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Proceedings of the 18th Annual Highway Geology Symposium April 20 and 21, 1967

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Proceedings Edited by
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Purdue University • Lafayette, Indiana

Purdue University is one of 68 land-grant colleges and universities which owe their origin to the Land-Grant Act of 1862, otherwise known as the Morrill Act. Three years after its passage, the General Assembly of Indiana voted to avail itself of the provisions of this act and began preliminary plans for a school devoted primarily to the agricultural and mechanical arts.

In 1869 the Assembly voted to accept a gift of land and money from John Purdue and other generous Lafayette citizens and, in appreciation, declared the name of the new institution to be Purdue University. The University is supported mainly by state appropriations, supplemented by federal grants.

Actual instruction began in 1874 with 39 students and a faculty of six. Today Purdue conducts classes and research in more than 70 principal buildings and controls over 7, 000 acres of land.

Undergraduate and graduate instruction is offered in agriculture (including agricultural engineering); aeronautical and astronautical engineering, chemical engineering, civil engineering, electrical engineering, engineering sciences, industrial engineering, mechanical engineering, materials science and metallurgical engineering, nuclear engineering; industrial management; home economics; pharmacy and pharmacal sciences; science; humanities, social science, and education; technology; and veterinary science and medicine.

Extensive experiment stations in both engineering and agriculture are maintained by the University. The Cooperative Extension Service provides a pipeline to the people of the latest scientific and technical information. Courses with credit toward a college degree and a two-year Diploma in Applied Technology are offered at regional campuses. The principal ones are in Fort Wayne, Indianapolis, Hammond, and Westville. A variety of courses is also offered through the adult education programs sponsored by the University.

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Preface

The 18th Annual Highway Geology Symposium was co-sponsored by Purdue University and the Indiana Highway Commission. The Symposium included a one-day field trip in northern Indiana followed by technical sessions the following day. During the field trip, pavement performance and soil conditions were observed; the limestone quarry at Kentland, Indiana, was also visited.

Ten papers were presented during the technical sessions of the Symposium. This volume contains the papers in the order in which they were presented at the meeting. The Co-chairmen are indebted to the authors for their contributions to the program. Special thanks are also extended to Mr. A. C. Dodson, Chairman of the National Steering Committee of the Highway Geology Symposium, for his assistance as Chairman of one of the technical sessions.

A panel discussion on subsurface investigations in the states of Indiana, Ohio, and Illinois was presented during the afternoon technical session. Dr. C. W. Lovell, Jr. of the School of Civil Engineering at Purdue University, served as panel moderator. His efforts in the organization and direction of the panel discussion are acknowledged. The panel discussion was recorded on tape, subsequently transcribed, and appears in these Symposium notes following the formal papers.

The two-day meeting was concluded with a banquet. Dr. G. A. Leonards, Head of the School of Civil Engineering, presented a talk at the banquet on the Aswan High Dam.

The co-chairmen wish to express their gratitude to Dean M. B. Scott for his welcoming remarks and services as banquet Master of Ceremonies, and to Professor R. D. Miles and Mr. C. F. Hotler for their aid and guidance on the field trip.

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Some Highway Problems of the United States Correlated with Physiographic Provinces

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ABSTRACT

This paper presents a summary of some of the work which has been underway at Purdue University in the School of Civil Engineering for more than 25 years. Emphasis is placed on Physiographic regions of the United States, which in some instances have been modified for engineering. A major attempt is made to show the correlation between such regions and all kinds of highway problems which in the broad sense are really Civil Engineering problems.

INTRODUCTION

It is the purpose of this paper to describe in some detail a physiographic region -- in this instance, Interior Low Plateau Province -- with considerable emphasis on highway engineering problems and their solutions. These problems will be discussed in the light of contrasting problems of adjacent physiographic regions such as the Central Till Plains Section to the north, the East Gulf Coastal Plain Section to the southwest, the Ozark Mountain Province to the west, and the Appalachian Plateau Province to the east and south.

Only one physiographic province is covered in detail with the hope that comparisons of regional problems will be of sufficient contrast to indicate the need of the regional concept with respect to highway location, design, construction, and operations.

BACKGROUND

In the mid 30's, as Soils Engineer for the Ohio Department of Highways, the author read an article by H. F. Janda (1) in which he related concrete pavement performance to a particular soil area in Wisconsin which had been mapped for agricultural purposes. Eno in Ohio, had been experimenting with the same idea (2) and in addition, he had performed early field experiments in the uses of base courses in certain soil areas of Ohio. In 1943, Belcher, Gregg, and Woods (3) reported on the regional concept of the soils and outcrops of Indiana. In 1945, Jenkins, Belcher, Gregg, and Woods (4) prepared a soils map and reported the origin of soils in the 48 states (See Figure 1). In 1960, Woods and Lovell (5) published material on the Distribution of Soils in North America. Currently the school of Civil Engineering at Purdue has a contract with the National Co-Operative Highway Research Program Project entitled, "Regional Factors Influencing Highway Design and Performance."

During this more than 30-year period, many engineers, geologists, and agronomists made similar contributions, to this concept, too numerous to mention here.

INTERIOR LOW PLATEAU

The Interior Low Plateau Province is located largely in western and central Kentucky and central Tennessee, but segments extend into southern Ohio, southern Indiana and southern Illinois, with a little portion in northern Alabama. (See Figures 2 and 3). In general, the boundaries between this province and adjacent ones are sharp and certainly the rock and soil conditions of the adjacent provinces are mostly in sharp contrast with those of the Interior Low Plateau.

Rainfall is about 40 inches annually and frost action is not a serious problem.

The rocks range in age from Pennsylvanian to Ordovician. The Pennsylvanian rocks are sandstones, shales, and some coal with the Mansfield sandstone standing out as a strong escarpment. The Mississippian rocks are composed of sandstones, shales and limestones with some of the limestone developing into strong solution topography including caves, sinkholes, and underground streams. The Devonian rocks are mostly shales, while the small outcrops of Silurian rocks are dolomitic limestones. The Ordovician materials range from massive limestones to massive deposits of shale interbedded with thin slabs of limestone.

The province is subdivided into the Kentucky Bluegrass, the Nashville Basin, the Highland Rim, and Shawnee Section. Much of the province consists of eroded remnants of the Cincinnati Nashville Basin. The Blue Grass area is composed mostly of Ordovician limestones. The Nashville Basin is made up of Ordovician rocks. The Highland Rim consists mostly of Mississippian and Pennsylvanian rocks of great thickness.

Blue Grass Section

The Blue Grass Section has an Inner and an Outer belt. The Inner Blue Grass is about 2500 square miles in area and has a rich phosphate soil developed from Ordovician limestones and there is some minor solution topography. In contrast, the Eden Shale Belt is of similar size, but the area is severely dissected. The Outer Blue

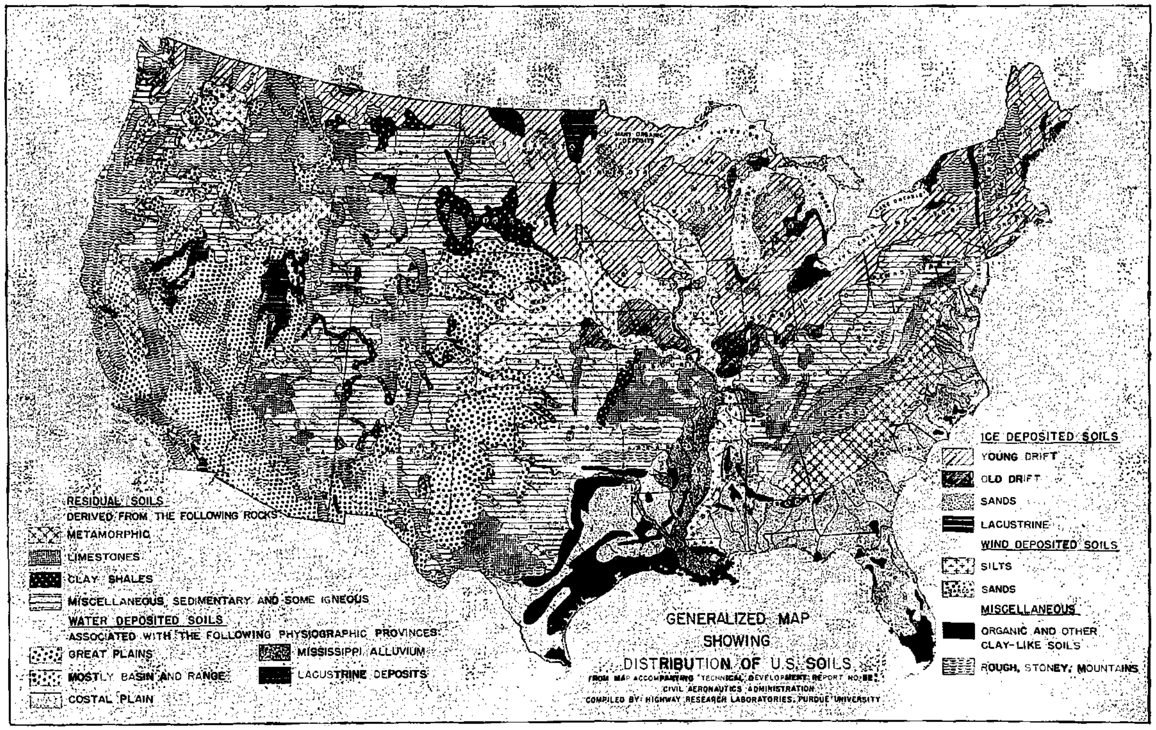

Figure 1. Distribution of Soils in the United States.

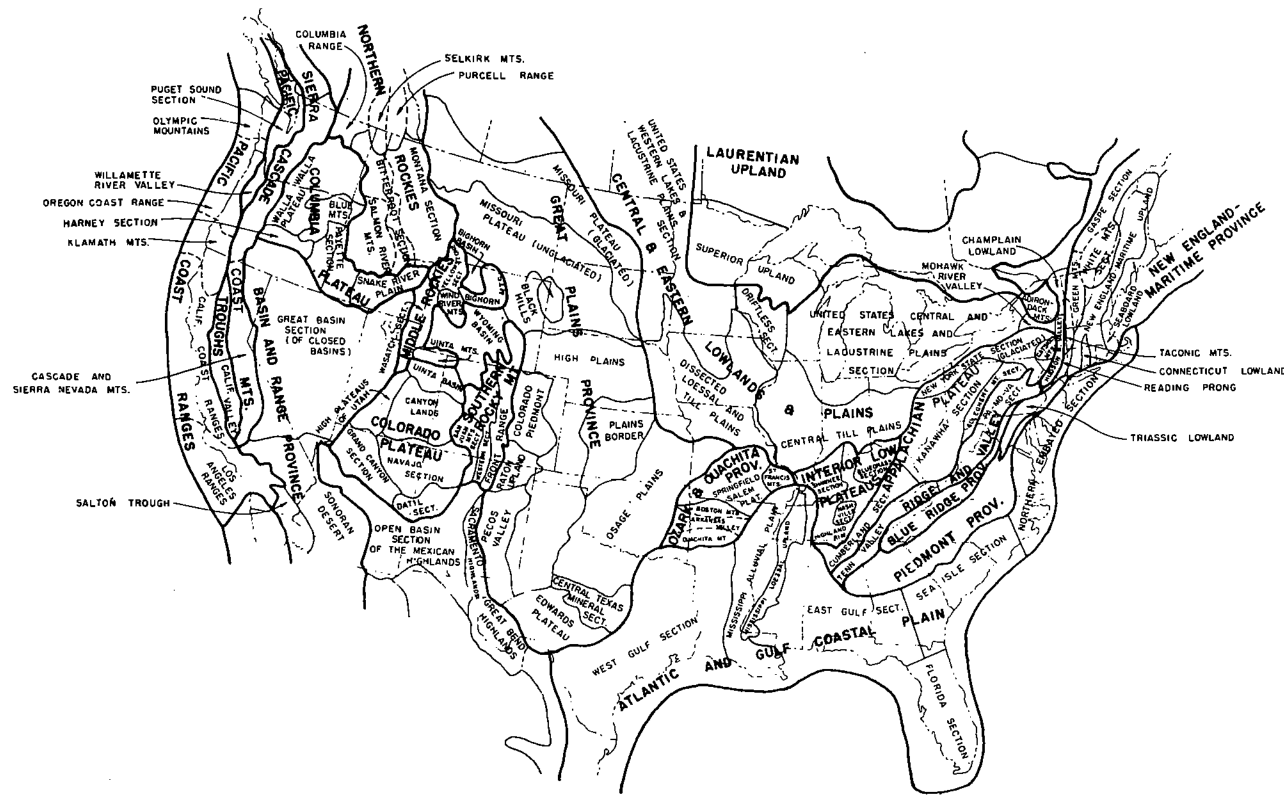

Figure 2. Physiographic Provinces of the United States.

INTERIOR LOW PLATEAUS

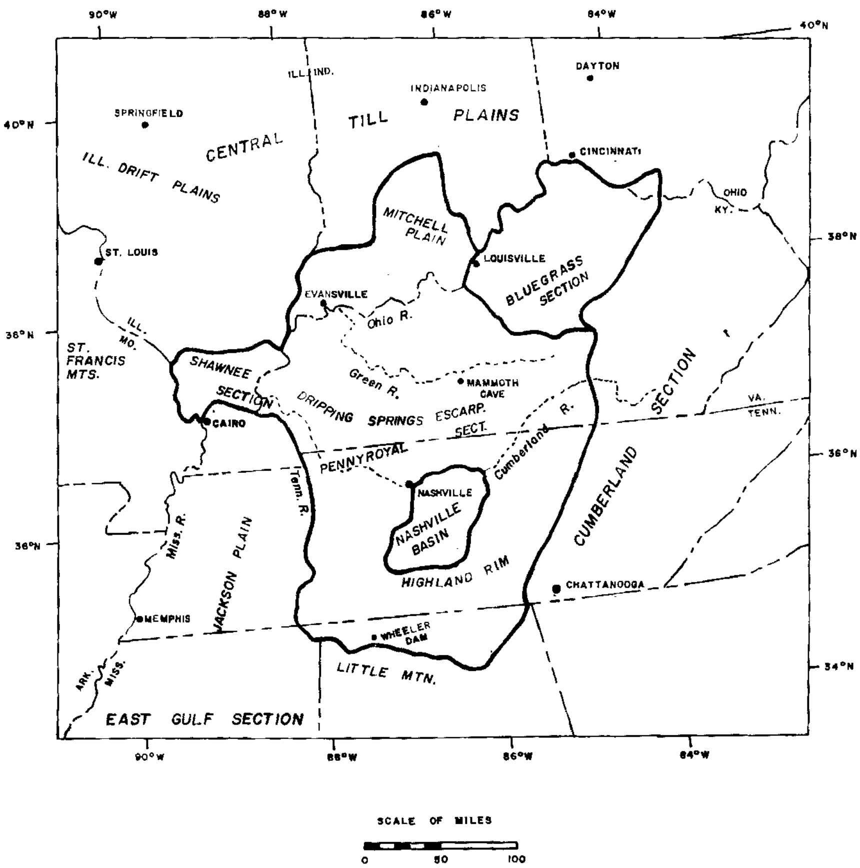

Figure 3. Physiographic Divisions of the Interior Low Plateaus.

Grass is about 3,000 square miles in area and is mostly Ordovician and Silurian limestones and shales with varying topography.

Nashville Basin

The Basin is about 7,000 square miles in area with the Cumberland River flowing through the northern portion. The northern and eastern walls of the river valley itself form the boundary between the Basin and the Highland Rim. Like the Blue Grass Region there are many areas of thin fertile soil -- but in contrast there are places of bare rock known as "the glades." Chert is a common material in the glades. River terraces are common and they generally contain an abundance of cherty gravels.

Shawnee Section

The Shawnee Section is bounded on the west and south by the Coastal Plain and a very small portion of the Ozark-Ouachita Provinces, on the east and south by the Highland Rim while a portion of the Central Till Plains are on the north. On the eastern and southern edges of the Section, bedrocks are limestones with massive sandstones, shales and coal and underclays in the remaining area. There is a thin cover of windblown silt on the west. Many of the valleys of the area on both sides of the Ohio River were blocked during glacial periods, thus resulting in flat, deeply-filled valleys of silt soil. The Shawnee Hills does produce lead and zinc as a result of the faulting caused by the St. Francis Mt. uplift to the west.

Highland Rim (Including the Pennyroyal District)

In addition to surrounding the Nashville Basin, this section partially encloses the Blue Grass, and Shawnee Sections. It also extends into south-central Indiana as far as Bloomington. Devonian and Mississippian rocks are the dominant materials. The topography varies and includes the "knobs" just west and south of the Blue Grass. Solution topography is quite common and is very extensive in southern Indiana. Mammouth Cave and others are developed in the limestone on the boundary between the Highland Rim and the Shawnee Section. In southwestern Kentucky, a large limestone plain is known as the Pennyroyal. The Tennessee Valley Authority and other federal agencies have located important projects in the Rim (6) including the Muscle Shoals Development (7). Also in the Rim and Shawnee areas some of the Mississippian sandstones have been impregnated with bitumen known as Kentucky rock asphalt (8, 8a) a product of widespread use in the 1930's and 40's in Kentucky and surrounding states. Rock asphalt has been used throughout the midwest for skid-proofing (9) of slippery highways.

Highway Performance

In the early 1940's and later, most heavily traveled roads developed severe pumping (10, 11) especially in the Nashville Basin and Highland Rim Sections. Modern design requires the use of well-graded granular base courses to prevent pumping.

Perhaps one of the most severe problems in certain portions of this section is that of aggregates, especially in the Cumberland, Tennessee, and Ohio River Valleys (12). The gravels are generally of the low specific gravity chert which has produced poor-performing concrete pavements (13,14). However, these materials are quite acceptable when "sweetened" with good-quality limestone aggregate. Also these cherty gravels can be used satisfactorily in mass concrete and as base courses for both rigid and flexible pavements. There are many limestone deposits throughout this region which are quarried successfully to produce good-quality limestones.

Highway location becomes a problem of considerable magnitude in areas of extensive solution topography. The Eden Shale is one of the problem materials both from the standpoint of landslides in cut-sections as well as in adjacent embankments because of the rock-soil mixtures which are very difficult to compact. The slabs of limestone must either be broken into small pieces or wasted to the side of the embankment if good compaction is to be achieved. Floods are expecially hazardous in that portion of the Ohio River which crosses the Interior Low Plateau (15,16).

In many areas of this Province especially where limestone predominates, water supply is difficult to locate because of the shortage of water in the bedrock.

PAVEMENT PERFORMANCE AND PROBLEMS IN SURROUNDING PROVINCES

Ozark and Ouachita Provinces

The Shawnee Ridge in southwestern Indiana and southeastern Illinois is a small segment of the Highland Rim which is adjacent to the Ozarks (Salem Plateau and St. Francis Mt. Uplift). The Salem Plateau consists of Cambrian-Ordovician Age. These materials are mostly cherty limestones with some interbedded sandstones. The local relief is sometimes great because of the differential resistances to weathering between the sandstone and limestone. The soil cover is a deep plastic red clay containing 30-35% chert pebbles. The soil is porous and well drained, despite its high plasticity, resulting in solution topography. There is practically no surface run-off. The Salem Plateau and the Springfield Plateau to the west are famous vacation areas. Some of the rivers are used for both Recreation areas as well as power (17).

The highway problems are of considerable magnitude; especially highway location in some heavy rock excavation. The deep soil mantle provides poor subgrade support in both cut and fill sections for rigid and flexible pavements. Rigid pavement pumping (18) has been a serious problem on heavily traveled roads. This is caused, in part, by the difficulty in obtaining compaction of the soil-chert mixtures. There are good-quality limestones in the Salem Plateau but the river gravels have great quantities of chert and have been a subject of laboratory and field investigations since 1924 (19,20,21).

Appalachian Plateaus

The eastern portion of the Highland Rim lies adjacent to the Cumberland Plateau while most of the eastern protion of the Kentucky Bluegrass is adjacent to the Kanawha Section of the Appalachian Plateau. The rock which makes up both these units consists primarily of sandstones, shales and underclays with sandstone being the prevailing material in the Cumberland Plateau. Softer sediments predominate in the Kanawha Section. The topography is strong as a result of the differential resistance of the various rocks to erosion. The Cumberland Plateau is even rugged in some areas.

The Appalachian Plateaus were a topographic barrier to the migrations from the east in the late 1700's and in the early 1800's (22, 23). In recent years, thruways and turnpikes (24, 25) have tunneled through some of the major physical obstacles but highways, airports, and railroad (26, 27) location continue as a major problem in crossing the Plateau. Aggregates (28) are non-existent excepting for river gravels. However, blast furnace slag is available in the Pittsburgh and Ohio River areas. In the Kanawha Section, landslides are common in both cut and fill sections (29,30). Snow removal (31) is difficult and costly with snow and ice appearing early in the fall in the higher reaches. Severe floods occur periodically in this entire province but the considerable flood control works of the Ohio and its tributaries have lessened its danger (32). Coal mining continues in the southern portion of the plateaus, but old mines constitute a hazard in highway location as well as in stream pollution (33). Earthwork is serious because of the mixture of shales, sandstones, and other materials and the difficulty in compacting these materials properly in embankments. Pavement pumping is common on the heavily traveled roads, and base courses are needed for both rigid and flexible pavements. Water supply and sanitation are difficult problems thus requiring the location of roadside parks near streams.

