish as necessary for this expansive reaction the criteria that have been discussed which are (1) a middle amount of dolomite as opposed to calcite, (2) a relatively high insoluble residue, and (3) a rather specific kind of texture, you are beginning to impose several restrictions which multiply rather than add. You are in fact severely limiting the number of rocks that can have the necessary type of texture. We plotted on a ternary diagram the composition of the rocks using as axes (1) dolomite content (2) calcite content, and insoluble residue, we found that the rocks that lie in the central area were all reactive, and as we went progressively out of the area toward either apex they got less expansive. Likewise, if we plot the number of samples against the dolomite/calcite ratio, the distribution shows many samples at either end (either all dolomite or calcite, but few with intermediate composition). In other words there appear to be relatively few rocks that exist that have a proper texture to react. And I think that this is found in geologic literature as well as our survey of Virginia. Perhaps such systems are unstable and in nature something happens to make the rocks avoid the unstable properties.

Lemish: The environment of deposition was relatively unstable to begin with. I would like to add that we have some old concrete with an excellent service record on U. S. 20 over by Manchester. The stone is Silurian Le Claire aggregate. The concrete is in excellent condition even though the road is obsolete. It has done more than its duty and the coarse aggregate is true dolomite with a low residue content.

Q: Could I conclude from this diagram Mrs. Mather, that the calcite-dolomite ratio is certainly one of the most meaningful tests to run.

Mather: I think that it happens to be convenient for us to do the proportions of dolomite and calcite in the carbonate part of the rock first. When we get results indicating intermediate amounts of dolomite, we then see if the rock expands. A little later we run insoluble residue, using acetic acid, so that we don't tear up the clay any more than is necessary. We look at the composition of the insoluble residue by X-ray diffraction. I think that in 100% of the cases in our experience the predominant clay mineral is just what you would expect, illite. Occasionally, there are small amounts of kaolin but illite seems to be predominant. There is usually a fair amount of silt-sized quartz and sometimes other minerals. I think that you want to know the proportions of calcite to dolomite, the insoluble residue, and the texture. It is a combination of texture plus calcite-dolomite ratio that causes our laboratory to run the expansion test. Now Dave?

Hadley: There is one thing that has been brought out and that is the importance of the type of calcite. These relationships with calcite-dolomite ratio hold only with this very fine-grained calcite matrix. Sparry areas and fossil fragments do not have any real expansive effects. It is a real danger to overlook this when you run routine X-ray analyses. In one point of our research I looked at thousands of thin sections of rocks from Indiana and I found that in every case the texture was a reliable indication. The whole rock might have this critical texture as was shown in the slides of Lemish. If you get even little pockets of this texture, you're in trouble. In practical terms and especially for someone who doesn't have a diffractometer and are apprehensive, look at it through a petrographic microscope and see if you have something approaching this texture. If you are in doubt as to the nature of the carbonate, then go to the fancier techniques. Actually this rock prism expansion test is rapid and a good indicator. So that for a geologist trying to make a screening process or an exploration for bad rock, he should first look for gross texture, follow this microscopically, and then finish it off with the rock prism test.

Q: How long does this work prism test take?

Hadley: If you've got something that is really expansive it will show up in a matter of a very few weeks.

Q: Then that is still quite a while in relation to these other tests?

Newlon: Well, let me say that in our shop although we have all this equipment, it turns out to be much more convenient as a first step to take the rock and to make a couple of thin sections. It is my impression, and I would say this as an engineer rather than a geologist that with thin sections you can get a rough idea of what the relative compositions are as well as the relative size of the components and other features. With regard to the prism test, if the rock is going to expand 6½%, you can place it in an alkali solution for the rock prism test and the next day it will have expanded 1%. There are other cases where it takes a year to do this. We must suit our analysis to the situation realizing that there are some rocks that don't expand for a long time. But at this stage of the game, we look at a thin section first, then make the prism.