The Central Lowland

North of the Interior Low Plateaus is the Central Lowland with the entire boundary of the two provinces being the Till Plains. The Lowland Province extends far beyond these boundaries to the east, west, and north. The Till Plains are essentially various ages of rock strata covered with glacial drift, and the latter material is divided into "Old Drift" and "Young Drift" for engineering purposes. The Old Drift has a deeply weathered profile with about two feet of "A" horizon (silts) and 10-12 ft. of "B" horizon (plastic clay-like soil). The Young Drift has a weathered profile of only 3 or 4 ft. of silt on moderately plastic "B" horizon with much of the area being monotonously flat as is true with the Old Drift with the exception of dissected areas and in thin glacial cover where the topography is governed by underlying rock. The Young Drift is characterized by some deposits of kames and eskers and outwash plains while the Old Drift, in contrast, is completely lacking in these landforms. Both areas are traversed by some rivers like the Wabash, Miami, and Sciota. Both are devoid of lakes. To the north in Ohio, Indiana and Illinois, across Michigan and eastern Wisconsin are the materials of the Eastern Lake Section. This region is characterized by moraines, lakes, lowlands, and the lacustrine plains. The climate of both sections becomes increasingly colder toward the north with frost penetration increasing also toward the north. Heavy snows and blizzards are common. For early classical work on glaciation of this midwest region the reader is referred to the work of Leverett (34,35).

In the Old Drift area, aggregates are frequently non-existent with the exception of the river gravels of the Ohio River and adjacent tributaries. The problems of chert are similar to those of the Interior Low Plateaus. Good quality limestones are available in adjacent areas. Pavement pumping prevails on heavily traveled roads in cut sections of the Old Drift (36) and granular bases are an absolute necessity. In the Young Drift, important test roads have been used to develop pavement designs including Bates, (37) the AASHO, (38) and many additional state tests in Ohio, Indiana, and Illinois. Good quality aggregates are generally available throughout the central section of all three states, but there are areas where aggregate sources are completely lacking. In some areas, serious problems are encountered in the use of laminated limestones which have been related to blowups (39) of portland cement concrete pavements. Frost action and spring break-up are frequently severe throughout this entire area (40,41). Immediately following World War II, pumping of rigid pavements was probably more widespread (42, 43) than in any other region of similar size on the continent. Floods have been common, hazardous, and economically destructive in this fertile midwest region (44, 45) in most of the tributaries flowing into the Ohio River from the north. Interestingly enough, this region was crossed from south to north by the Michigan Road (46).

Among the more serious highway problems in the Eastern Lake Section include icing of pavements when the temperature is at or just below freezing and blizzards with resulting snow drifts (47,48). Peat bogs are quite common throughout the eastern lakes area (49, 50). In the lacustrine areas, i.e., Cleveland, Toledo, Detroit, and Chicago, deep deposits of clays occur some of which are varved. (51,52). Although not a highway problem, it is nevertheless a problem of magnitude in connection with the cyclic high and low elevations of the Great Lakes (53). Also water pollution of the Great Lakes, especially Lake Erie (54), has become a matter of great concern to both Canada and the United States.

Coastal Plain

Forming the western boundary with the Interior Low Plateaus is the East Gulf Coastal Plain. The contact is parallel and close to the Tennessee River (55). The contact is limited to that area from Cairo, Illinois, and Paducah, Kentucky to a small portion of Alabama, southwest of Florence near Muscle Shoals. This portion of the Coastal Plain is sometimes called the Belted Coastal Plain with wider and thicker belts than occur in the Sea Island Section of the Coastal Plain to the east. Cretaceous sands and gravels constitute the immediate boundary; with the Tennessee River having entrenched itself in the Highland Rim in Paleozoic rocks to depths of 150-300 ft. The topography in this portion of the Coastal Plain is fairly well dominated by the underlying rocks.

The highway problems are relatively minor. Good quality limestones are available in the adjacent Highland Rim and some of the cretaceous gravels are of good quality. The cherts of the Tennessee River are of inferior quality. Compaction of the sands for subgrade is a problem throughout the Coastal Plain. Clay soils to the south and west are derived from Selma chalk and give rise to problems of "swelling soils". There is also a complete lack of good quality aggregate in the so-called "black belt". Sand-clay materials are used extensively in the southeast areas of the Coastal Plain (56,57,58).

SUMMARY

1. It is the author's conclusion, both from years of study and from travel as well as from the technical literature, that the regional concept of physiography and the engineering applications is essential to the efficient location, design and performance of highways.

2. Borrowing from Jenny (E) the most important factors influencing the location, design, and performance of highways are:

   a. Topography
   b. Texture of rocks and of soils
   c. Geologic structure
   d. Climate
   e. Man made features, i.e. abandoned mines, waterways, irrigation works, and other land use.
   f. Economics
   g. Sociological factors

3. There is need for continued research to more adequately evaluate the factors listed above.

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Anyone Can Interpret Soil Groups from Aerial Photographs

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ABSTRACT

A presentation of basic concepts of airphoto interpretation of engineering soil groups for engineering site selection studies. Fundamental principles are discussed as are the concepts of airphoto patterns. Airphoto patterns are subdivided on the basis of elements of form and elements of tone. Applications of color photography and infrared imagery are briefly discussed.

INTRODUCTION

Eldon Yoder, when he asked me to discuss airphoto interpretation, restricted me to a title of 10 words or less. A more complete title would be "Anyone Can Interpret Soil Groups From Aerial Photographs IF he were properly trained and is familiar with and accepts certain precepts in *Geology, Pedology* and *Soils Engineering.*"

Airphoto interpretation is a technique of correlation. It is defined as the art of analysing elements recorded on aerial photographs and by inference and logical deduction determining the soil group pattern and its significance on a particular problem such as engineering site selection.

Concepts for correlation and analogy are developed by the geologists, pedologists and soils engineers. The most significant concepts are contained in the classical texts on geomorphology, soil science, and soil engineering. The professionals in these fields obtain the basic control used by the civil engineer engaged in photo intepretation studies. Analogous to a photogrammetric mapping project, an airphoto interpretation study is only as good as the field control. The aerial photograph is the record used to extend control, deduce changes or anomolies and recorrelate these changed conditions or patterns.

PATTERN CONCEPT

The entire philosophy is based upon the concept of classification of patterns or their anomolies. The basic principles are: (1) the aerial photograph records the results of natural and man-made processes by reflecting surface features called elements, (2) the elements can be grouped together to form a characteristic pattern, and (3) patterns are repetitive and correlative.

The resolution of photography is such that it is impossible to record the particle size and shape of earth materials except on very large scale photography. Similarly, as you drive along roads in certain parts of southern Indiana you see red soil change rapidly to whitish rock with uniquely rounded exposures and rolling topography with distinct basins. You cannot see the silt and clay size particles that make up the soil or the minerals that constitute the rock. You do see elements that collectively form a visual image of "terra rosa" soil and limestone rock. Your mind examines the information and compares the visual images and deduces the soil and rock type from this indirect evidence. Airphoto interpretation is analogous to identification of rocks and soils by examination of field exposures and/or hand samples except that elements of identification consist only of *elements of form* and *elements of photographic tone*. These elements form patterns.

LAND FORMS AND PARENT MATERIALS

The discrete patterns that are repetitive in nature are the land form type and the parent material type. The engineering soil group is derived by inference by relating factors of soil formation to photo pattern elements of land form type and parent material type.

The concepts and terminology of land forms are derived from geologic and geomorphic literature. A partial listing of important land form types associated with the consolidated rocks is shown in Table 1. They are arranged according to origin as Plutonic, Volcanic, Tectonic, or Remnantal (erosional form). They are grouped according to first order topographic forms that are readily identified on aerial photographs. These forms have the following limiting conditions:

a. mountain - single summit, linear summit or multiple summit or multiple summits with local relief greater than 1000 feet.

b. hill - single summit with less than 1000 feet of local relief.

c. ridges - linear summit with less than 1000 feet of local relief.

d. plateau - constant summit levels with local relief greater

TABLE 1

ROCK LAND FORM TYPES

	Plutonic	Volcanic	Tectonic	Remnantal
Mountain	Batholith Dome Laccolith Stock Massif	Cone Dome	Anticlinal Mt. Horst Synclinal Mt.	Bornhardt Cirque Exfoliation Dome Horn Mesa Monadnock
Hill	Batholith Boss Cupola Dome Island Laccolith Pluton Steptoe Stock	Ash Cone Ash Flow Cone Dome Island Plug Lava Shield Tuff Cone	Dome Horst Island Salt Dome	Bornhardt Butte Exfoliation Dome Flatiron Hums (Pepino) Inlier Mesa Monadnock Neck Outlier Pinnacles
Ridge	Dike Pluton Massif	Coulee Dike Flow Tongue	Anticline Dip Slope Fault Scarp Homocline Horst Syncline	Atoll Reef Anticlinal Ridge Clastic Dike Cret Cuesta Scarp Dip Slope Fault-Line Scarp Hogback Homoclinal Ridge Rock Drumlin Sill Scarp Synclinal Ridge
Plateau	Shield Pluton	Coulee Dome Plateau Lava Shield	Bench Karst Plateau Plateau	Bench Karst Plateau Plateau
Plain	Island Shield	Flow Island Plain	Bench Island Karst Plain Plain	Bench Karst Plain Plain Pediment
Valley	Dike Valley Joint-Line	Joint-Line	Fault-Line Joint-Line Rift Valley	Anticlinal Valley Canyon Glacial Trough Gully Homoclinal Valley Synclinal Valley Water Gap
Basin		Caldera Center Lava Sink	Atoll Graben	Atoll Sinkhole Cirque Tarn

than 500 feet, bordered by an escarpment on at least one side and with bedding planes in essentially a horizontal attitude.

e. plain - constant summit levels with local relief less than 500 feet with bedding planes in essentially a horizontal attitude.

f. valley - linear depressed feature with a surface drainage outlet.

g. basin - curvilinear depressed feature without a surface drainage outlet.

The land forms associated with unconsolidated or drift materials (i. e., the regolith, mantle rock, deposits, or soil in the engineering sense) are tabulated in Table 2. The land forms are arranged according to environment of origin and first order topographic forms.

The airphoto pattern of consolidated and unconsolidated land form types can be described for various climatic environments. The primary airphoto interpretation feature for detailed development is the element of form to include all the elements of topographic form, drainage form and erosional form. Elements of photographic tone play only a secondary role usually to assist in evaluating the climatic environment, the parent material, and general soil conditions.

Parent material refers to the core material within the land form. There are two groups of parent materials - the consolidated materials or rock and the unconsolidated materials or drift. These are subdivided according to environment of origin. The classification of parent materials for purposes of airphoto interpretation in engineering studies is very important.

The consolidated parent material type can be related to the core material of the rock land form type by airphoto interpretation. Once identified, the bedrock type is used to modify the land form type to differentiate the land form-parent material type into discrete units. For example, a limestone plain, a basalt shield, a rhyolite tongue, a granite shield, etc., are all discrete land form-parent material types. Basic and supplemental ground control from sample areas is important in this interpretive process.

The unconsolidated parent material classification pertains to the drift types. The engineer is interested in this earthen material before diagenesis.

The drift parent material types are invariably combinations of individual particles - gravel, sand, silt, and clay. In the author's experience, a silty eolian drift, or a silty glacial drift may perform differently and require different engineering designs. A

DRIFT LAND FORM TYPES

	Eolian	Glacial	Fluvial	Lacustrine	Marine	Littoral	Gravitational
Hill	Barchan Dune Clay Dune Complex Dune Dune Loess Hill	Drumlin Kame Ablation Cone	Fan Cone Pingo				Talus Cone Talus Slope
Ridge	Complex Dune Loess Ridge Longitudinal Dune Transverse Dune Whale Back	End Moraine Esker Gacier Kettle-Kame Interlobate Moraine Lateral Moraine Medial Moraine Ridge Moraine Terminal Moraine	Antidune Dam Escarpment Levee Island River Bar Point Bar	Beach Ridge Strand	Antidune Atoll? Barrier Reef? Beach Ridge Coral Reef? Fringing Reef?	Bar Baymouth Bar Beach Ridge Berm Offshore Bar Spit Submarine Bar Tombolo	Debris Slide Rock Slide Rock Glacier
Plain	Desert Pavement Erg Gassi Plain Hammada Loess Plain	Esker Delta Ground Moraine Outwash Plain Kame Terrace Ablation Moraine	Alluvial Plain Alluvial Slope Apron Arcuate Delta Digital Delta Estuarine Delta Floodplain Outwash Plain Fluvial Plain Terrace	Estuarine Plain Lacustrine Terrace Lacustrine Plain Lagoonal Plain	Coastal Bench Coastal Plain Coastal Terrace Lagoonal Plain Marine Bench Marine Terrace Tidal Delta	Wave-Built Delta Strand Terrace	Debris Flow Earth Flow Mud Flow Slump Terrace Solifluction Apron Terracets
Valley		Esker Trough Kettle Chain	Arroyo Canyon Gully Rill Valley	Inlet	Inlet	Inlet	Avalanche
Basin	Blowout Deflation Basin Gash Sink	Bog Kettle Pit Infiltration Basin	Oxbow Meander Scroll Slough	Bog Kettle Swale Swamp	Bog Marsh Lagoon Elliptical	Swale Swamp	Avalanche

classification of this type is a warning to engineers that special explorations and studies are required before designs are developed.

The airphoto patterns of consolidated and unconsolidated parent materials involve the entire subject of pattern elements. All variations in elements of form (topographic form, drainage form, erosional form) as well as elements of tones such as tone and texture related to land use, tone related to vegetation and tone related to soil conditions and soil materials are important both quantitatively and qualitatively.

ENGINEERING SOIL GROUPS

The land form type and parent material type do not completely differentiate an area into discrete engineering soil groups. Within any land form-parent material unit there may be distinct variations in texture of material produced both by origin and by weathering processes. These variations are soil differences and can be mapped as unique areas of unique soil groups.

The primary variables in mapping soil groups, after the parent material type has been classified, are topography and horizon. At this point in mapping of discrete units, time, parent material, climate, and organisms are essentially constant. Topography and its influence on soil texture and soil horizon development is a primary element.

There are two approaches to the delineation of soil groups by the technique of airphoto interpretation. One approach is to obtain descriptions of agricultural soil pedons, soil types and soil families as related to parent materials and slope classes. By field correlation studies, the engineer determines the engineering characteristics and construction problems associated with these soil groups. This is an analogy technique. It has the advantage that published agricultural soil surveys serve as supplemental control. The aerial photographs are used to compare one area with another and, by analogy, the agricultural soil types are correlated as engineering soil groups.

The other approach to the delineation of soil groups is the use of an engineering soil classification system as a soil identification system in airphoto interpretation studies. This is a deductive technique. The Unified Soil Classification System has merit as a system for delineating soil groups by the technique of airphoto interpretation. A system of this type differentiates mineral soil groups from organic soil groups. Each of these form distinct airphoto patterns. The system is based on a major subdivision of soil textures into coarse-grained soils and fine-grained soils which also form distinct airphoto patterns. The fifteen engineering soil groups are represented by symbols that can be used as mapping symbols and as soil profile symbols. The engineer using these symbols is estimating that, on the average, the soil group designated will occur over the area delineated on the airphotos. The airphoto interpretation and mapping of engineering soil groups by this method requires that within any land form-parent material area, topographic forms and unique areas of tone be differentiated.

The field sampling required, within a given land form-parent material area, to define engineering properties of the soil group will vary. Topographic form and horizon within topographic class must be held constant in the field sampling operation.

ELEMENTS OF FORM

The elements of topographic form that are important in mapping soils are related to the surface expression before erosion (interstream areas) and the surface expression after erosion (gross area). The regional stereoscopic view provided by the aerial photograph allows the engineer to examine the cross-sectional shape of the ground and relate this to areas that show the depositional forms as well as the eroded forms. Land forms of a depositional type or an erosional type have unique topographic expressions. Parent material types also have unique topographic expressions. *Flat, undulating, rolling, crested, blocky* surfaces and *combinations* of surfaces produce different soil types. The more uniform soil types occur on flat topography and the most variable on rolling topography. Deeper development of a soil profile occurs on flat topography while rolling and crested topography exhibit shallow soil profiles and thin soil horizons.

The element of drainage form is particularly significant in land form and parent material analysis. The plan of the drainage and the density of development is also important in interpretation of soil groups. Uniform soils composed entirely of sand size particles do not exhibit an external drainage form but show a swallow-hole type with infiltration basins. Uniform soils composed entirely of clay show a poorly developed dendritic drainage system. Mixtures of various soil fractions develop drainage systems in a complex arrangement in accordance with cohesiveness, stratification, particle size, slope, amount of rainfall, intensity of rainfall, time and other factors.

It is important to emphasize in this context that on eroded surfaces the dendritic and rectangular drainage forms are the most important. They are individually defined to be used to differentiate drift from rock respectfully. The dendritic system is free-flowing without any joint or bedding control. The rectangular system and all its variations is controlled by rock joints, faults, and/or bedding.

The erosional forms are the minor features of the landscape produced by localized concentrations of erosive energy, whether wind or water. Erosional forms produced by eolian activity are blowouts, deflation basins, and streaks. Fluvial activity will produce forms that are related to sheet, rill and gully processes. The density of occurrence of rill and fully forms, their cross-sectional shape and their gradient are important indicators of soil groups. Whenever possible measurements of range in depth, range in gradient, and range in density are determined and correlated with ground control.

ELEMENTS OF TONE

The tones of grey on black and white aerial photographs respond to variations in a wide range of factors primarily due to reflectance geometry (i. e., the relationship between light source, reflecting surface, and camera position). Tones may vary from photograph to photograph, but they are always relative for any time of year and under any photographic processing condition. Elements of tone on aerial photographs can be correlated with land use, crop type, vegetation, soil groups and soil conditions. Elements of tone and textures (assemblies of tone) are difficult to define in words because they are evaluated by individuals with varying sensory perception.

With reference to materials on the surface of the ground devoid of vegetation, the following general conditions prevail as to tone reproduction. (1) The finer the grain size, the lighter the tone due to the sum of reflectances of all surfaces. (2) The higher the surface moisture content, the darker the tone as water absorbs light energy. (3) The darker the natural color of the material, the darker the tone. These ideal conditions seldom are obtained and the civil engineer should never use the conditions alone; they must be leavened with knowledge of other factors, i. e., climate, time, vegetation, etc.

In any case, each photograph and the photographs of an area will exhibit tones and textures that collectively and individually form elements for identification and correlation. Varying land use types produce distinct tonal patterns. Broad classes of vegetation are imaged on photographs in tonal and textural ranges. Rocks produce different tones relative to drift materials, and a sand texture will produce different tones that a clay texture or an organic condition.

The pattern on black and white aerial photographs is made up of individual elements of topography, drainage plan, erosional shape, tones and texture of land use, tones and textures of vegetation, and tones of soil groups and soil conditions. With training the engineer can differentiate these elements and correlate them with engineering soil groups.

COLOR AERIAL PHOTOGRAPHY

Color aerial photography has no significant advantage over black and white photography in evaluating elements of form. Topographic drainage and erosional forms can be identified readily on either type.

Color aerial photography has a significant advantage over black and white in identifying and correlating soils and rocks that are uniquely identified by color in the field. Organic soils can be differentiated from mineral soils more readily because of color. Many massive rock exposures show color differences in the field that are identifiable on color aerial photography. If a geographic area is mantled with eolian drift of silt or sand, then color differences of soil groups may not exist or may not be identifiable. Color differences of soil conditions particularly as influenced by moisture may be more easily identified on color photography. Color may have the advantage of showing the difference between vegetation and bare soil which frequently cannot be determined on black and white photography. Color photography may have more applications in arid and semi-arid areas than in humid or tropical areas. If vegetative types are correlated with soil groups and soil conditions and the vegetation is classified by color images then color photography may be of value. False color (color-infrared) has some value in this regard, and vegetated areas are easily mapped because of the red color.

The cost of a mission to obtain color aerial photography is not much increased over the cost of black and white photography assuming that color corrected cameras are readily available. The cost of the film (whether positive or negative type) is about the only difference. Time of flight and delays of weather may be more critical and, therefore, produce some additional costs.

The reproduction of color prints from positives or negatives produces the most significant increase in costs. In small quantity reproduction, prints cost \$3.00 to \$5.00 each versus \$0.50 to \$1.50 for black and white prints. The color negative system will significantly affect interpretation results when color prints are obtained economically. For important soil group mapping projects or special engineering site investigations, color photography may be completely justified on the basis that the additional information provided by the color of the image outweighs the additional cost of color prints. Color negative types of film have the additional advantage that black and white prints and diapositives can be made for photogrammetric mapping purposes.

INFRARED AND MICROWAVE IMAGERY

The infrared and microwave regions also may be of interest to the civil engineer in site selection studies. Imagery obtained in these regions of the electromagnetic spectrum supplements that obtained in the visible. Side looking radar (SLAR) imagery is invaluable in regional site selection studies where reflections from various terrain types create patterns for identification. These are not standard airphoto patterns, but with training an engineer can interpret soil-rock conditions over broad geographic areas.

Unique thermal conditions of soils, rocks and soil-water systems may be recorded on infrared imagery. Thermal reflections and thermal emissions from surface and shallow subsurface features, in certain cases, may assist in detecting changes in soil conditions. Our understanding of the reasons for the patterns produced on infrared imagery is far from complete. Additional research is required. At the present time infrared imagery is a supplement to standard photography. If overflights can be made at both maximum and minimum thermal conditions, then the changes on the imagery can be correlated with field control to further define unique soil groups.

SUMMARY

The civil engineer can recognize on black and white photography, color photography, infrared imagery and radar imagery changes between one area and another. These changes produce patterns characteristic of land form-parent material types. Correlations can be made between land form-parent material types and engineering soil groups as influenced by topographic changes and environmental factors of vegetation and climate. Unique engineering soil groups can be separated and mapped. Statistical sampling in the field can define the engineering properties of the soil groups differentiated. As areas are sampled, direct correlations with airphoto patterns and imagery patterns can be made and field studies reduced in quantity for future projects. Field effort can be devoted to investigation of problem areas where economy and better performance can be achieved by engineering analysis and design.

Civil engineers can develop their ability to interpret soil groups from aerial photographs. Correlations gleaned from the literature in geology, geomorphology and soil science are of significant value in developing the technique of interpretation. Formal courses, such as the three course sequence offered in the School of Civil Engineering at Purdue University, can shorten the time span so that anyone can interpret engineering soil groups from aerial photographs.

Combined Investigation Techniques for Procuring Highway Design Data

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ABSTRACT

Several combinations of direct and indirect soils exploration techniques (combined-technique methods) have been evaluated over a variety of terrain in Ohio. The components of the combined-technique methods are air-photo interpretation, electrical resistivity and seismic refraction surveys, and the currently acceptable boring methods. Research indicates that selected technique combinations furnish suitable highway design data in certain regions of the State.

Ohio can be subdivided into four regions on the basis of bedrock stratigraphy or thickness of glacial drift. For each of these regions a selected technique combination is recommended for procurement of soil and rock data for use in highway design. Two additional technique combinatione are recommended for the special investigations required for soft-subsoil and landslide-susceptible regions.

INTRODUCTION

From 1961 to 1965, research on terrain investigation techniques was undertaken by the Engineering Experiment Station at Ohio State University. The research was sponsored jointly by the Ohio Department of Highways and the Bureau of Public Roads.

The purpose of the research was to determine which combinations of indirect and direct soils exploration techniques supplied data sufficiently reliable for preliminary highway design.

The research consisted of three phases. First, all indirect and direct soils exploration techniques were reviewed. This phase of the program was followed by the selection and field testing of the techniques that provided the greatest amount of qualitative and quantitative highway engineering data. The final phase of the research program was the evaluation of the technique combinations with respect to the variations of the Ohio terrain and special investigation problems. This paper summarizes the findings and recommendations which resulted from the research.

The indirect soils exploration techniques are: (1) airphoto interpretation using panchromatic, infrared, and color photography at several film-filter-scale combinations; (2) electrical resistivity surveys using two field and four interpretive methods; and (3) seismic refraction surveys using two interpretive methods. Widely spaced auger, core-bearing, split-spoon or press-sampling data were obtained for calibration purposes and to supplement the indirect technique data.

Airphoto interpretation was selected because it furnishes continuous lateral data on the extent, texture, and composition of the shallow soils and bedrock. Electrical resistivity and seismic refraction surveys were selected to confirm the airphoto interpretation of the shallow subsurface. The geophysical surveys complement the airphoto interpretation by adding information about the texture and layering of the soil, the position of top of bedrock, and the lithology and stratification of the bedrock. Widely spaced direct method soil investigation techniques were selected to verify the indirect technique interpretations and furnish samples for both field and laboratory analysis.

Analysis of the data collected through the use of the technique combinations at study sites in differing terrains indicates that six technique combinations can be recommended for use in Ohio. Figure 1 shows the locations where the technique combinations were tested.

RECOMMENDED COMBINATIONS OF TECHNIQUES

Ohio can be subdivided into four regions on the basis of bedrock stratigraphy or the thickness of the glacial drift. For each of these regions selected technique combinations and procedures are recommended for the procurement of soil and rock data for use in highway design. Two additional technique combinations are recommended for special problems investigations. The next paragraphs give the applications of the technique combinations for the terrain types indicated.

Technique Combination I

Technique Combination I is recommended for procuring data on upland terrain where residual soils are thin and the bedrock sequence consists of alternating layers of thin to medium-bedded limestone and shales or inter-bedded shales, sandstones, siltstones and coals. These rock sequences occur in parts of the outcrop belt of the Ordovician, Mississippian, Pennsylvanian and Permian rocks of Ohio. A generalized distribution of the terrain where Combination I is applicable is shown in Figure 2.

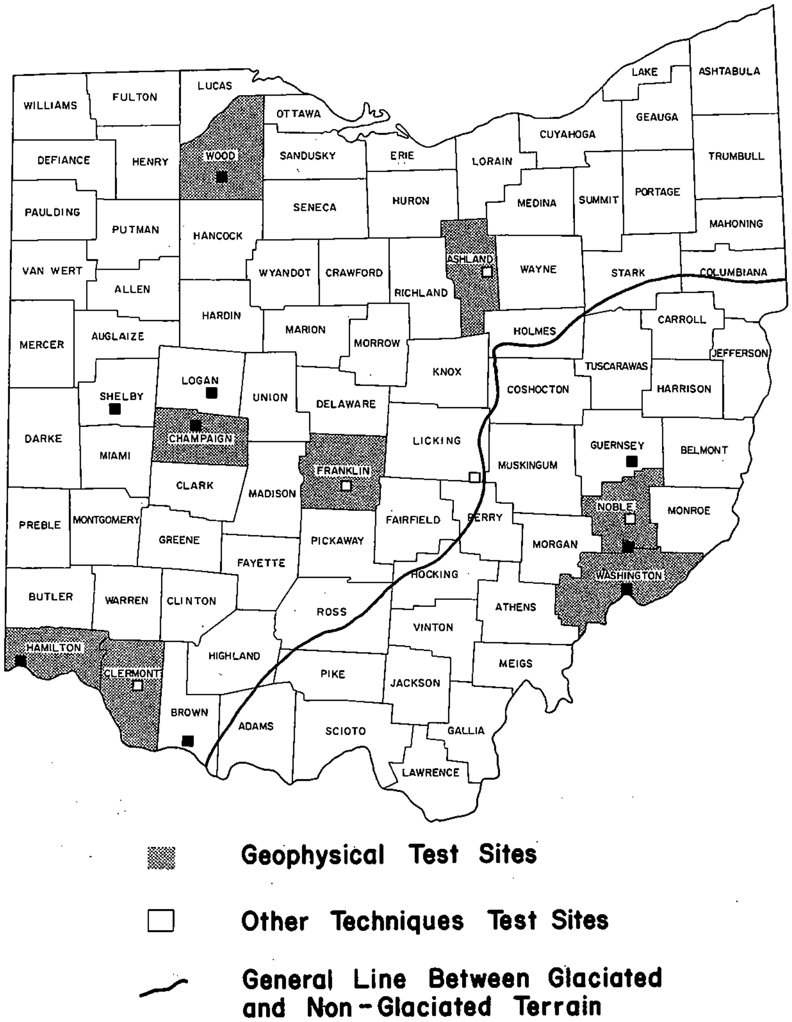
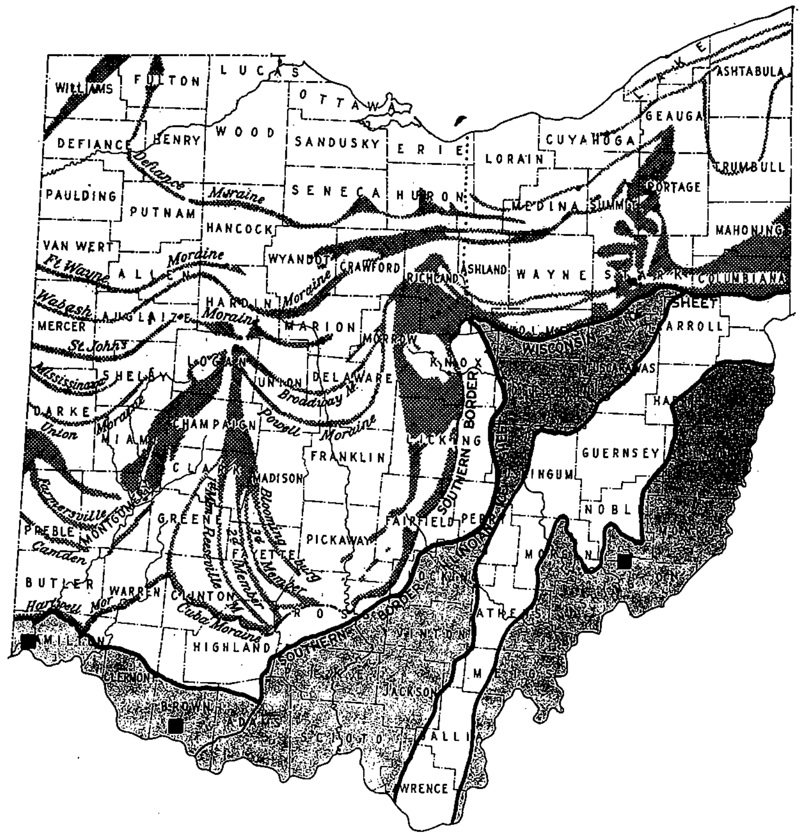


Figure 1. The locations where the Technique Combinations were tested in Ohio.



Recommended Areas for a Combination of Airphoto Interpretation Electrical Resistivity Surveys and Core-Borings

Sites of Field Investigations

Figure 2. A generalized distribution of the terrain where Technique Combination I is applicable. (After Ohio Geological Survey)

The combination consists of airphoto interpretation using panchromatic film at scales of 1:20,000 and 1:9600, electrical resistivity surveys, and core-borings. Electrical resistivity surveying is recommended as the geophysical component of the combination, because in alternating sequences of hard and soft strata it furnishes the most accurate interpretation of the bedrock stratigraphy. In this type of terrain velocity inversions occur when a dense stratum overlies a stratum of lower density, and the seismic method does not permit accurate interpretations of depths to interfaces below the first velocity inversion.

This technique combination was tested in Brown, Hamilton, and Washington Counties, Ohio. The results of the tests indicated that the combination furnished useful subsurface-depth data concerning major stratigraphic interfaces in both the soil and rock sections. Interpreted depths from resistivity data were generally within ± 3 feet of the depths indicated in the borings. Lateral soil changes and the dominant soil texture were interpreted correctly from air-photo and electrical resistivity data in nearly every case. However, the combination did not have the resolution to permit accurate interpretation of soils of similar textures, i.e., soils with equal mixtures of silt and clay could not be differentiated from silty clays.

Technique Combination II

For terrain investigations in areas where glacial drift is greater than 25 feet thick, Technique Combination II is recommended. The general areal distribution of the terrain where Combination II is applicable is shown in Figure 3. The glacial deposits in these areas include ground moraine, end moraine, lacustrine deposits, glacial outwash, and alluvium.

The combination consists of airphoto interpretation using panchromatic film at scales of 1:20,000 and 1:9600, electrical resistivity surveys and power auger or split-spoon sampling.

The selection of the components for Combination II is based primarily upon the general thickness of the soil layer and also in a limited way upon the data required for preliminary design.

Since depth to, and attitude of, the bedrock surface are generally not design considerations in thick overburden areas, the electrical resistivity technique is recommended instead of the seismic refraction method. Data from all test areas indicate that interpretations of electrical resistivity data provide a more detailed and accurate profile of the soil stratigraphy than do seismic interpretations.

Technique Combination II was tested in Shelby, Logan, and Champaign Counties, Ohio. Evaluation of the test data indicated the combination furnishes useful data for preliminary highway design. Interpreted depths from resistivity data for major soil discontinuities were generally accurate to ± 3 feet. Lateral soil changes and

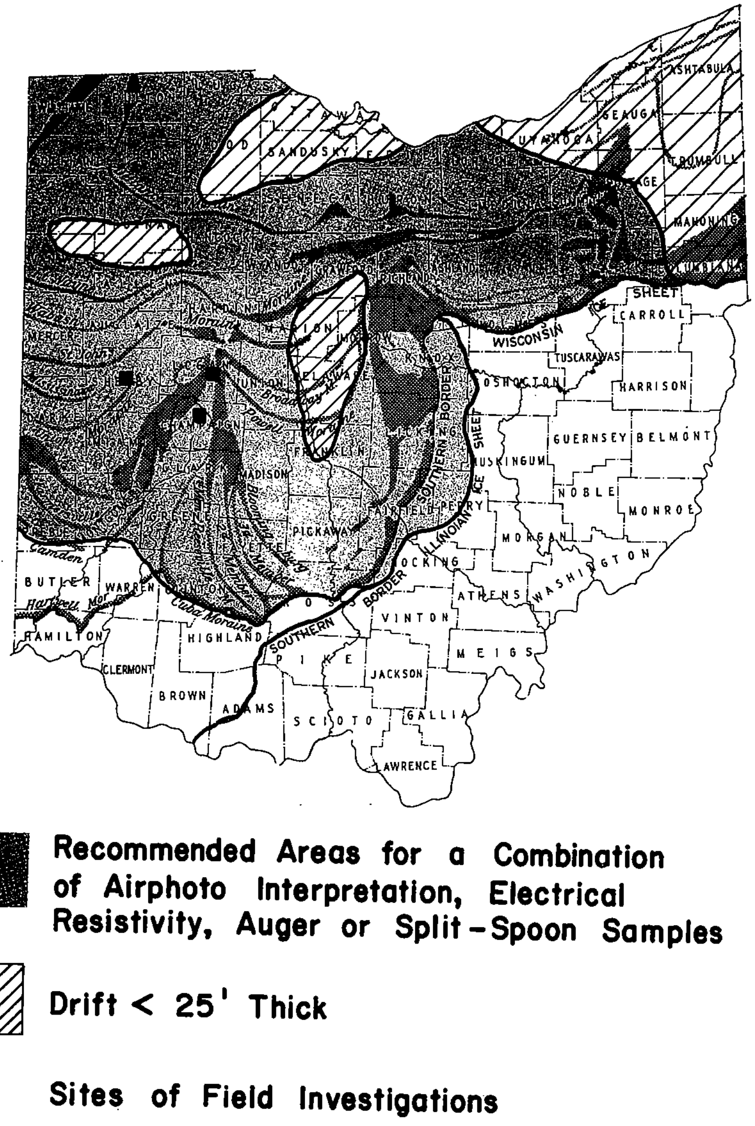


Figure 3. The general areal distribution of terrain where Technique Combination II is applicable. (After Ohio Geological Survey)

the dominant soil textures were interpreted correctly from airphoto and resistivity data. However, from the data obtained by this combination, we could not accurately interpret the occurence of soil layers less than one-foot thick or to make separations between silt and clay and silty clay.

Technique Combination III

Technique Combination III is recommended for terrain in Ohio where the thickness of glacial overburden is less than 25 feet thick. The landforms in these areas are ground moraine, lacustrine deposits, beach ridges, dunes, and floodplain deposits. The general distribution of this type of terrain in Ohio is shown on Figure 4.

The combination consists of airphoto interpretation using panchromatic film at scales of 1:20, 000 and 1:9600, electrical resistivity surveys, seismic refraction surveys and power auger or split-spoon sampling.

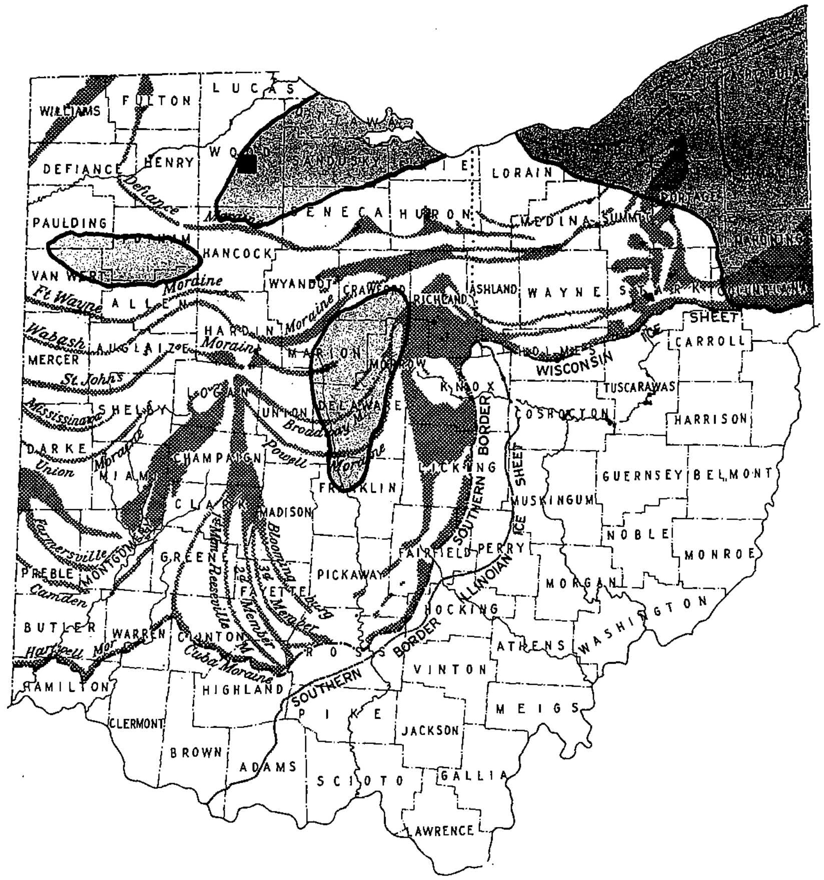
The thickness of the glacial drift is the primary consideration in the selection of the components of the combination, but design criteria are also considered. Where bedrock is near the surface and affects the design, the seismic refraction method is included in the combination because more accurate depths to bedrock can be interpreted from the seismic data. Since detailed soil stratigraphy is also an important consideration, the electrical resistivity data are needed to supplement the seismic survey data.

Combination III was tested in Wood County, Ohio. The soil profile data interpreted from the techniques of Combination III correlated well with the direct method profile furnished by the Ohio Department of Highways. Lateral changes in soil textures were interpreted correctly from airphoto and electrical resistivity data. Subsurface soil layers and the dominant subsurface soil textures were interpreted correctly from electrical resistivity data. The depth of major soil interfaces were generally accurate to within ± 3 feet. The seismic refraction interpretation was accurate to within ± 2 feet in determining the top of bedrock, and the seismic data clearly indicated the composition as crystalline limestone and dolomite.

Technique Combination IV

Combination IV is recommended for areas where the terrain consists of upland residual soils developing from predominantly shale sequences. This terrain in Ohio is restricted primarily to the outcrop belt of the Conemaugh Series of Pennsylvanian age. The areal distribution of the Conemaugh is shown in Figure 5.

The combination consists of airphoto interpretation using panchromatic film at scales of 1:20, 000 and 1:9600, seismic refraction surveys and core-borings. The selection of the components of Com-

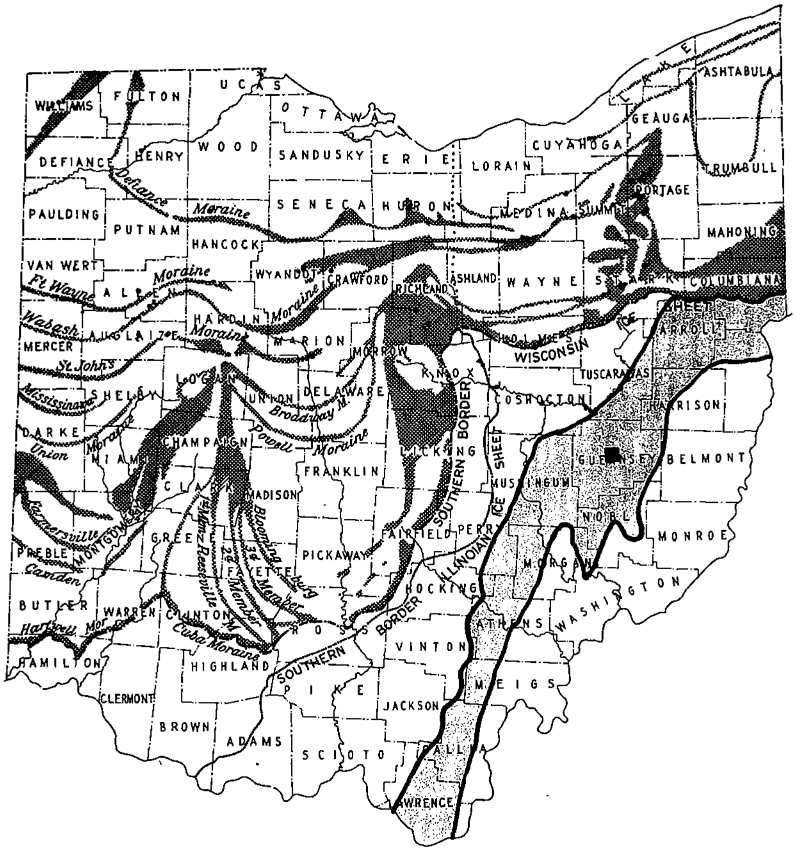


Recommended Areas for a Combination of Airphoto Interpretation, Electrical Resistivity and Seismic Refraction Surveys and Auger & Core-Borings

Drift > 25' Thick

Site of Field Investigation

Figure 4. The general distribution of glacial terrain in Ohio. (After Ohio Geological Survey)



Recommended Area for a Combination of Airphoto Interpretation, Seismic Refraction, Core-Borings

Site of Field Investigation

Figure 5. The areal distribution of the Conemaugh (Pennsylvanian) outcrop belt in Ohio. (After Ohio Geological Survey)

bination IV is based primarily upon the bedrock stratigraphy but in a limited way upon design considerations.

Seismic refraction is recommended as the geophysical component because the seismic interpretation provides a more accurate depth to top of bedrock in areas where the soils are thin (less than 10 feet) and the underlying bedrock is predominantly shale. In areas of thick shales, velocity inversions are minimized because very few high velocity interbeds exist.