Annom: We have a thin section machine.

Lemish: And that's a point that I wanted to make. We don't have a thin section machine and maybe some of you don't even have your X-ray diffractometer yet. There are however, several straight forward rapid versene or EDTA analytical techniques for calcium, magnesium, aluminum and iron. You can get your silica gravimetrically from the residues. With this chemical data which is truly quantitative, one can make a quick estimate on basis of Ca/Mg ratios and insoluble residue content of potential trouble makers. Rocks with high residue which show a magnesium content of about 15% can be immediately placed in the critical category and other tests should then be tried.

Newlon: Now I can say that in Virginia, as far as expansive rocks are concerned, I have never seen one that didn't have the characteristic appearance in thin section.

Axon: We run prism tests on all stones submitted for approval for use in concrete, and we have found some to be expansive even though the acid insoluble and dolomite content is low.

Q: By low acid-insoluble, do you say less than three?

Axon: Well, less than 10. Most of the rocks we are working with are those submitted for approved use in concrete and many pass all normal acceptance tests. The rock represented by the sample on display here was rejected because it did not comply with some of these tests. But we have found some expansive rocks that did pass all of these tests.

Mather: I wanted to make a comment on the usefulness of the texture. We once spotted an expansive rock which came into the laboratory in some concrete cores from a powerhouse. The people were getting unhappy because it was hard to keep their turbines in line. We looked at the micrographs and this was a rock where there were only pockets of this critical texture. Now this is a bit of an odd-ball because it is only a very slightly expansive rock but apparently, over time, the expansion affected the alignment of the turbines. It might have had no recognizable effect in other kinds of structures.

Q: This type of texture that expands, is it more or less equigranular or otherwise?

Mather: The texture is that of little euhedral rhombs of dolomite floating in a fine-grained muddy calcite.

Lemish: If it were an igneous rock you would call it a porphyritic texture. It's a two component texture; larger rhombs of dolomite in a matrix of dirty fine grained calcite. In hand specimen the rock still retains its primary structures.

Newlon: There are obviously exceptions but we are talking here about the vast majority of cases.

Stark (Portland Cement Assn.): I just wanted to make a couple of comments on D-cracking. Dave Hadley's last slide is typical of what we studied for 5 years. That happened to be a concrete with Bethany Falls limestone. The concrete was good for about 4 years when D-cracking started to show up and in about 4 more years it progressed to that extent. That type of D-cracking is typical of what you saw on the slide. You recall there are cherts, gneisses, schists, sandstones which are also used as aggregate in concrete. We have tested small prisms of all these carbonate rocks which we use in concrete where they were associated with D-cracking, and we have no expansion or essentially no expansion. We have been working on certain concrete in which we had no expansions, so I come to the conclusion that reactivity is not necessarily the factor of D-cracking. I also mention that we had a variety of alkali content in cement that we used in these pavements. We can see no relationship there and also whether it is air entrained or not. Even with 7 or 8 percent air in the concrete the rocks would still go. As far as rims are concerned there are both positive and negative rims and one can see the same amount of rims out in the middle of the pavements as you see in the joint areas. The rim particles with either type of rim may or may not D-crack.

Q: I think Mr. Dunn talked about absorption, etc. Does this tie in in any way with this absorption percent that you pictured later?

Newlon: Also, how does your term absorption relate to the "engineering" absorption; that is the absorption of water when gravity or vacuum saturation?

Dunn: We find that type of absorption on this last diagram that I showed you. One is absorption in water for 24 hours. The second is vacuum absorption for six hours which gives us an approximation of total water which a rock can hold, and a third measurement is adsorption in 100% humidity. We find that by interrelating these - and we have a good hard predictability of field deterioration and of frost sensitivity as defined by the New York State Department of Public Works. We have not related this to concrete yet.

Q: Does this absorption give any indication of freezing at a certain level or is it something by itself?