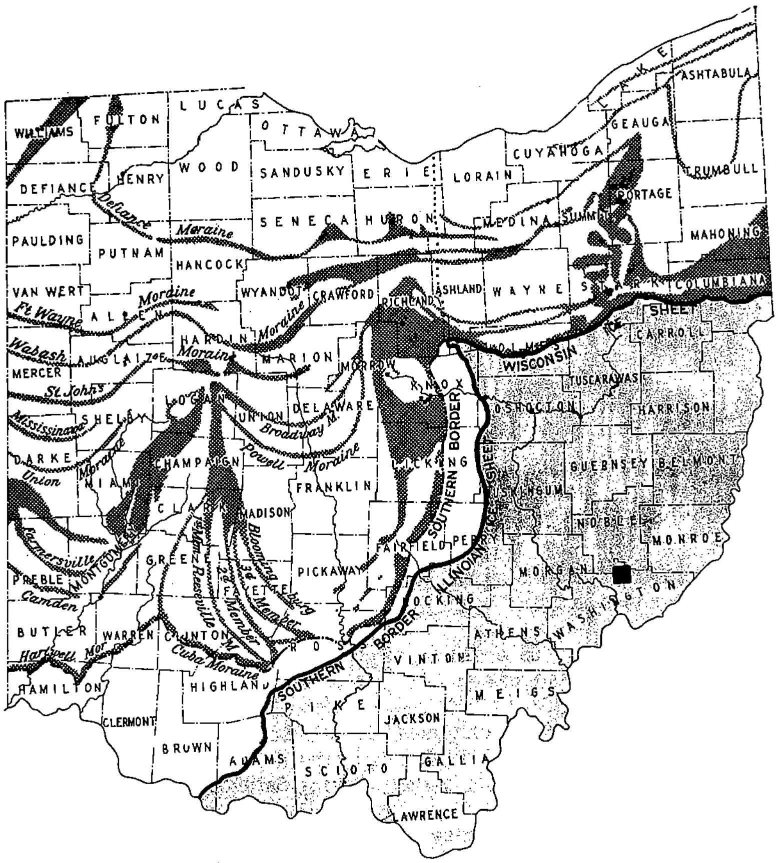
Technique Combination IV was tested in Guernsey County, Ohio. Lateral changes in soil texture were interpreted correctly from the combined airphoto and seismic refraction data. The thickness of the soil layer, the depth of weathered bedrock and the top of sound bedrock were correctly interpreted from the seismic refraction data. Interpreted depths to the rock/soil interface were generally accurate within ± 3 feet.

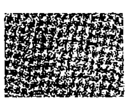
Technique Combination V

Technique Combination V is recommended for procuring data on stratigraphy, areal extent and classification of failed slopes.

The combination consists of photo interpretation with Panchromatic Tri-X film exposed through a Wratten 38A filter at a scale of 1/2400, electrical resistivity surveys and core-borings. Combination V permits accurate identification of the bedrock stratigraphy in the highwalls of strip mines or in rock cuts of existing highways where stratigraphic units are identifiable over fairly wide areas. Photography obtained with this film-filter-scale combination reveals stratigraphic indicators of terrain susceptible to landslides. For example, the identification of the thin Ames limestone exposed in slopes or in road cuts locates the Conemaugh shale which is landslide susceptible in southeastern Ohio. Once the Conemaugh sequence is identified in the 1/2400 scale photos, it can be laterally extrapolated over large areas. Figure 6 shows the areal distribution of landslide-susceptible terrain.

Interpretation of infrared panchromatic film exposed through a Wratten 21 filter, at a scale of 1/2400, flown in the Spring before a vegetative cover grows, permits the designer to interpret the areal extent and classification of existing landslides and unstable conditions along a proposed alignment. This film-filter-scale combination has a range of sensitivity between 550 and 900 millimicrons. Blue and green register dark, while browns and reds are light-toned. The maximum light tones represent colors around the 700 millimicron wave length, so contrasts are expected between blue-green and red objects. Color contrasts between green vegetation and the red and brown soils exposed in slide scarps, reflected by contrasting photo tones, assist the interpreter in locating incipient slope failures.





Recommended Areas for a Combination of Panchromatic Tri – X Wratten 38 A Filter, Electrical Resistivity Surveys and Core – Borings

Site of Field Investigation

Figure 6. Areal distribution of the landslide - Susceptible terrain of Southwestern Ohio (After Ohio Geological Survey)

Where bedrock is exposed, gray rocks are the darkest, while the red rocks are the lightest toned. The areal extent of scarps and breaks in slopes are easily delineated where the contrast in gray tones represent gray vs. red rock. In the test site, wet areas occurring on the slopes emphasize the changes in stratigraphy. In terms of areal extent, the red-brown soil areas, which failed most often, are emphasized by this film-filter-scale combination.

In the Washington County test site, accurate data regarding bedrock stratigraphy, location of seepage zones, and the identification of glide-plane elevations of existing slumps and slides were interpreted from the electrical resistivity surveys. Rock-core data complemented the airphoto and geophysical interpretations and locally determined the thin residual soils, the location of glide planes and the bedrock layering and composition.

Technique Combination VI

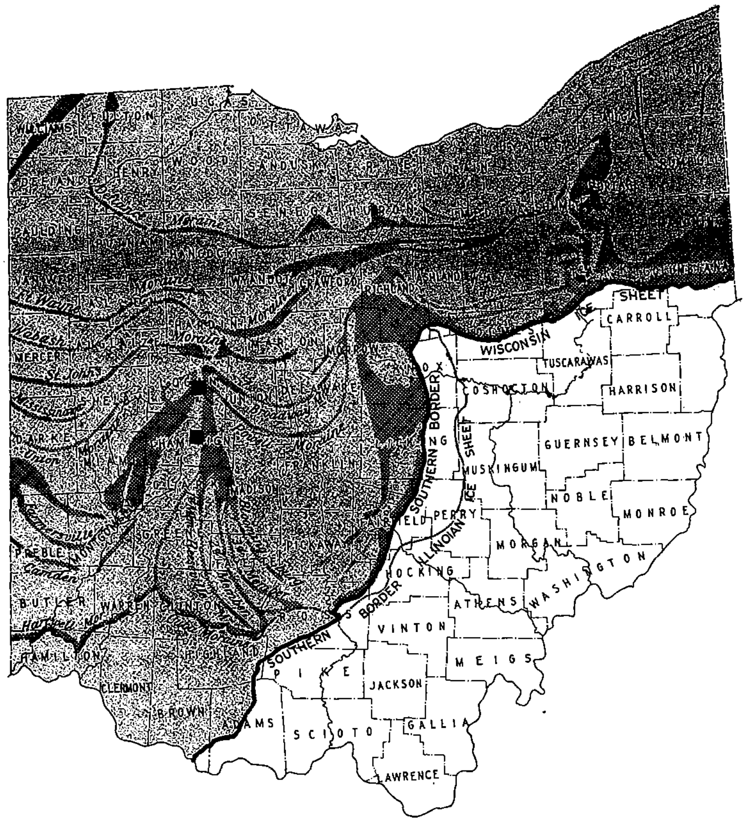
Technique Combination VI is recommended for special investigations of soft-subsoil materials in areas where the glacial drift layer is greater than 25 feet thick. The areal distribution of glacial drift deposits in Ohio is shown in Figure 7.

The combination consists of photo interpretation, electrical resistivity, power-augers and/or press-samplers.

The selection of techniques for this combination is based primarily upon the general thickness of the soft subsoils and also in a limited way upon the type of engineering data required.

Combination VI includes the interpretation of panchromatic Tri-X film exposed without a filter at a scale of 1/4800 or larger and flown during the Autumn season. Photo interpretation utilizing this film-filter-scale combination identifies the existence and delineation of most soft-subsoil deposits.

At a peat study site located along a portion of the proposed relocation of U. S. Route 68 just north of Grimes Airpost, Champaign County, the field tones varied from green to brown to a yellowish tan. The soil color varied from gray to brown to black. With this film-filter-scale combination, the range of sensitivity is in the 450 to 575 millimicron band. Light-blues and light-green to greens produce subdued light photo tones; oranges, browns and reds produce dark photo tones and the shadows are dark-toned. Hence, a green weed patch appeared light-toned, a mature corn covered area was dark-toned, the shadow in the ditch was dark-toned and the dry black soil was gray-toned. A brown stubble-hay field underlain by brown top soil produced an intermediate tone in contrast to the dark-toned corn field. A depression, occupied by a corn field emphasized the peat-muck area; this dark-toned area made the muck easily detected by the photo interpreter.



Recommended Areas for a Combination of Panchromatic Tri-X Film Exposed without Filter at a Scale of 1/4800 or Larger, Electrical Resistivity Surveys and Auger or Split-Spoon Samples

Sites of Field Investigations

Figure 7. Areal distribution of terrain where Technique Combination VI is applicable (After Ohio Geological Survey)

The electrical resistivity survey delineated the areal extent and depth of soft-subsoil deposits. The resistivity interpretation of the peat-glacial till contact corresponded well to a change in slope that was observed during the stereoscopic study of photography. This change in slope enabled the interpreter to mark the areal extent of shallow depression. By extrapolating the topographic expression it was possible to delineate the entire areal extent of soft-subsoil deposit.

In a test site in Logan County an electrical resistivity survey was successfully used to determine the distribution and depth of a large soft-subsoil deposit. A constant-spacing electrical traverse indicated lateral changes in electrical resistivity for the upper 20-foot of soil layer and defined the soft subsoil-glacial till boundary. At one electrical depth-profile station the depth of peat was indicated between 39 and 40 feet. A calibration boring found the peat at a depth of 37 feet. At another station the depth of peat was interpreted between 18 and 21 feet from the electrical resistivity data, whereas the soil boring indicated the soft peat subsoil at a depth of 18 feet.

SUMMARY

Six combinations of terrain investigation techniques have all been tested in the field and the interpretation of the combined data shows that useful highway design data can be obtained by the combinations within the terrain types for which the combinations have been recommended. In all cases the airphoto interpretation supplied useful, continuous lateral data on the distribution, texture, and composition of the shallow soils. For special problem investigations, infrared panchromatic film (as shown in Figure 8) and conventional panchromatic Tri-X film (as shown in Figure 9), exposed at the recommended filter-scale combinations, offer excellent means of acquiring specific data in landslide-susceptible and soft-subsoil terrain. The geophysical survey techniques complement the airphoto interpretation by confirming the airphoto interpretation data and supplying additional information on the subsurface soil and bedrock discontinuities, textures and compositions.

The most accurate interpretations from the electrical resistivity data were obtained when the depth-profiling technique was employed with electrode spacing increments of 3 or 5 feet. The Barnes-layer method and the Moore-cumulative method of interpretation provided the most accurate subsurface interpretations of the resistivity data.

With respect to the seismic refraction surveys, the critical-distance method of interpretation furnished the most accurate data on the depth of bedrock.

Boring, split-spoon and press-sampler data complemented the

Figure 8. A pancromatic infrared photograph of the landslide site, exposed through Wratten 21 filter, scale 1/2400

Figure 9. Panchomatic Tri-X, photograph exposed without filter (Scale 1/4800) at the soft subsoil site

geophysical techniques by providing calibration data for the geophysical techniques, by confirming the indirect method interpretations, and by providing samples for field and laboratory analysis.

A tabular summary of the recommended combinations for selected Ohio terrains appears below.

SUMMARY OF SELECTED COMBINATIONS OF INDIRECT AND DIRECT TECHNIQUES; FOR PROCUREMENT OF SOIL AND ROCK DATA FOR HIGHWAY DESIGN

The Recommended Techniques for General Terrain Conditions

TERRAIN	Thin Overburden of residual soils
TECHNIQUES	Photo interpretation, electrical resistivity surveys and calibration core borings
TERRAIN	Upland, residual soils-shales
TECHNIQUES	Photo interpretation, seismic refraction surveys and calibration core borings
TERRAIN	Upland plains and moraines, thick glacial drift, greater than 25 feet
TECHNIQUES	Photo interpretation, electrical resistivity surveys and power-augered calibration soil data.
TERRAIN	Upland, lake plains and moraines, thin glacial deposits, less than 25 feet thick
TECHNIQUES	Photo interpretation, electrical resistivity surveys and seismic refraction surveys and power-augered soil samples and/or calibration core borings.

SUMMARY OF SPECIAL COMBINATIONS OF INDIRECT AND DIRECT TECHNIQUES, FOR PROCUREMENT OF SOIL AND ROCK DATA FOR HIGHWAY DESIGN

The Recommended Techniques for Special Problems Investigations for Soft-subsoil and Landslide-susceptible Terrain

TERRAIN	Soft soils and sub-soils-peat, muck, fine-grained, water-logged soils
TECHNIQUES	Photo interpretation using panchromatic film exposed without filter at 1/4800 scale, electrical resistivity and press-sample and/or power-augered calibration borings

TERRAIN	Landslide-susceptible slopes, stratigraphy
TECHNIQUES	Photo interpretation using B & W film exposed through Wratten 38A filter at 1/2400 scale, complemented with electrical resistivity surveys and calibration core borings.
TERRAIN	Landslide-susceptible slopes - shape, extent and classification of failed slope
TECHNIQUES	Photo interpretation using infrared panchromatic film at 1/2400 scale, exposed through Wratten 21 filter, (or Ektachrome film at 1/4800 scale, or Infrared Ektachrome Film at 1/1200 scale) complemented with electrical resistivity surveys and calibration core borings

Comprehensive Investigations Facilitate Design of Interstate Highway Over Bottomland Soils

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ABSTRACT

Reported are investigations for 15 miles of I-255 near East St. Louis, Illinois. The major problem involved: determining the extent, properties and treatment of extensive, irregular and frequently buried deposits of soft organic and compressible soils.

Described are: development of the exploration program; sampling and testing methods, presentation and analysis techniques and partial results.

Non-routine aspects of the program included: extensive use of split-spoon borings (vs. auger borings), relatively deep pilot borings, swamp probes and undisturbed sampling; cone penetrometer soundings, vane shear tests and ground-water observation wells; standard and special consolidation and triaxial tests; economic studies of borrow sources and swamp treatments; and, complete documentation of the investigations.

The Use and Abuse of Geophysics in Highway Engineering

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ABSTRACT

The two geophysical methods most commonly used in highway engineering problems are the refraction seismic and electrical earth resistivity methods. Proper use is dependent upon thorough knowledge of a) what earth properties are involved; b) how the method utilizes these properties; c) how the local geology affects the results and d) the field procedure best suited to the problem. Lack of attention to any combination of these factors may result in a poor survey and the unforuntate development/of negative bias toward a good method, improperly used. Much useful engineering data may be obtained from a survey that is classed as a failure as in the case where seismic depths may be in error but the velocities still faithfully record the elastic properties of the buried materials.

The geophysical methods most used in highway engineering projects are the refraction seismic method and the electrical earth resistivity method. Both are used to evaluate site conditions and for the location of aggregate materials. In addition, they may be used to locate sources of groundwater for rest and service facilities along a right of way. With the advent of reasonably priced equipment, often accompanied by glowing reports of successful use, it is necessary to review what the methods can and cannot do for the highway engineer or geologist.

Neither method may be considered a source of primary data because of the need for subsurface control for evaluation of results. Basic to the use of either method is a thorough understanding of the operating principles. In our discussion this will include a knowledge of the energy used, how it is used and recorded and the effect of the geological or subsurface conditions on the energy distribution. Each method will consistently provide quantitative data about the subsurface when used. However, the answer to a particular problem may not be obtainable from the data. Only knowledge of the method and local conditions will permit the operator to discern reliable information from misleading (not incorrect) information. In actual practice, the site should be evaluated concerning the probability of successful use of a geophysical method before it is ever applied in the field. It is easy to believe the reports of some manufacturers that the successful use of the seismograph, for instance, requires only that time breaks on a refraction record be read and plotted and plugged into a simple equation for a two-layer problem. This approach can only lead to complete disenchantment with a method that has much to offer when properly used. It should not be expected to provide results that are precluded by the local conditions. The same may be applied equally well to the resistivity method.

The refraction seismic method will be discussed first because the nature of the output data and the calculation methods create an aura of credibility that is not always justifiable. The method operates on the principle of propagating compressional waves through subsurface materials. The rate of wave propagation is a function of the combined physical properties which affect the elasticity of the material. The various combinations of texture, composition, porosity and consolidation result in ranges of propagation speed in any rock or soil of given general name, such as limestone, sandstone, glacial till, etc. The ranges in turn overlap so that without control it is not safe to name a subsurface unit from its velocity alone. An example is a glacial soil that because of composition and consolidation will have the velocity of a shale or sandstone. At this point we often go astray. We have found what we should have been looking for, namely an indication of the physical properties of the material. However, we are so conditioned to wanting to name everything, that we bypass the obvious, pin a name on the velocity unit and then feel the method has failed when exposure of the material reveals something else.

During the average site inspection we may encounter subsurface conditions that cannot be mapped in terms of depth by the refraction method regardless of how much control we have. But the method is often called upon under such conditions to map the area and then condemned because of the magnitude of error shown between seismic results and drill hole or excavation data. In my younger days I have been sent on surveys, by superiors who knew just enough of the method to be dangerous, to map depth to rock where we did not stand a chance of success. The poor direction was the result of lack of knowledge about the restrictions imposed by the refraction method and/or lack of knowledge of the geology of the area. The classic error-causing conditions are a) where a low-speed zone such as an outwash deposit lies between an overlying till and the underlying rock surface; b) a well-consolidated till overlying a rock of lower or the same velocity; c) insufficient thickness of beds at depth to propagate the energy long enough to be received at the surface before arrival of the next fastest underlying layer and d) lateral changes in velocity in the surface soil, for instance, that will alter the recorded velocities along the profile and thus the depth calculations. The usual calculation techniques provide no solution to a, b, or c. Adjustment of field layout is one way of solving d. A survey may be conducted intentionally under such conditions listed above, however, if we are interested, as we should be, in getting information which does not always have to be in terms of depths. For some reason, depth determinations are the only thing many expect of the refraction method. What if we should work over an irregular buried gravel deposit? We will encounter attenuation of energy of wave fronts that arrive from material below it wherever it is present in thicknesses great enough to be important. Discontinuous sections in the travel-time graph may also be noted. Graphical or iterative solutions may be used to estimate thickness but generally this is beyond the means of the survey and the main task has been accomplished, namely, to locate an anomalous point where a boring is needed.

Although more could be said about operations such as tailoring the field layout to the local conditions, etc., I would like to terminate the discussion of the refraction seismic method with a plea to its users. That is, to become acquainted with wave-front diagrams for a variety of conditions encountered in practice. Ray-path diagrams are useful but often misleading. It is only through construction of wave-front diagrams that the engineer or geologist in charge of a survey can visualize the effect different velocities, thicknesses, bed sequence and attitude can have on the design of the geophone layout and length of spread, the generation of apparent velocities by dipping beds and the travel-time graph as being representative or not of the true subsurface conditions. Every highway engineer or geologist who uses the refraction method should be thoroughly familiar with this even if we have to conduct short courses just for that purpose.

The electrical earth resistivity method is not abused as much judging from my experience. The low precision of the data and the qualitative interpretation methods used do not normally mislead the individual into expecting too much from the method. However, we often attach too much meaning to the magnitude of a reading forgetting that we are dealing with conductivity of a) the pore water in a rock or soil and b) the mass effect of combined layers of soil or rock with increasing depth. Certain interpretation methods are helpful in eliminating the second problem. I have made surveys for buried low resistivity clay deposits under high resistivity alluvial sands. The clays were successfully mapped but at values of 100, 000 ohm-cm or more where the indication of the clay was relative to the high base or background resistivity imposed by the surficial sands. Thus, the values are not unique but the inflections in the plotted values are of great interpretive value, given some control.

With the resistivity method it is possible to obtain similar values for different materials or conversely, different values for similar materials because we are measuring fluid conductivity and not the physical properties of the material. The method may be an outstanding success or equally outstanding failure in the eyes of its user for the same reasons as given for the refraction seismic method, namely: the state of knowledge of the method and the local geologic conditions. The method may complement the seismic method because it is ideally suited to mapping buried sands and gravels that are difficult to map seismically. A thorough survey of an area can be done best by combining the two methods if the geology indicates the presence of such problems.

In conclusion, proper use of either the refraction seismic or the electrical earth resistivity method requires that we understand what is being measured, how it is being measured and the influence of the local subsurface conditions on the reception of shock waves or voltages. Only when we meet these requirements will we use the equipment properly and not expect the impossible from two valuable subsurface exploration methods.

A Foundation Problem in Cavernous Dolomite Terrain, Pulaski County, Missouri*

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ABSTRACT

The catastrophic formation of a sinkhole in a dry valley in central Missouri provided an opportunity to investigate a cave system passing partly beneath U. S. Highway 66. The cave contained several vertical shafts which are potential geologic hazards to the highway. Foundation problems in similar limestone terrains may be avoided by early recognition of surface indications of cavernous bedrock, including underfit streams, dry streambeds, poorly graded alluvium, absence of terraces, low water tables, absence of bedrock outcrops, sinkholes, and abrupt changes in valley profiles. In addition to detailed drilling of suspect areas, seismic and resistivity methods may be used to detect geologic hazards in cavernous limestone terrains.

INTRODUCTION

A sinkhole appeared in late February, 1966 in the bed of a dry valley at the downstream end of a box culvert beneath U. S. Highway 66 near Buckhorn in Pulaski County, Missouri. The authors, upon invitation of the Missouri State Highway Department, investigated the sinkhole, which proved to be the entrance to a sizable cave system, and discovered several large, potentially hazardous vertical shafts beneath compacted highway fill. Complete investigation of the cave system beneath the dry valleys at this locality strongly emphasized the need for detailed foundation investigations beneath dry valleys in similar localities.

We gratefully acknowledge assistance given us by C. Helmer Turner, then District Geologist, Missouri State Highway Department, Springfield, Missouri. Dr. Alden B. Carpenter, University of Missouri at Columbia, Columbia, Missouri made x-ray analyses of minerals from the cave, and our colleagues on the staff of the Missouri Geological Survey made many helpful suggestions.

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* Publication authorized by the Director, Missouri Geological Survey and Water Resources.

REGIONAL GEOLOGY

Pulaski County in southern central Missouri (Fig. 1) is in a region underlain by cavernous dolomites of Ordovician (Canadian) age. The extent of cavern development is best shown by the more than 200 known caves in Pulaski County; no other county in the nation has as many. Much of the drainage is subterranean; tributary valleys commonly are dry, and the trunk streams are fed by springs. A few miles to the north of the Buckhorn area, the master stream, the Gasconade River, becomes a "sinking river", losing most of its flow for a time to channels beneath its bed. The flow is restored downstream in the form of springs and bed resurgences (Bolon, 1952).

Pulaski County lies wholly within the Salem Plateau of the Ozarks province (Bretz, 1965). U. S. Highway 66 generally follows a divide trending NE - SW across the plateau. Consequently, the topography in the vicinity of Buckhorn is characteristic of the divide-gently rolling uplands, broad, sinuous ridges with accordant summits, and frequent sinkholes on the uplands (Fig. 2). Average elevation of the ridge summits is 1000 to 1200 feet. To the north and south of the divide the topography is much more rugged, a region of maturely dissected Palezoic bedrock with local relief exceeding 600 feet. Upland valleys characteristically are broad, with flat, gravel-choked floodplains. Underfit streams are the rule rather than the exception. There are few terraces. Structural control of physiographic features is easily descernible, particularly in valley segment orientation.

Three bedrock formations of Ordovician (Canadian) age crop out in the Buckhorn area. Together they form a limestone (dolomite) terrain of unusually high porosity. As mentioned previously, there are more than 200 known caves in Pulaski County; many more will remain hidden beneath a thick mantle of residuum until some chance event such as the catastrophic formation of a sinkhole or pentration by water well drilling reveals their presence. These strata form excellent ground water aquifers. Their description follows:

Jefferson City Dolomite A fine-grained, evenly bedded, slightly cherty dolomite. It crops out as minor ledges on the broad upland ridges, but elsewhere it is generally soil covered. The rock weathers evenly, and characteristically has a uniform soil cover five to ten feet thick.

Roubidoux Formation An interbedded sequence of dolomite, sandy dolomite, sandstone and chert, ranging from 110 to 130 feet in thickness. Many of the dolomite and chert beds exhibit relict reef structure. The Roubidoux weathers deeply, producing a residual soil varying in thickness from 25 to 50 feet. The residual soil is a poorly consolidated, poorly graded debris of chert and sandstone boulders mixed with sand, silt and clay, resulting from removal of carbonates by solution. Relict structure is commonly present in the residuum. Both the residuum and the Roubidoux bedrock are moderately to highly permeable.

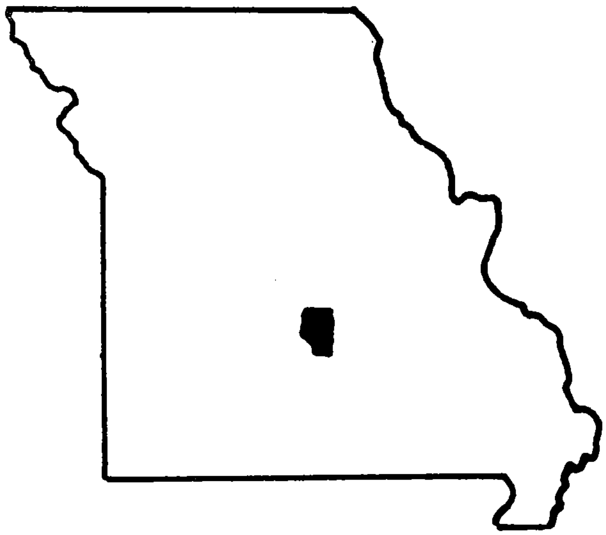

Figure 1. Key map showing location of study area, Pulaski County, Missouri.

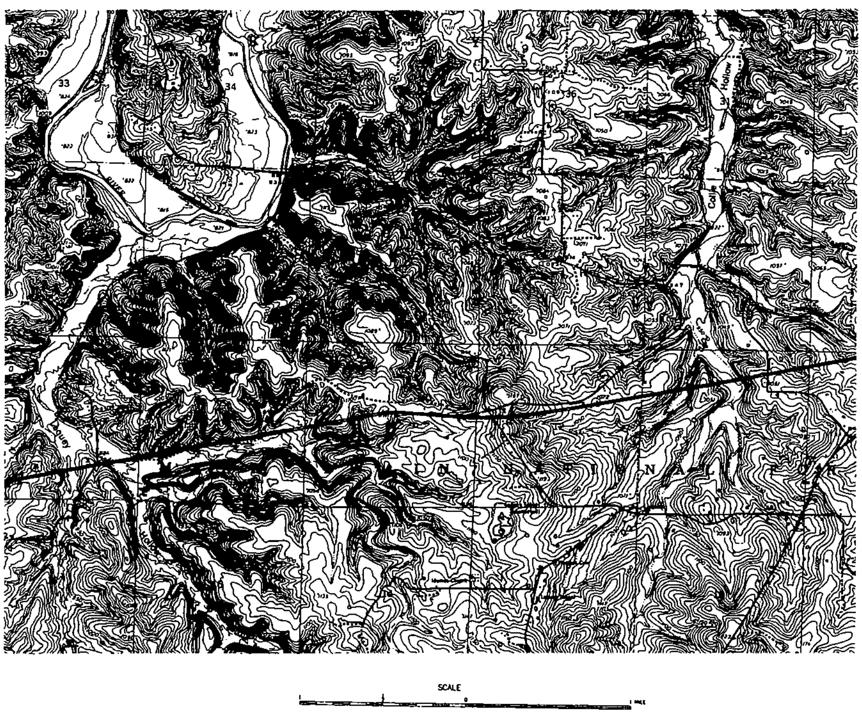

Figure 2. Part of the Ozark Spring 7-1/2 minute topographic map showing the relation of Highway Cave to local topography. Part of a meander loop of the master stream, Gasconade River, is visible in the upper left corner of the map.

Gasconade Dolomite A massively bedded, medium-grained dolomite with some chert. Most of the chert is in reef structures, three to six feet in thickness, which are areally peristent over much of central and southern Missouri, where they form useful stratigraphic makers. The Gasconade is a prominent bluff-former, cropping out along the entrenched stream valleys of the Ozarks. The Gasconade Dolomite probably is the most cavernous formation in Missouri, but intense surface karst is rarely found because of the masking effect of the overlying Roubidoux Sandstone and residuum. The Gasconade also weathers to a thick residuum.

The bedrock structure is essentially flat in Pulaski County. A regional dip varying from one to five degrees toward the northwest, away from the Ozark uplift area of southeastern Missouri, cannot be observed on surface exposures: Local dips of 10 to 30° or more, in the Roubidoux Formation, are generally associated with solution collapse in the Salem Plateau area. Such local dips do not necessarily imply cavernous subsurface conditions because the voids may have been filled by collapse of the overlying material.

The cave passages have developed along enlarged joints in the upper part of the Gasconade Dolomite, but the vertical shafts have breached the transitional Gasconade-Roubidoux contact so that the present dome roofs are in the lower Roubidoux. The lower Roubidoux is not present as a competent rock formation but exists as a poorly graded, residual mixture of angular chert boulders, gravel, and, silt, and red clay.

The roof of the entrance sink was approximately 15 feet above a distinctive, porous Roubidoux chert bed that could be recognized within the cave. When inspected in March 1966 prior to the closing of the sinkhole entrance by State Highway Department personnel, the ceiling of Dome 2 appeared very unstable. A pile of recently fallen boulders six to eight feet high contrasted with a much smaller accumulation of recent roof fall under the other domes. An active flow of water from the roof of Dome 2, probably supplied by the built-in storage capacity of the overlying highway fill, and vibrations from overhead traffic have hastened ceiling collapse.

The unstable nature of the foundation is shown by the large bituminous patches that had to be applied over the concrete pavement on both the eastbound and westbound lanes of U. S. Highway 66 crossing Highway Cave. Subsidence of the concrete pavement was attributed to settlement of the compacted fill over the dry valley, but whether by coincidence or cause, there is an easily discernible sag in the pavement directly over the upstream portion of Highway Cave.

Similar bituminous patches -- the only ones between Waynesville and Springfield -- had to be placed over sections of pavement crossing the dry valley of Collie Hollow; is Collie Hollow underlain by similar cave systems?

DESCRIPTION OF THE PROBLEM

Catastrophic collapse during the latter part of February, 1966 of the bed of a tributary valley to Collie Hollow, SW$\frac{1}{4}$, SW$\frac{1}{4}$, NE$\frac{1}{4}$, NE$\frac{1}{4}$, sec. 7, T. 35N., R. 12 W., at the downstream end of a box culvert beneath U. S. Highway 66, produced a cylindrical cavity 22 feet deep and about 20 feet in diameter (Fig. 3). Collapse apparently began in weakened, porous chert about 15 feet below the valley bottom, and was propagated vertically to the surface. We saw the sinkhole on March 2nd, and conducted our investigation of the sinkhole and cave during that and the following several days. Upon completion of our studies, the hole was filled with local borrow and surfaced with blacktop. Subsequently, the asphalt coating was breached because of settling in the fill. It was refilled and recoated, but the second covering has now been breached and an incipient sinkhole is developing.

From the bottom of the sinkhole, a small opening could be followed downward to the floor of a cave, which extended upstream (SE) beneath the westbound lane of Hwy. 66 (Fig. 5) and downstream beneath the western flanking ridge of the dry valley.

Ordinarily a tube-shaped cave passing at a depth of a few tens of feet beneath compacted highway fill would present no hazards. However, there are four vertical shafts or domes, in the upstream portion of this cave, which in our opinion are geologic hazards to the highway passing overhead. The most hazardous of these is Dome 2 (Fig. 4), which is the largest of the four. One can hear traffic passing overhead while standing in Dome 2. Its ceiling is of porous chert -- the same bed that in failure produced the entry sinkhole -- and some collapse already has occurred.

Dome 1 has undergone partial collapse. It lies partly beneath the box culvert, with water draining into the dome from above. Dome 2 also has undergone some collapse, and has a hazardous ceiling. The ceiling of Dome 2 is being weakened by entry of water from the surface. Domes 3 and 4 are smaller and very little collapse debris has accumulated at their bases.

Downstream, the cave becomes larger and there are more vertical shafts. Dome 5 is littered with large breakdown blocks, while Dome 6 is largely concealed by sink debris accumulated after partial collapse of its roof.

Beyond Dome 6, the passage becomes smaller and drainage passes through a low crawlway, presumably on its way to a resurgence in a spring along the Gasconade River. It was not possible to penetrate beyond this crawlway because of its small size. The stream was dry during the short time the cave was accessible, and no water tracing tests were made. However, after heavy rains much of the flow of the dry valley (a drainage area of about 400 acres) is pirated by the cave system.

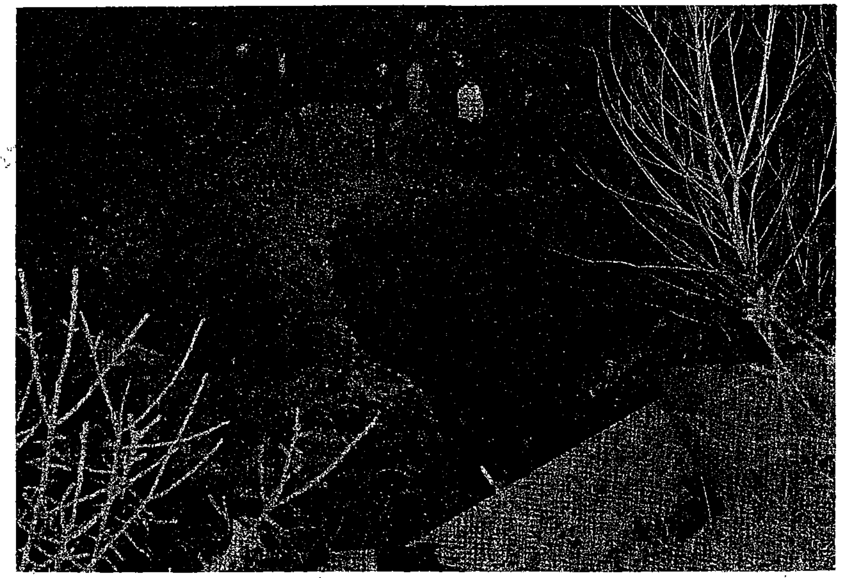

Figure 3. Sinkhole entrance to Highway Cave. The concrete structureiin the lower right corner is the discharge end of a 9' x 9' box culvert passing beneath U. S. Highway 66.

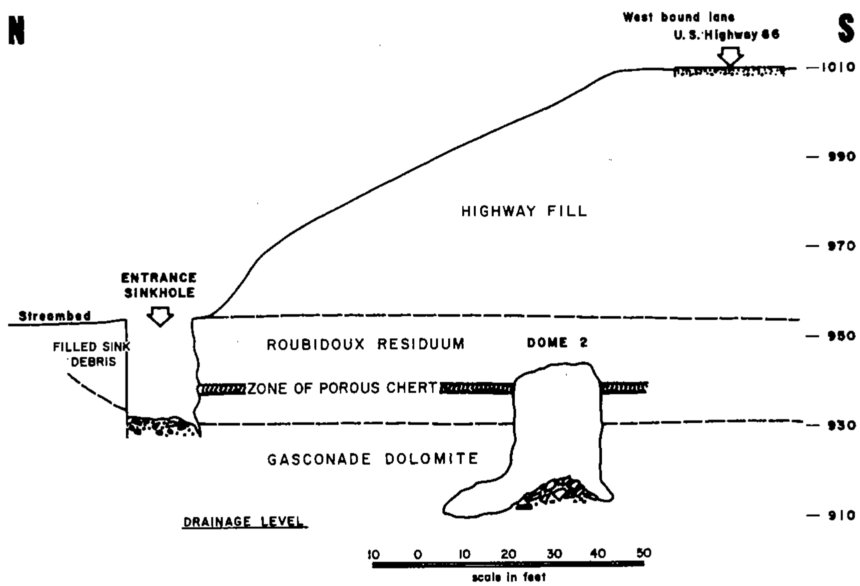

Figure 4. Map of Highway Cave, showing relation of cave to surface drainage, west bound lane of Hwy. 66, and the box culvert.

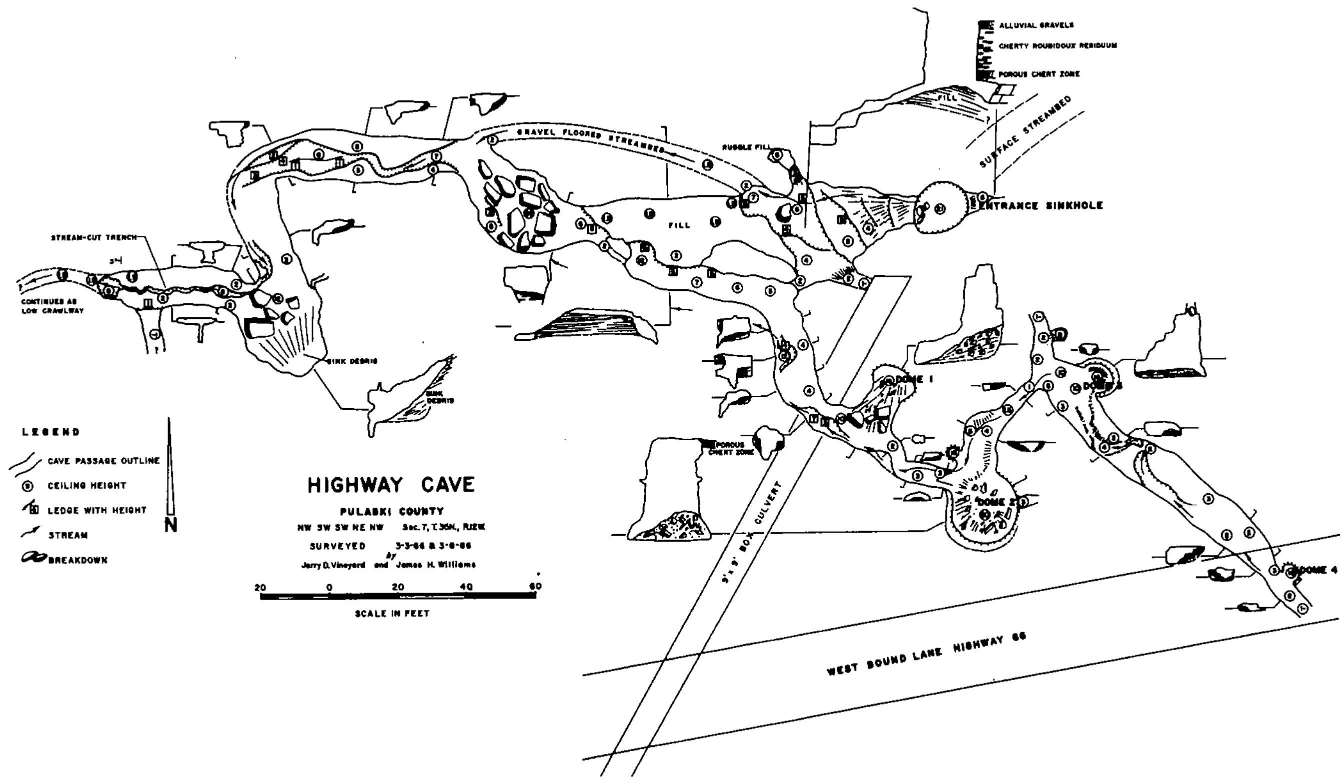

Figure 5. North-south cross section through Highway Cave, showing relation of Dome 2 to the highway fill and local stratigraphy.

The presence of a cave system beheath a dry valley suggests well-integrated subeterranean drainage. Bretz (1956) ascribes a phreatic origin for Ozark caves, and if he is correct, the cave system predates the valley. With valley deepening by surface streams, leakage through joints began the subterranean stream piracy by the cave system. Descending ground water produced the vertical shafts, which grew upward until roof failure in a former shaft produced the sinkhole entrance to the cave. The cave is now protected from erosion only by a section of Roubidoux residuum; further collapse is imminent.

INTERPRETATION

Analysis of the cave map (Fig. 5) shows a joint-controlled cave passage trending roughly WNW-ESE, nearly at right angles to the trend of the surface valley. We interpret the cave as predating the valley. Initial development of the passages probably was sub-water table, a small part of the ground water drainage net existing prior to entrenchment of the master stream, the Gasconade River. During the formative period the cave was water filled and received a complete fill of red sediment, as shown on cross sections in the upstream area of the cave. With valley deepening and dissection of the upland plateau, the water table passed below the cave, draining the formerly water- and sediment-filled passages. Compaction of the fill due to water loss left room for a free-surface stream, which began to remove the fine-sediment fill, and to alter the passage cross sections.

The several vertical shafts are a late development in the history of the cave. Weathering of the bedrock over the cave caused leakage of surface water into the cave system. Major leakage occurred first in what is now the entrance sinkhole. Pirated surface water so weakened the roof rock that collapse occurred, and the cave became analogous to a storm sewer. The first sinkhole collapse allowed the influx of much coarse clastic material, part of which was deposited in the cave between the entrance sink and Dome 5, as shown on the map. The first sinkhole eventually became clogged and remained so for an undetermined period, probably several hundred years. We base this assumption on the unusual secondary mineralization in the downstream area of the cave (Fig. 6). The unusual minerals include aragonite, soft, white opal (cristobalite), and protodolomite. The latter is very rare, known only from a cave in France (Carpenter, 1966, pers. comm.). These minerals are found as encrustations on walls and floors of the cave, as well as on alluvial fill. Curiously, there are no stalactites or stalagmites anywhere in the cave. We attribute the development of unusual mineralization to the restricted air circulation resulting from clogging of both ends of the system by debris.

A record of the earlier sinkhole collapse is preserved in the north wall of the entry sink, where alluvial fill extends downward into Roubidoux residuum.

Figure 6. *Looking downstream in the drainage passage of Highway Cave. White crystalline clusters on the ceiling are rare minerals characteristic of the downstream parts of Highway Cave.*

Figure 7. *View of the dry valley looking downstream from the highway fill over Highway Cave. Note wide, flat flood plain, coarse stream sediments, and abrupt valley sides. Toe of the fill is visible in the lower right corner.*

APPLICATION

Study of the Buckhorn foundation problem and of data from lake site investigations in similar settings has shown that it is possible to anticipate cavernous conditions beneath otherwise normal-appearing stream valleys (Fig. 7) by the recognition of some or all of the following features:

1. Valleys dry or nearly so most of the year.
2. Poorly graded alluvium due to lack of sorting by sustained flow.
3. Wide, flat floodplains bounded by steep valley slopes.
4. Absence of terraces.
5. Abrupt variations in valley profiles, which may suggest subterranean stream piracy.
6. Lack of water-supported vegetation.
7. Abrupt changes in valley alignment, which may indicate intersection of an underlying, structure controlled cave system.
8. Low water tables.
9. Absence of bedrock outcrops and/or pinnacled bedrock.
10. Limited amounts of fine sediment in floodplain deposits.
11. Sinkholes and/or sinking streams.

SUMMARY AND RECOMMENDATIONS

Collapse of the bed of a dry valley in Pulaski County, Missouri, permitted entry into a cave system posing foundation problems to an overlying highway. Exploration of the cave and analysis of the data gathered emphasizes the need for careful foundation investigations in geologically similar localities. It is not customary to conduct detailed subsurface exploration in valleys where highway fills are to be placed, but in limestone terrains there are possible geologic hazards caused by caves under dry valleys. We recommend that surface features as noted in this paper be used to anticipate foundation problems before they are encountered in construction.

Valleys which exhibit several or all of the features found in practice to be characteristic of valleys overlying cavernous areas should be investigated for foundation hazards. A routine drilling program is useful in such cases, but it should be supplemented by geophysical exploration methods. We have found that a 12-channel shallow depth refraction seismograph can delineate subsurface voids. Love (1966) working with the California State Highway Department, has found resistivity equipment to be useful in detecting similar cavities.