Dunn: Remember that diagram where there were four sound rocks which plotted close to that line separating sound and unsound rocks, a couple below as I recall, and a couple above? All four of these rocks were in the same category. They were very high carbonate rocks, low clay, low dolomite. They were, in effect, nearly pure calcitic rocks, and these had high adsorption relative to the total water they could take on but they had a low total absorption. In other words the amount of water they could take on was less than about 0.3%. We would eliminate these rocks automatically as non-frost sensitive because there simply isn't enough water in there apparently to cause a deleterious expansion from absorption or freezing forces. So, if we eliminate those, that diagram that I plotted gives 100% predictability for at least these rocks which we worked with.

Q: Are you saying that this water is absorbed below the surface and not under the influence of surface forces? Does this mean one is to conclude that these rocks that form this way have smaller pores and internal surface area than the rocks that give up their water more easily?

Dunn: It depends on whether your expansion is due to absorption forces or forces. The absorption of water is usually accompanied by expansion of a rock. In rocks which are sensitive to wetting and drying, the expansion may be considerable. In rocks which are frost sensitive expansion may be much less.

Newlon: For example, a sponge will take up a lot of water, but it will give it up just as easily. There are some rocks that will take up much less total water than others but can't get rid of it as it freezes. Its pore structure is such that water can't get out.

Q: Wouldn't the combination of absorption test and freeze test related to the specific gravity delineate the property we are talking about?

Dunn: As I said, we ran three sorption measurements and by interrelating them, we can separate those rocks which are frost sensitive from those which are not. I don't see any single test or any pair of tests which will give us an answer although the tendency to approach critical saturation in 24 hours is definitely a thing which all the unsound rocks we worked with have in common.

Q: With expansion and saturation plus possibly another of these tests, I think it would point directly to the rocks which you were talking about.

Mather: I want to comment on this question. You cannot in practice pick out the sort of rocks which fail on wetting and drying, the particular kind of rock that Dr. Dunn has worked with, which are very closely related to the rocks which are alkali-carbonate reactive by the normal physical tests. You can't pick these things out on the normal physical tests of aggregates. There is a spread in specific gravity consequent upon the dilution of the clay portion of the insoluble residue with quartz, and a spread consequent upon the changing proportions of calcite and dolomite, thus a wide range of specific gravity. You have also a range of rocks which have low absorption in the standard engineering tests and those of much higher absorption. The standard engineering tests don't delineate these rocks.

Q: What about the greater saturation method?

Mather: This is not a test done on a routine basis, and I don't know a really satisfactory way to run it without getting terribly fancy.

Newlon: With great regret I must interrupt this discussion. I would say that those of us who were somewhat less than overjoyed by having a meeting scheduled for Saturday morning, can thank the forward thinking of the people who scheduled it. In view of the excellent discussion, if it were not getting on past noon on a Saturday, we probably would spend the rest of the afternoon here discussing this particular subject. We do, however, have to stop. (We are also running out of tape).

I would like to summarize by reading, if you will bear with me, one paragraph. As chairman of HRB Committee MCB2 I was asked to write a forward to a recently published HRB Record 45 (for those who are interested in getting more background, it is still perhaps the best place to go first, and it is available from the Highway Research Board). In that summary I said, "The papers in this symposium present a considerable volume of data on the chemistry of reaction, the behavior of reactive aggregates in concrete, the geographical distribution and geologic characteristics of reactive rocks. Sufficient field studies have been presented to indicate the need for some control of certain carbonate aggregates. The task of determining through systematic studies the extent and nature of such control remains. The great need at this point appears to be systematic and documented studies to correlate the various laboratory results with service records and field performance.

This statement is particularly appropriate in Iowa, where they have what I believe to be a Highway Department with perhaps the best service records with which I have come in contact. Perhaps the summary of all we've been saying is given in the title of one of the HRB Record 45 papers by Peter Smith. I will thus leave you with the title of this paper which is: "Learning to Live With a Reactive Carbonate Rock."