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Stabilization of Abandoned Mine Under an Interstate Highway

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ABSTRACT

During the design of Interstate Highway 635 in an urban area in Kansas several large collapses occurred in an underground limestone mine over which the highway was to be constructed. The mine covers an area approximately 1 mile long and from 400 feet to 1/2 mile wide. A geology field party made a detailed investigation of the mine to determine the cause of the collapses and mapped the mine under the highway right of way. Although the mine was still in reasonably good condition beneath the right of way it was felt that the mine was becoming progressively weaker and would suffer collapses at some future time. Estimates were made of several different methods of treating the problem and the decision was reached that the most feasible plan was to use limestone from a nearby cut section to fill the mine under the highway right of way.

The subject of this paper is an abandoned limestone mine in Kansas City, Kansas. The interest of the Kansas Highway Commission in the mine stems from the fact that a portion of the mine lies under a proposed interstate highway route. Preliminary investigations of the mine were initiated in 1963 and at that time the primary concern was an area of the mine where the immediate mine roof was at or near the surface and was suffering the most damage from weathering processes.

In January of 1965 the mine started caving in. There were two collapses in January of that year and there have been four more since then. Contrary to the original thinking, the cave-ins were occurring at the highest elevations over the mine where the overburden was the thickest, from 100 to 120 feet, and where the immediate roof appeared to be in the best condition.

After the second cave-in, the Geology Section was directed by Mr. H. O. Reed, Engineer of Design, to map the mine and to make a complete investigation of it. The problem was to determine why the mine was caving in and to estimate what the life expectancy might be of the part of the mine under the highway right of way. As a final result, some recommendations for treatment of the mine at the time of the highway construction were requested. A four man geology party made up of Larry Rockers, Bill Jones, Larry Knoche, and myself was designated by Mr. Virgil Burgat, Chief Geologist, to make the investigation.

The following pages will give a general description of the mine, the results of the investigation, and the final recommendations that were made.

The cave-ins are circular and bowl shaped with a tension crack around the outside and they vary from 200 to 350 feet across. In one case where a cave-in occurred in the lower part of a drainage area, heavy rains caused a slide and mudflow along the shear zone and most of a house sitting on the shear zone slid down into the mine about 90 feet below.

It was recognized that as far as construction costs were concerned it would be more economical to let the highway cave in with the mine at some future date and repair the damage from the top, however, this approach would introduce an extremely dangerous situation to the motoring public and with the tremendous amount of traffic and a lack of any detour route there would be mass confusion in the traffic pattern and a considerable inconvenience to the public. In addition, repairing a mine cave-in from the top creates a long period of maintenance due to continued settlement of the cave-in material, so our studies of the mine were pointed mainly toward preventing failure of the mine.

The entrance to the mine is in the lower right hand corner of the map, Figure 1, and the distance between the principal streets is 1/2 mile. From the entrance to the far southwest end of the mine is about 4600 feet and the mine varies from approximately 300 to 1000 feet in width. It is abandoned and nearly all of the mine contains water varying up to 8 or 10 feet in depth. The cave-ins are indicated by the circles on the map. Centerline extends from south to north across the two lobes at the west end of the mine.

The mine is in the Argentine-Frisbie section of the Wyandotte Limestone. It was worked by the room and pillar method with the pillars left on a diagonal system with a distance of 35 to 45 feet face to face. In the north highway crossing, the pillars vary from 16 to 30 feet in diameter which is about average for most of the mine. In the south crossing, the diameter ranges up to 50 feet. The immediate limestone roof is 11 feet thick with the opening varying from 14 to 19 feet depending on the amount of limestone taken out of the floor. Where there is 19 feet of opening all of the limestone floor is gone and the pillars are sitting directly on shale.

The basal portion of the Argentine-Frisbie Limestone is thin bedded with several shale layers. The lower 4 feet represent the maximum floor thickness. In some sections only the basal one foot Frisbie Limestone was left as a floor and in other sections the un-

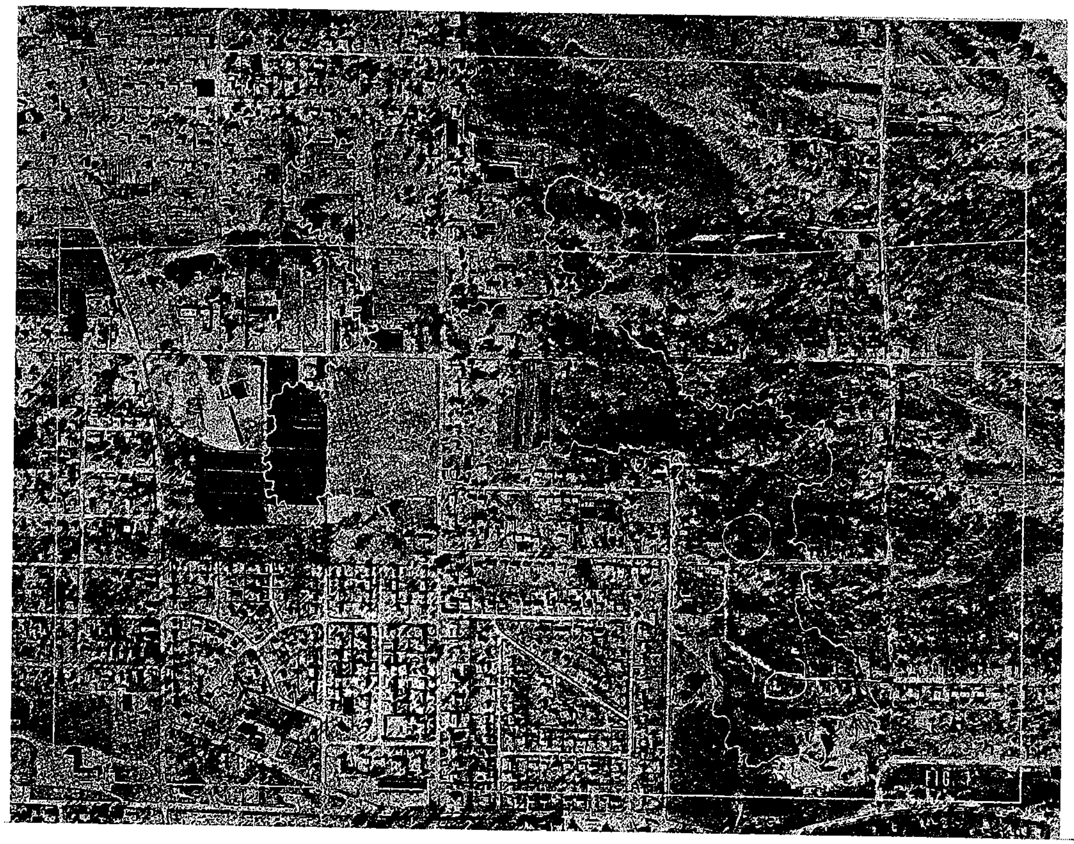

Figure 1. Aerial photograph of mine area along proposed interstate

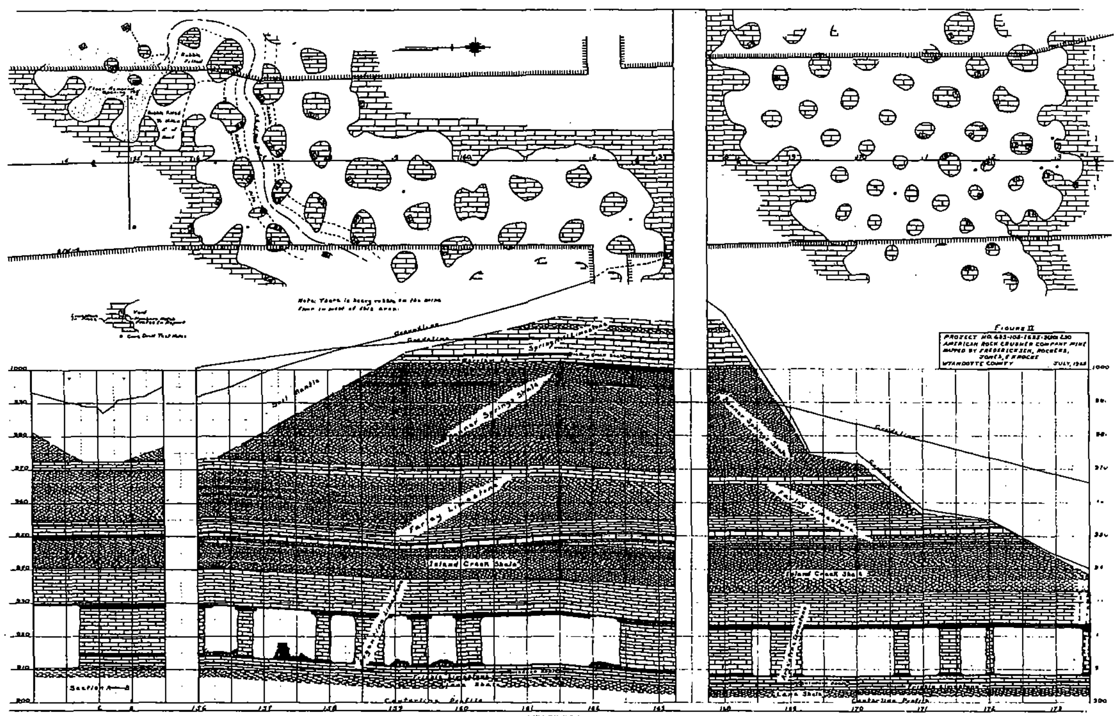

Figure 2. Plan view and cross section of limestone mine

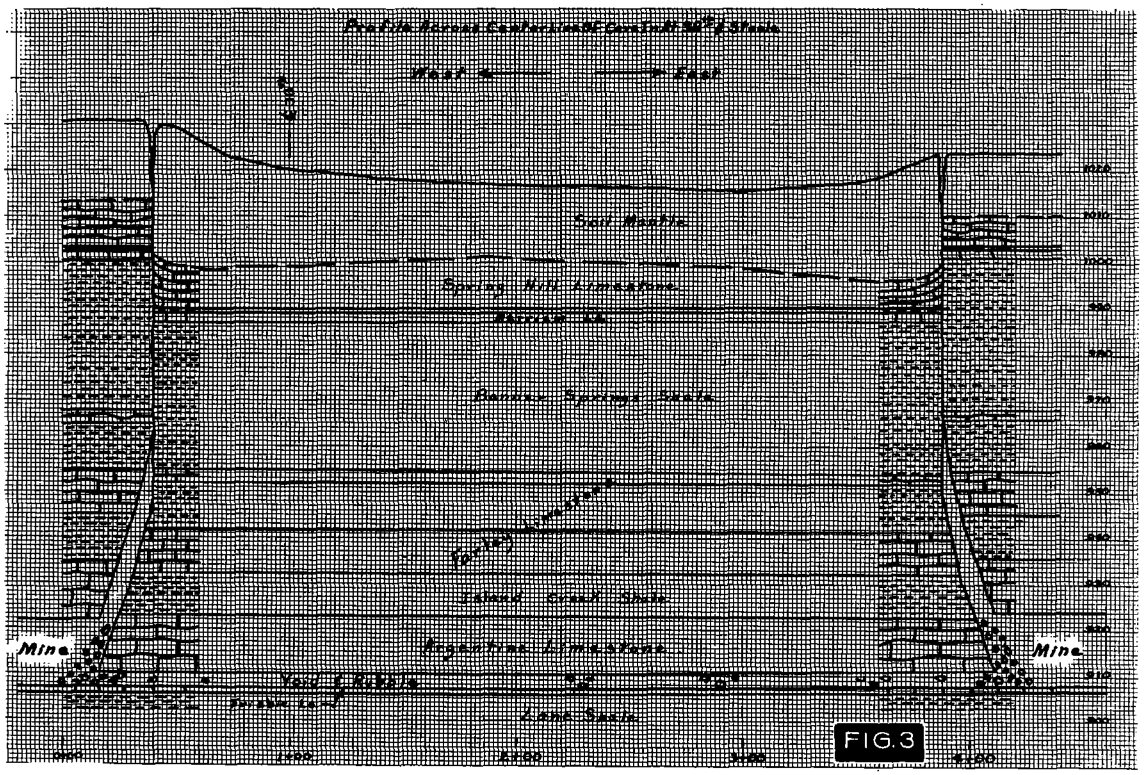

Figure 3. Profile across cave-in at 38th and Steele Streets

derlying Lane Shale is the floor of the mine. At the mine entrance, the limestone is a thin, irregularly bedded limestone with very thin shale partings. The 2 foot layer above the opening that makes up the mine ceiling is a thinly bedded, shaly limestone that weathers to shale in outcrop exposures. This zone has no commercial value and is also poor roof rock as indicated in Figure 10.

We could walk and wade back about 1200 feet into the mine and from there on had to go by boat. In the area shown in Figure 7 several of the pillars were leaning slightly and down through the clear water we could see the floor of the mine bulging up and large cracks opening up in the floor. The water was deepest next to the pillars and from all appearances the pillars were settling into the underlying shale. On the last morning that we entered the mine, the floor had split open in the foreground of this picture. Off to the right we could hear rocks rolling down the side of a pillar and into the water. Our lights were too weak to see the pillar but it sounded as if the pillar was tipping over and the top was shearing off. This was the only route in and out of the mine so we completed our investigation on that particular day. Actually it was about a month before the mine finally collapsed here.

Figure 8 is looking along side of one of the collapses. This is the old haul road and the pillars are sitting on 4 feet of limestone floor, however, beneath the cave-in, there is only 1 foot of limestone and water is standing over the floor. The near pillar is leaning slightly due to the weight behind it.

The geologic profile across the collapse, Figure 3, shows that the beds are still in their proper positions but 13 feet lower. There was a fairly recent map available of part of this area and it showed approximately 30 pillars beneath the cave-in area. 90% of the limestone was removed and 10% left in the pillars. If the pillars were still standing they would protrude above the Argentine Ledge and should be reflected at the surface as humps in the ground, however, there is no evidence of the pillars at the ground surface. An attempt was made to find one of the larger pillars by drilling. The roof was down at the same elevation as the surrounding roof in the cave-in and cores showed the roof to be in apparently good condition but we found only rubble beneath the roof where the pillar should have been. It is thought that most of the pillars beneath the cave-in have collapsed, not necessarily because the pillars themselves were weak but because the foundation material beneath the pillars failed. Cores taken of the underlying shale in the cave-in area were sent to the lab for unconfined compression tests and a typical series of tests show 0.2 tsf directly beneath the floor, 0.9 tsf a little over a foot below the floor, 13 tsf 2-1/2 feet below the floor and the tests increased up to 90 tsf 13 feet below the floor. The loading on the pillars in this area was 79 tsf.

As we progress towards the newer part of the mine and under the highway right of way the unconfined compression strength of the shale directly beneath the floor increases up to anywhere from 35 to

60 tsf. By comparing the strength of the shale and the time elapsed since the area was opened, between the cave-in area and the area under the highway project, we can come up with a figure of loss of strength of the shale at the rate of 1.5 tsf per year and in applying this to an average location under the project we can figure that the strength of the shale will be down to near zero within about 30 years and the mine will be collapsing. Admittedly this is only a wild estimate but it does give some basis for thought and actually the age of the mine where the collapses are now occurring varies from a little over 15 years up to 23 years so this figure of 30 years is probably not too far off since the newer section of the mine is in a little better shape structurally than the older part.

The north section of the mine under the highway project is in good condition. The amount of removal is 86% with 14% left in the pillars. This is about average for mines in the Kansas City area. The pillars are irregular in size and some have had blast damage. I think this weakness determines the center of the cave-ins. As a group of pillars start settling the weakest one fails and throws an extra load on the next concentric circle of pillars. Those in turn fail and the load is transferred to the next circle until the roof reaches its maximum spanning ability and shears off in front of either the second or third circle of pillars. One of the things that we noted in the older part of the mine was that the roof was in good condition out in the central part but in the rooms next to the wall the roof was nearly always spalling off. Apparently this is due to the pillars and roof settling more or less uniformly, however, at the wall the roof is clamped and can't give so it starts shearing at the wall first.

Going from the north location to the south location we rather abruptly encountered an area where nearly all of the weak layer of limestone in the immediate roof had spalled off. This occurs over all of the south highway location. The only reason we could see for the roof failures is that back to the north the limestone joints are tightly closed while down in the south section some of the joints are pulled apart. Our surface geology shows a fault just south of the mine and it may be that the structural activity at the time of the fault caused the joints to separate here. The pillars are larger in the south area with 21% of the limestone left in the pillars and 79% removed. There is a small area of the south location where the floor is completely gone and the pillars are sitting on shale (Figure 11). Some evidence of deterioration is showing up at the present time and when water reaches this shale, there may be failure of the pillars within a short time, possibly 4 or 5 years. The remainder of the mine under the two crossings is in good condition at the present time with 4 feet of limestone floor and the south crossing is relatively dry. However, it is felt that the mine is slowly deteriorating and, as mentioned previously, will fail at some future date.

After studying several methods of treating the mine, the decision was made to take limestone out of the cut above the mine, along with some additional borrow, crush it to a maximum 6" size and place this material in the mine as fill. Also, the roof fall material in the mine will be used as bulkheads. The material will be placed up to within 6" of the roof except under Gibbs Road where we have a bridge located half on and half off of the mine and here the fill will be placed tightly against the roof. The thinking behind this is that we can place the rock within 6" of the roof fairly easily and economically and eventually as the pillars settle into the underlying shale the floor will bulge up due to the shale swelling as it weathers and due to migration of shale from beneath the pillars and as a final result the roof and floor will clamp down onto the fill material with very little if any settlement of the roadbed at the surface.

It was thought that the bidders on the project should have an opportunity to inspect the mine, so last summer we built an entrance into the north area by trenching and tunneling in from a low area on the right of way (Figure 12).

The section of the mine that we are going to work in is still in good condition and actually is probably much safer to work in now than when the mine was in operation since nearly all of the loose slabs have fallen from the roof. The temperature is around 60 degrees. This is probably the nearest to an air conditioned project that we'll ever have and it should be a very interesting project.

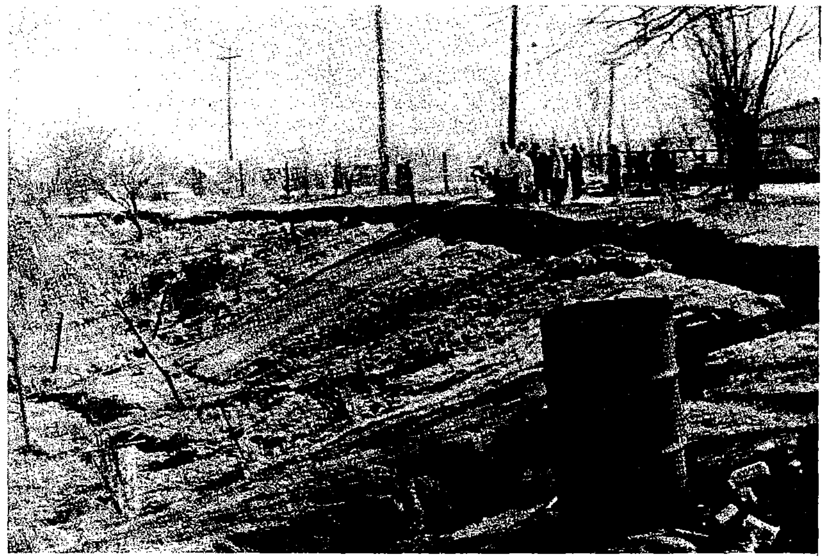

Figure 4. Tension cracks around outside of cave-in

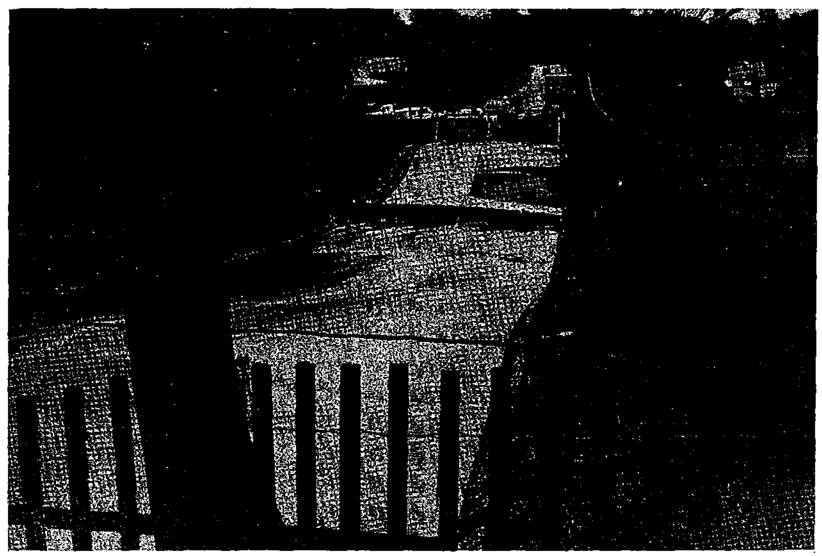

Figure 5. Cave-in on heavily traveled street

Figure 6. Entrance to mine

Figure 7. Area which caved shortly after investigation

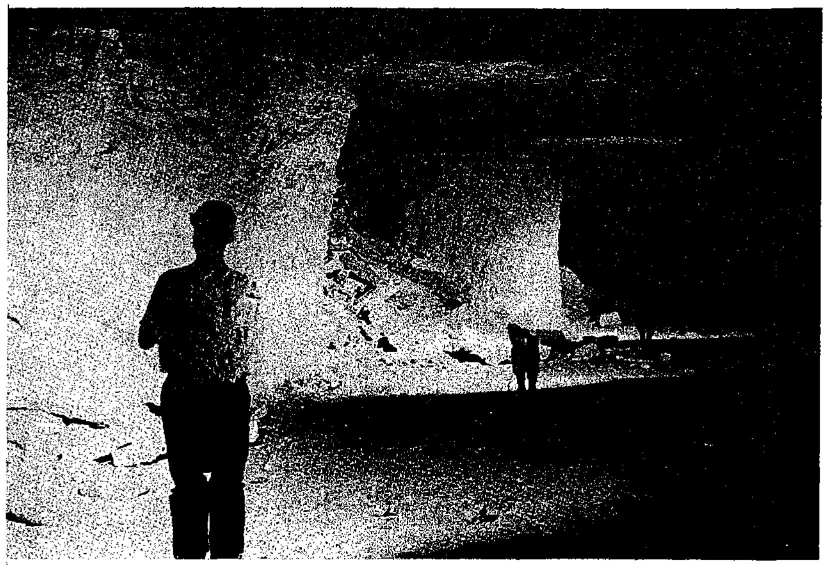

Figure 8. View along side of cave-in area

Figure 9. Mine under north highway crossing

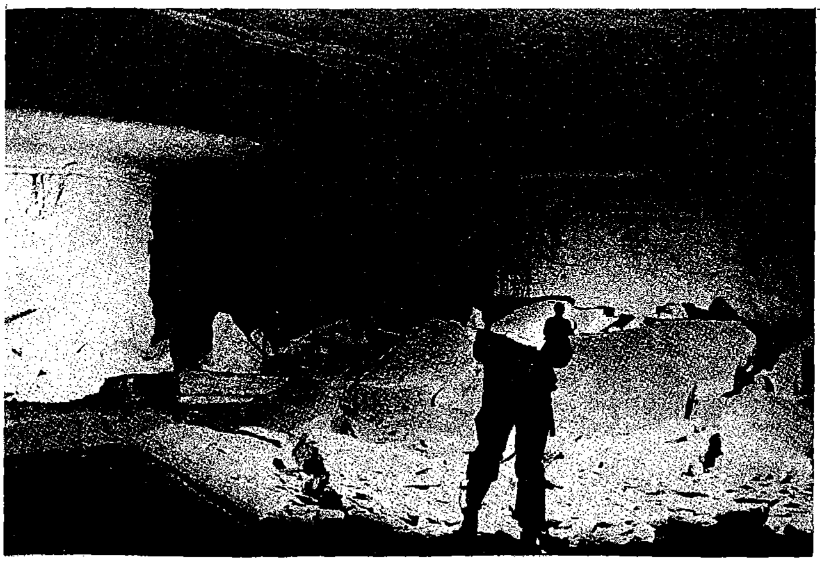

Figure 10. Mine under south highway crossing

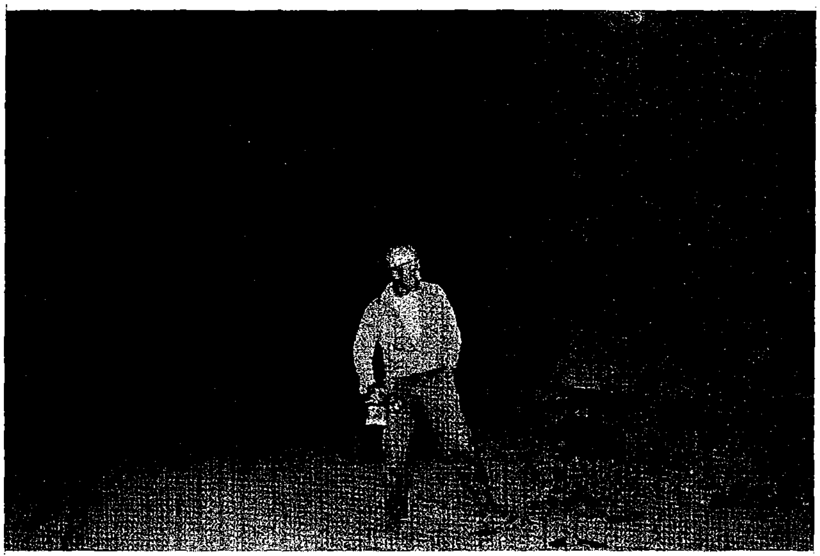

Figure 11. Pillars founded on shale

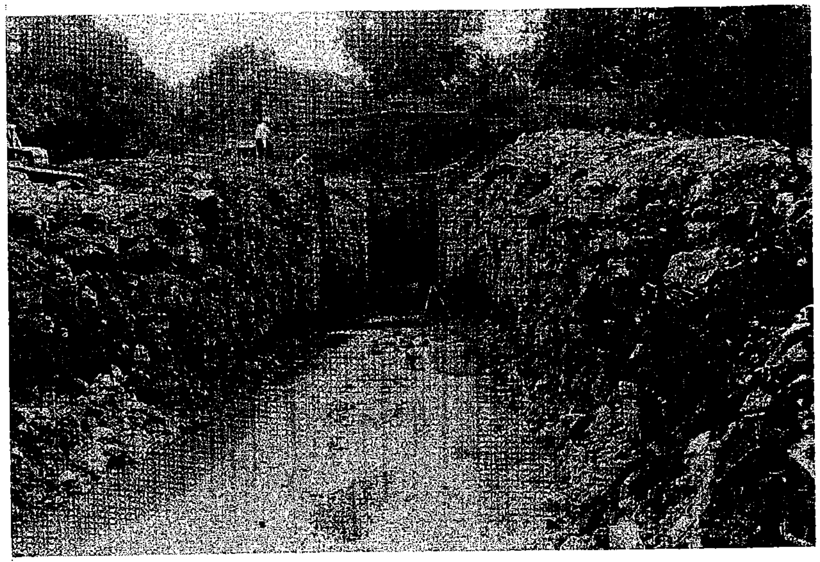

Figure 12. Entrance to mine constructed on highway right of way

Soil Survey Practices in Indiana

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ABSTRACT

Indiana has a variety of geological formations which influence highway design and construction. The major portion of the surface soils are those developed from Illinoian and Wisconsin glacial deposits and the residual soils of non-glaciated areas. Lesser areas of other formations are encountered. Bedrock is also encountered, primarily in the non-glaciated areas. Each of these areas present certain problems which require investigation.

This paper discusses the major soils problems which are encountered in highway design and construction in Indiana. Soil surveys are directed at determining the problem areas, determining the soils conditions, and making recommendations as to how these problems may be overcome. Procedure followed in making soil surveys are explained, including preliminary planning; location, frequency, and depth of borings; types of boring and sampling; laboratory testing; analysis; and the Soils Report. Illustrated case studies are presented of a side-hill cut and fill in an Ordovician formation, a tri-level separation structure in which saturated silts were encountered, a highway cut in loess, and a 170-foot rock cut.

How many times have we, as Geologists, Engineers, or Engineering Geologists, pondered our problems connected with determining what materials Mother Nature has provided and how they best could be used in highway construction? The question often arises, "I wonder what other people do?" This Highway Geology Symposium seemed an opportune time to provide a partial answer to this question by discussing Indiana's practices in soil investigations and then show you a few examples of typical problems which are encountered in Indiana.

As many of you are aware, Indiana has a variety of geological formations which influence highway design and construction. Surface soils developed from Illinoian or Wisconsin glacial deposits, and the residual soils of non-glaciated areas predominate. Lesser areas of wind-blown deposits of loess, wind-blown sands, and lacustrine deposits are encountered. Bedrock is encountered in, but not restricted to, the non-glaciated areas. All of these areas present their own individual problems which must be investigated.

Some of the major soil problems which must be considered are:

1. Deposits of peat, marl, muck, or other unstable soils.

2. Saturated, fine grained soils and water-bearing strata near grade in cuts. These present subgrade problems, also back-slope problems.

3. Settlement and embankment foundation stability in lacustrine deposits and at stream crossings.

4. Stability problems arising from cuts, or side-hill cut and fill in certain of the unglaciated areas.

5. Erosion of granular soils, silty soils, and some residual soils.

There are, of course, other problems which occur, but not on the scale or frequency of the aforementioned.

Modern highway design standards, with their flatter grades and straighter alignments, require much deeper cuts and higher embankments than were required 25 years ago. These requirements magnify some of the problems. For example, on a recent project the subsoils in an embankment area appeared reasonably stiff and would not have been considered to be a problem for a 20 to 25-foot fill. However, the proposed construction was an ascending grade on an 80 to 90-foot fill, going directly into a 100-plus foot rock cut. Testing and analyses showed that the foundation would be unstable for this height of fill. The grade could not be altered. Cost analysis showed that a structure was more economical than embankment stabilizing measures.

Indiana's efforts in roadway soil investigations are directed toward determining the problem areas and conditions, then making recommendations as to how the problems may be resolved.

Present policy requires a complete roadway soil survey for all Interstate and Primary Federal Aid, as well as some Secondary Federal Aid projects. The Soils Department of the Division of Materials and Tests, Indiana State Highway Commission, is limited in its capabilities at the present time because of inadequate facilities and shortage of personnel. Therefore, the majority of this work has been done as a part of the Design Engineering Agreements by Soils Consultants, under supervision of the Soils Department. However, a new laboratory building is now under construction and it is anticipated that, with future expansion, the Soils Department will do most of this work.

Prior to beginning a soil survey for a particular project, County Engineering Soil maps, Pedalogical maps, Geological maps, and other available data are reviewed. These, along with general experience and knowledge of the area, are of great assistance in planning the investigation. Actual boring locations are established by field check. In general, at least one boring is made in each cut and each fill. In extremely long cuts or fills, additional borings are made at intervals of 500 to 800 feet or more, depending on the terrain and conditions. Borings extend seven feet below grade in cuts and six feet, or two-thirds the fill height, whichever is greater, in fills. Either hand or machine-auger borings are permitted. In the case of machine borings, continuous flight auger is not permitted. A two-foot length auger bit is used and the maximum increment of advance is two feet.

In areas of suspected weak, or soft foundation conditions, machine borings with split spoon sampling are made to determine penetration resistance as an aid in evaluating conditions.

When rock is encountered, a minimum of two core borings at a representative cross-section is made in each cut to determine the quality and stratification of rock. Soundings to rock are made on a grid pattern at offsets of 60 feet at each station.

In areas of peat, marl, or other unstable soil, soundings are made on a grid pattern to determine the depth and areal limits of material to be removed. These are generally at 50-foot offset intervals to the construction limits at each station.

During the borings, soil samples are obtained for laboratory classification testing. Generally, one sample per soil type encountered per mile is obtained. A larger sample of each major soil type likely to occur at subgrade is obtained for laboratory determination of California Bearing Ratio. When soft or questionable soils are encountered in fill areas, undisturbed samples may be required for consolidation and strength testing. These are two-inch and three-inch diameter samples.

Laboratory testing consists of grain size analysis, Atterberg limits, California Bearing Ratio, natural moisture content, loss on ignition, consolidation, and shear tests. Settlement and stability analyses are made when conditions warrant.

On the basis of the borings, field observations, laboratory testing, and any analyses which are made, a Soils Report is prepared. General soil conditions are described and general recommendations as to design and construction are made. Special problems or conditions are explained along with recommendations relative to special embankment construction; cut slopes in soil or rock; subgrade removal, replacement or treatment; removal of unsuitable soil; rock swell factors; drainage installations; the use of channel change materials; or any other factors affecting design or construction of the project. Included as a part of the Soils Report are all test data, analyses, boring logs, and the plotted Soils Profile.

After initial review by the Soils Department, the Soils Report is reviewed at a conference by the Design Division, Design Consultant, Bureau of Public Roads, and Soils Department. Here, decisions as to design and construction are made on the basis of the information and recommendations in the Soils Report.

Now, let us look at a few conditions which occur in Indiana and the problems they present.

Figure 1 is a view of Interstate 74 near West Harrison in eastern Indiana, looking east. This section of I-74 goes through the northern portion of the Ordovician units. The Ordovician units consist of limestone and shale and are considered one of the most treacherous materials in the State with respect to slides. One engineer described it by saying that whenever a teaspoonful of dirt is moved, a slide occurs.

In this area the proposed location of I-74 was near an existing highway, SR 46, which goes through the bottom of Logan Creek valley. This presented a difficult location problem. There were three choices available: (1) relocate SR 46 and locate I-74 through the bottom of the valley; (2) leave SR 46 in its present location and locate I-74 on the side of the valley; or (3) use a different location for I-74 which would avoid side-hill construction. Although it was recognized as a poor choice, one which could lead to serious problems, the location mentioned in (2) was selected.

Figure 2 is a view looking over the left edge of embankment of the eastbound lane. SR 46 can be seen to the left of and below the westbound lane. SR 46 at this location is on a fill of two to four feet. The pavement elevation is 551. The natural ground sloped upward to the right from SR 46 at an approximate 6 to 1 slope for 225 feet, then approximately 4 to 1 for 200 feet, then approximately 2-1/2 to 1. As finally designed, centerline of the westbound lane is 268 feet right of SR 46 at elevation 613, or 62 feet above SR 46, on 28 feet of fill. Centerline of the eastbound land is 186 feet right of the westbound lane at elevation 661, or 48 feet above the westbound lane, on 38 feet of fill. Due to the slope of ground, there is a slight cut on the right of the eastbound lane which daylights at elevation 684, 160 feet right of the eastbound lane. Fill slopes and cut slopes were 2 to 1.

In case you have not already guessed it, this choice of location proved to be costly. Slides developed during construction, first in the embankment for the westbound lane. Further sliding developed into the embankment for the eastbound lane. Correction of the slides was costly and time-consuming. The displaced soil had to be removed, side-hill benches excavated to shale, drainage installed, and the removed soil placed again in compacted lifts. In addition, a stabilizing berm was placed on the left of the westbound lane. The top of this berm was at elevation 596 and was 40 feet wide; side slopes were 2 to 1.

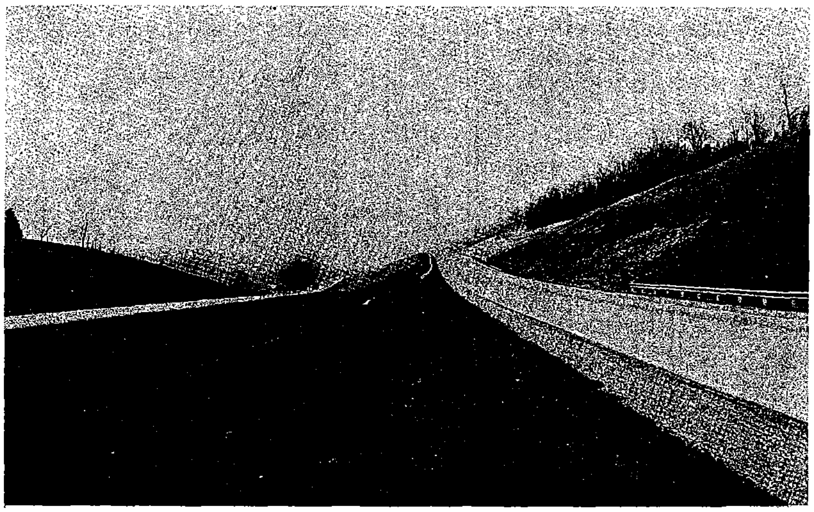

Figure 1. View looking east on I-74 in eastern Indiana through a deep Ordovican rock cut.

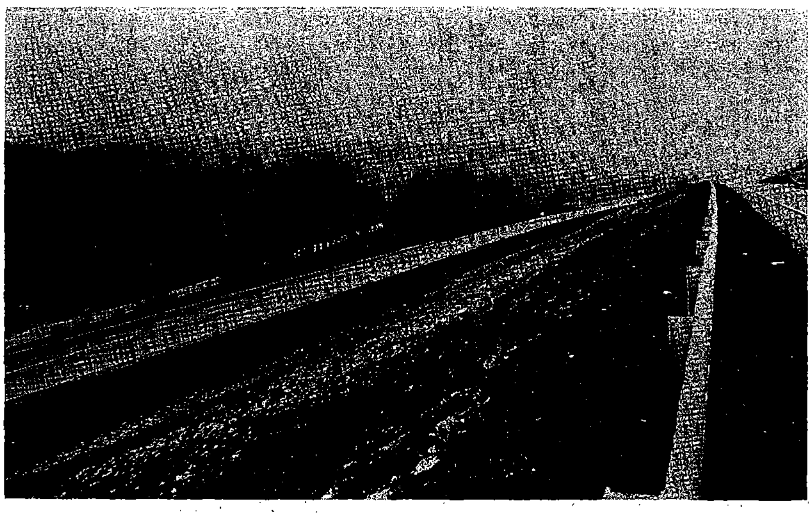

Figure 2. View of slide area on I-74 in Ordovican rock. West bound land to left is 48 feet below eastbound lane. SR 46, at extreme left, is 62 feet below the west bound land.

Figure 3 is a view of the westbound lane, looking west. All of the grading and the slide correction were done on a grading contract. The paving was done on a later contract. This view shows the embankment slope, which was reconstructed at a 3 to 1 slope rather than 2 to 1, and the top of the stabilizing berm.

Figure 4 is also looking west. This shows the embankment slope of the westbound lane and the top and side slope of the stabilizing berm. Just out of view, on the right is SR 46.

Figure 5 shows the outlets for some of the drains which were installed. The guard rail is along SR 46 near the toe of the stabilizing berm.

Figure 6 is a view of a cut on the same section of I-74, but east of the slide area. The view is looking west and shows a frontage road to the south of I-74. The east and westbound lanes are out of view to the right. At this particular area the westbound lane is on a five-foot fill; the eastbound lane is in 15 foot of cut; the frontage road which is shown is 82 feet right of the eastbound lane and is in 35 feet of cut; and the top of the cut slope is 602 feet right of the eastbound lane at elevation 770, 205 feet above the east- and westbound pavements. There are 8 benches, each 20 feet high and 12 feet wide, on 2 to 1 slopes. The last slope to the top of cut is 40 feet high, also on a 2 to 1 slope. This cut daylights approximately 50 feet beyond the top of the hill where the ground is nearly level. No stability developed in this cut.

Figure 7 is a view of a tri-level structure on US 41 and US 50 at Vincennes. The top level is eastbound US 50, going to the left. The middle level is northbound US 41 going to the right. The bottom level is a connector from westbound US 50 and carries southbound US 41 to the left. The bottom level is in cut of approximately 15 to 18 feet. The middle level is in fill of approximately 3 feet and the top level is in fill of approximately 28 feet on the right and 18 feet on the left. Thus, the top level is on an equivalent fill of 43 feet on the right and 36 feet on the left.

Borings were made for the structure foundations. However, this project was in the design stage prior to adoption of present procedures and there was no roadway soil survey made. During construction while the cut for the bottom level was being made, a saturated silty loam soil was encountered at approximately 6 to 8 feet below ground. This soil is locally referred to as "bull's liver," a term very descriptive of its appearance and condition. Of course, this type of soil has little stability. The immediate problem was to determine what measures should be taken to insure stability of the subgrade for the bottom pavement. The most desirable method was to undercut the material three feet below subgrade and backfill with granular material, which was done. However, it appeared that there could be a more serious problem with respect to the spillthrough portion of the top level structure. Pier construction was under way at the time, and the approach embankment constructed to within 5 to

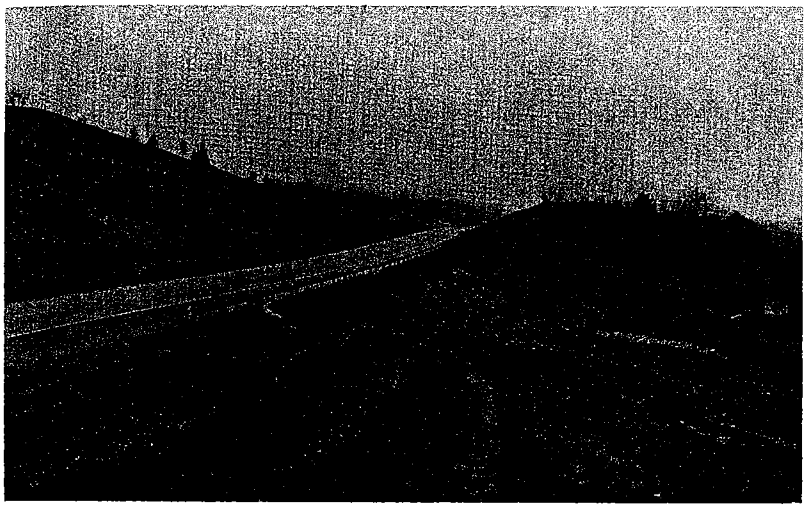

Figure 3. I-74 slide area looking west with east bound lane at upper left. Stabalizing berm for the west bound lane can be seen on right side of photo.

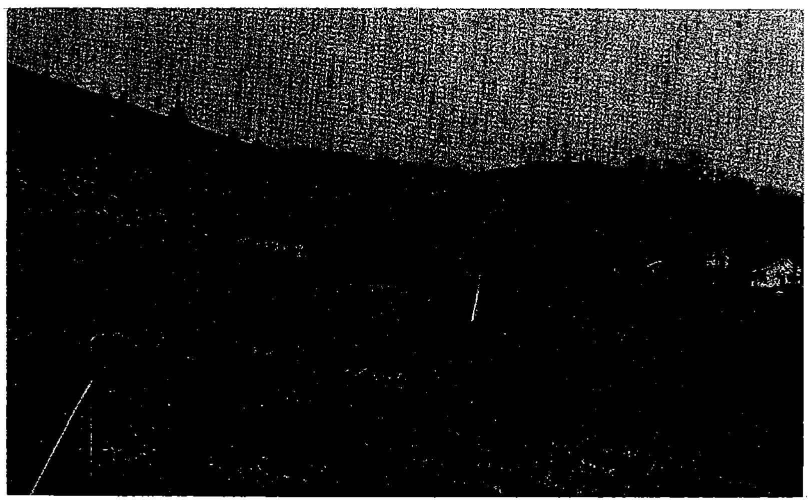

Figure 4. I-74 slide area looking west at overall view of top and side slope of the stabalizing berm.

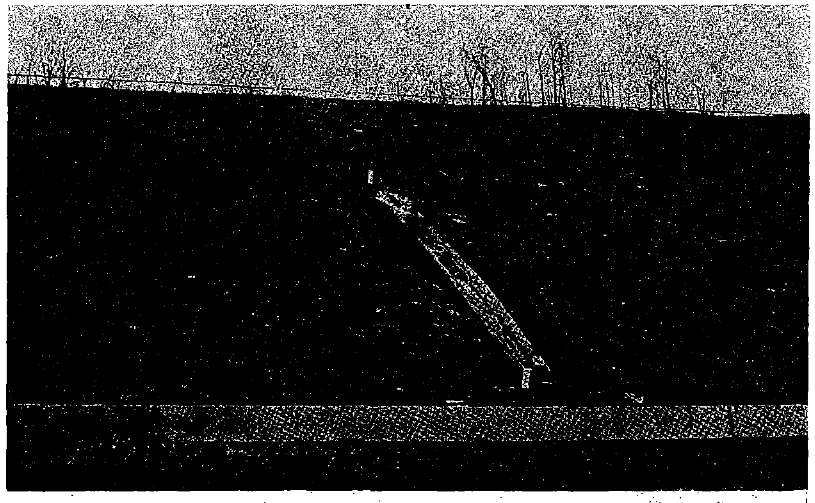

Figure 5. I-74 slide area with view at outlet for drains installed as part of slide correction. Paved ditch is on the slope of the stabalizing berm.

Figure 6. Another deep, side hill cut on I-74. View is looking west along frontage road with the east bound and west bound lanes out of view to the right. There are eight benches in the cut with the top 205 feet above the I-74 pavements and 602 feet right of the east bound lane.

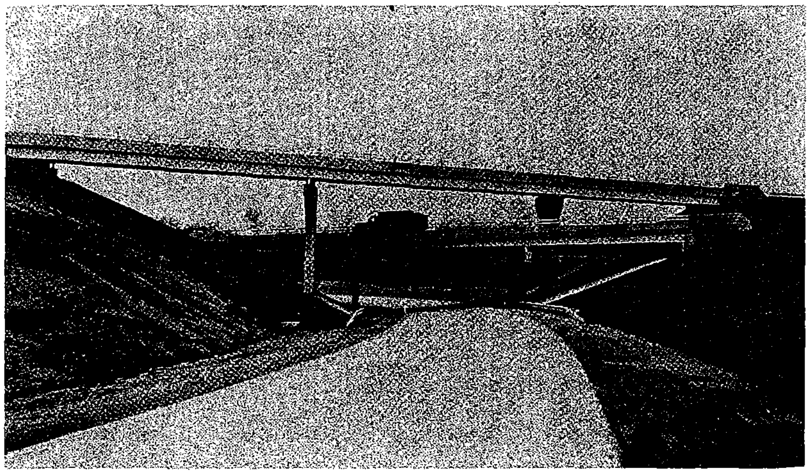

Figure 7. A tri-level structure on US 41 and US 50 at Vincennes. Bottom level is a 15 to 18 foot cut in which saturated silty loam or "bull's liver" was encountered. The top level is on a 43 foot fill.

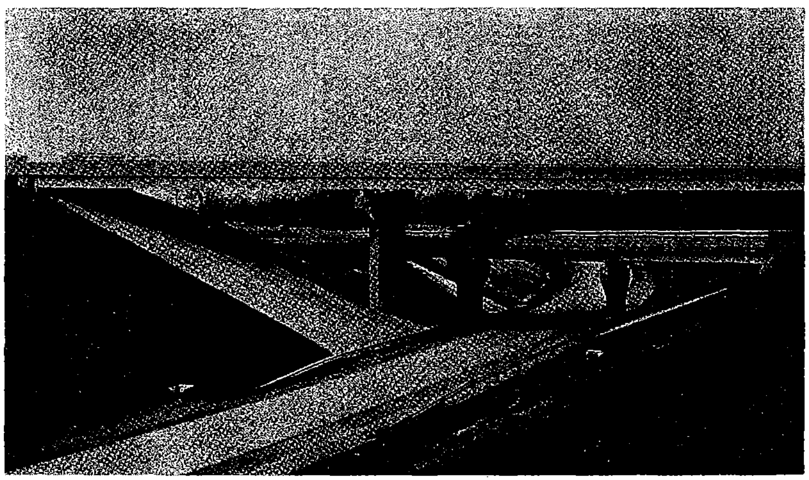

Figure 8. Another view of tri-level structure on US 41 and US 50. The shallow fill for the middle level is indicated in this view.

10 feet of finished elevation. Undisturbed samples of the saturated soils were obtained and strength tests performed. Analyses confirmed that factors of safety against shear failures at the end bents were less than unity. Several methods of correcting this condition were suggested. Due to the stage of the construction, it was decided that the quickest and most economical method of treatment would be to drive sheet piling into stable material behind the first pier on each end. This method of treatment was used.

Figure 8 is another view of the tri-level structure. The low fill at the end of the middle level structure is more apparent in this view.

Figure 9 is a view of a cut in loess above sandstone and shale. The location is the northbound lane of US 41 south of Patoka, looking south. As stated previously, loess is a wind-deposited soil and has a vertical structure. It is not identified by test data but must be recognized in the field by observation. The outstanding feature of this type of formation is that it will stand on nearly vertical slopes. When cut on flatter slopes, it tends to revert to a near vertical slope; therefore, it is normal practice to cut loess on a near-vertical slope. Saturation of the edge of the slope at the top will cause sloughing of the slope. Hence, drainage of surface water away from the edge of the slope is necessary. Such a slough can be seen in the picture, on the top face. Figure 10, taken from the bench, shows this slough more clearly, as well as an accumulation of other material which has sloughed.

Figure 11 is a view looking east of I-64 near New Albany. This is a cut of approximately 170 feet in the lower Mississippian System. The rock is shaly siltstone of the Osage or Borden Group. Core borings in the cut encountered a 2- to 3-foot thick layer of limestone, probably Floyd Knobs Limestone, at a depth of approximately 40 feet.

The excavation is down to approximate subgrade elevation but the pavement has not been placed. The left pavement in this area will be at elevation 739 and the right pavement at elevation 764. Presplitting was used in the rock excavation. The benches are 30 feet high, on a 1 to 6 batter, and 12 feet wide.

Figure 12 is from the same vantage point, but looking west. The limestone layer mentioned previously is clearly visible. Also note the rock barrier between the west and eastbound lanes.

Figure 13 shows an interesting situation encountered on SR 64, east of Marengo. The view is looking west. The highway is in cut through an old, abandoned quarry in which Ste. Genevieve limestone was mined. Figure 14 is a view from the same location, looking north.

These have been only a few typical examples of soils and conditions encountered in Indiana. And, although time doesn't permit showing you more, I can assure you that there are enough additional and varied problems to keep one interested all the time.

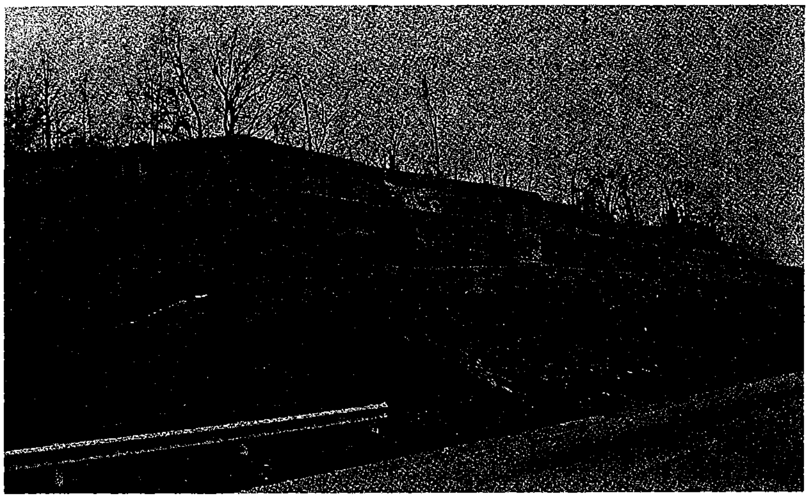

Figure 9. Loess cut above sandstone and shale on north bound lane of US 41 south of Patoka.

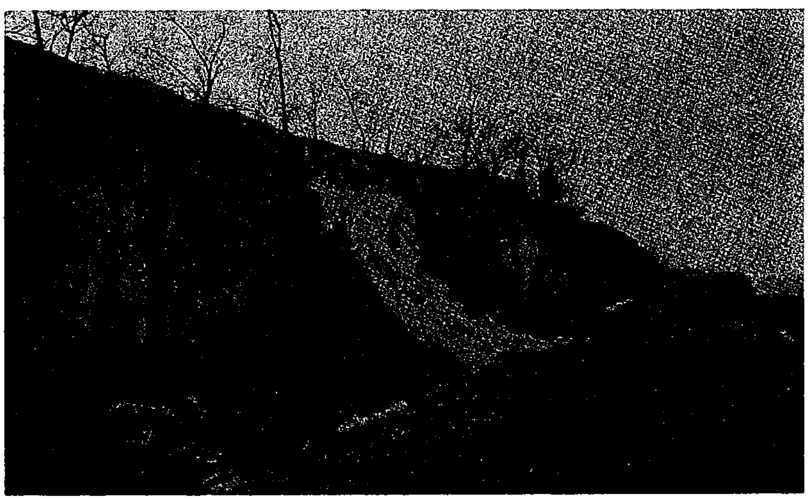

Figure 10. Sloughing of vertical edge of top bench of loess cut on US 41.

Figure 11. I-64 near New Albany looking east at a 170 foot cut in lower Mississippian strata in which shaly siltstone predominates. Pre-splitting was used during excavation; benches are 30 feet high, 12 feet wide, with a 1 to 6 batter.

Figure 12. Same cut on I-64 looking west. A prominent 2 to 3 foot thick limestone layer, approximately 40 feet from the top, is visible.

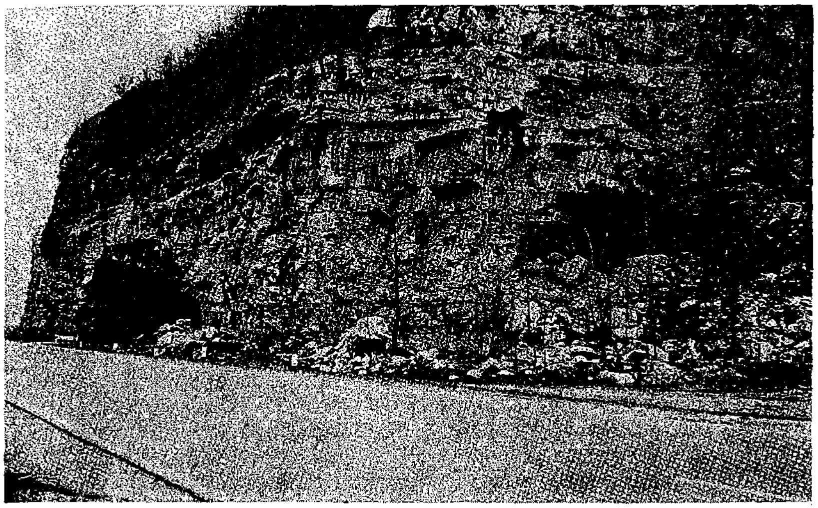

Figure 13. SR 64 east of Marengo looking west. This cut is through an abandoned quarry in the Ste. Genevieve limestone.

Figure 14. Looking north from SR 64 into abandoned mine rooms in Ste. Genevieve limestone.

Geologic Factors Affecting the Exploration for Mineral Aggregates in the Indianapolis Area*

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Indiana Geological Survey

ABSTRACT

Marion County is one of the largest markets for mineral aggregates in Indiana. Sand and gravel are presently supplied by eight plants that exploit glacial deposits in the valleys of the White River, Eagle Creek, and Fall Creek. Crushed stone is being shipped into the metropolitan area by road and rail from 11 quarries in Owen, Putnam, Hamilton, Madison, Shelby, and Bartholmew Counties.

Examination of geologic factors, such as variations in the composition and distribution of unconsolidated materials, thickness of overburden, quality and thickness of exploitable rock units, and potential water problems, indicates that other sites may be available for mineral extraction within Marion County. Several sites within the major stream valleys and in very low-density residential areas are suggested for potential sand and gravel production. A few possible open-pit quarry sites with less than 50 feet of overburden are present in the northern and northeastern parts of the county. Underground mining of crushed stone may be possible in northwestern Marion County, where competition from quarry operations is minimal.

INTRODUCTION

Four quarries that were not in operation prior to 1966 will probably produce and ship stone to the Indianapolis area this year. This illustrates the rapid growth of the aggregate market in Indiana. During the last 20 years, sand and gravel production has more than tripled; crushed stone production has more than quadrupled. Aggregate production now stands at an all-time high.

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Indianapolis is typical of many large metropolitan areas whose expansion demands more and more aggregate but whose sources of aggregate become increasingly restricted by land use and zoning regulations. The Indianapolis metropolitan area (Marion County) leads the state both in population growth -- 254 per cent increase from 1900 to 1960 (Indiana State Board of Health, 1961, p. 10 -- and in the consumption of aggregate for the construction industry. Eight active pits in Marion County currently supply the Indianapolis market with sand and gravel (Fig. 1). Crushed stone is imported from 11 quarries, some as far away as 45 miles.

Finding an adequate market demand and a suitable aggregate material that can be produced and transported to the market for a profit determines the location of a new pit or quarry site. This paper examines the geologic factors affecting the present and potential sources of aggregates available in and near the rapidly growing market area of Indianapolis.

SAND AND GRAVEL

Present Production

In 1965 eight plants along the valleys of White River, Eagle Creek, and Fall Creek (Fig. 2) produced 4, 494, 201 tons on sand and gravel valued at $3, 921, 629. Marion County has the largest production of sand and gravel of any county in Indiana, almost 18 per cent of the state's total output. Harrison (1963, p. 5) reported about 1, 000 widely scattered active, active-on-demand, or abandoned sand and gravel pits in the county.

Characteristics of the Surficial Materials

Glaciation has had a profound effect on Marion County. Not only has ice sculptured the bedrock surface, but it has left behind thick deposits of unconsolidated materials that in places contain large reserves of sand and gravel. These unconsolidated deposits range from about 15 to 400 feet in thickness (Fig. 3). We have grouped the unconsolidated deposits of the county into three types according to their genesis: till, kame, and outwash and alluvial.

Till -- About 80 per cent of the ground surface in Marion County consists of till that is composed mainly of fine sand, silt, and clay. Scattered coarser material, such as granules, pebbles, cobbles, and boulders, may make up about 10 per cent of the total composition (Harrison, 1963, p. 32). Till was deposited directly by the melting of glacial ice. Lenses and beds of stratified sands and gravels, however, occur within till deposits and these were deposited by meltwater streams. Generally these intertill sands and gravels cannot be mined economically because they are not thick enough to warrant the removal of the till overburden. However, the inactive McMahan Pit in eastern Marion County, near the junction of U. S. 40 and Franklin Road, has produced gravel from an intertill deposit.

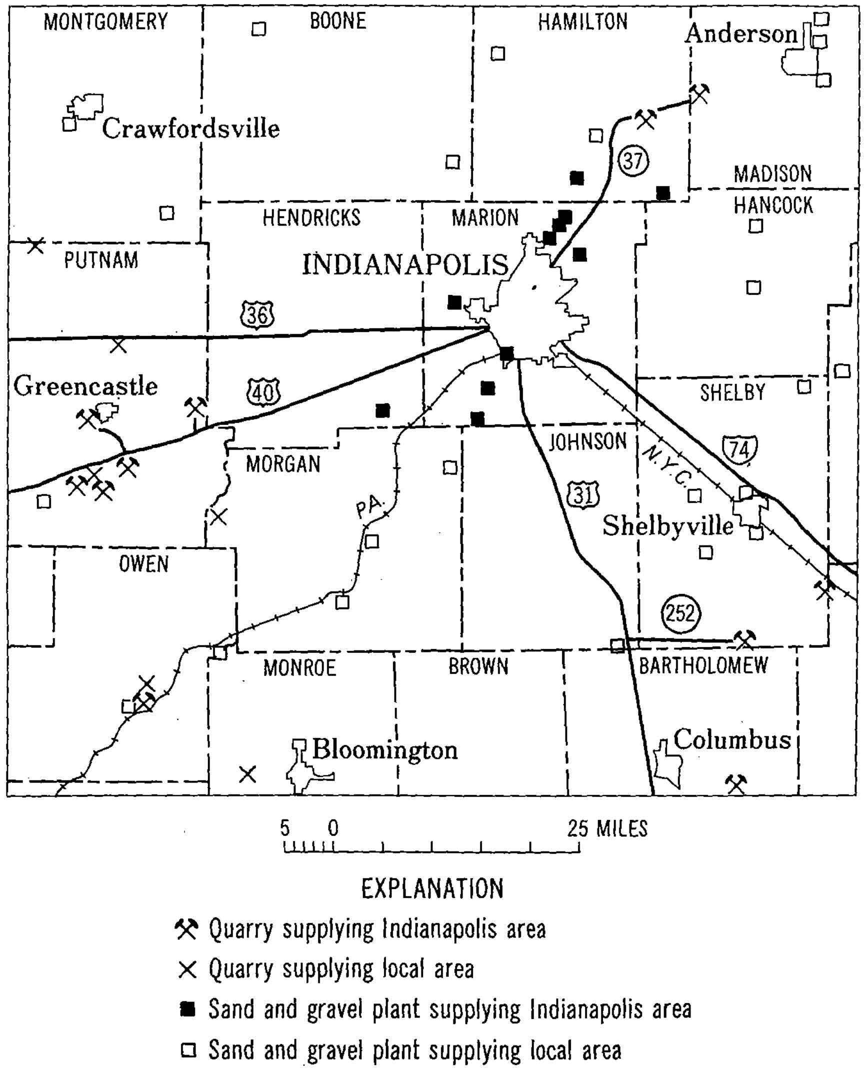

Figure 1. Map showing location of pits and quarries in the Indianapolis region.

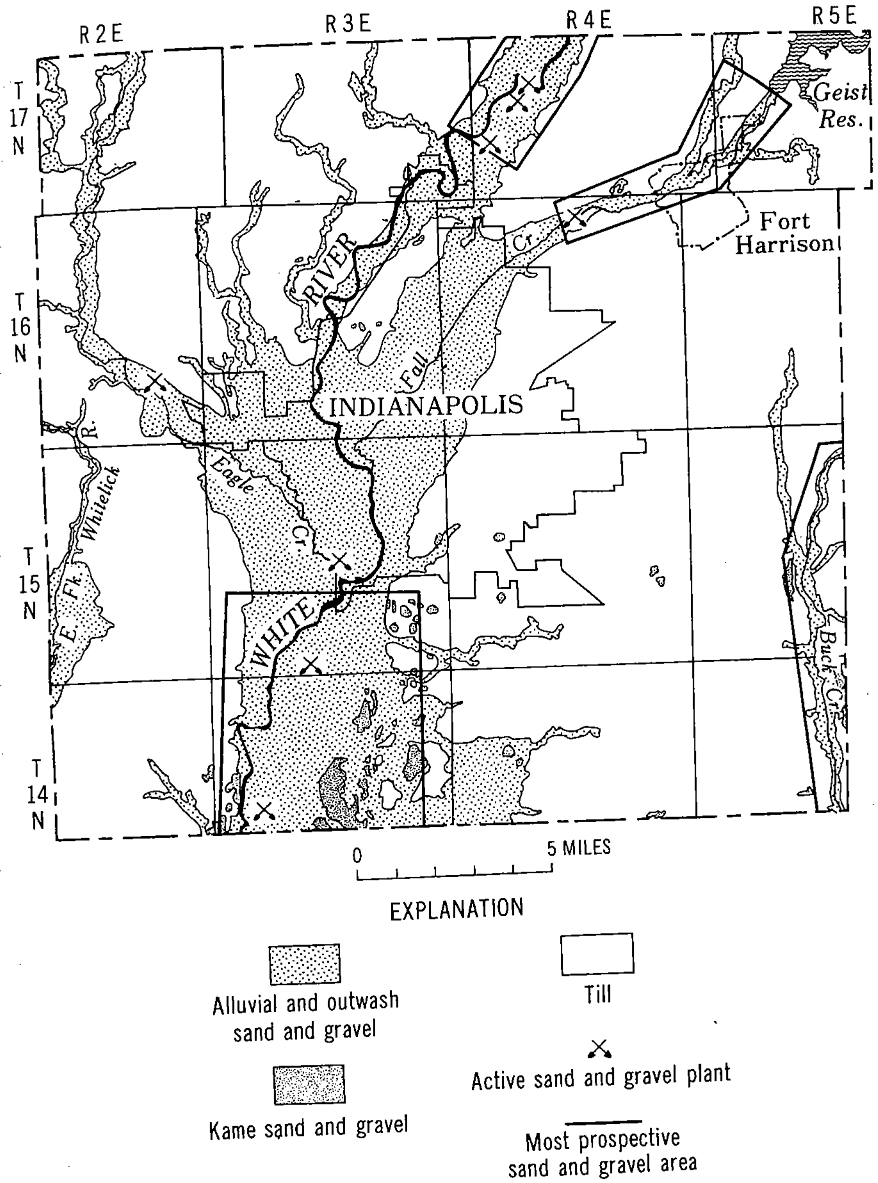

Figure 2. Map of the surficial geology of Marion County and locations of active sand and gravel plants. Modified from Harrison, 1963, pl. 1.

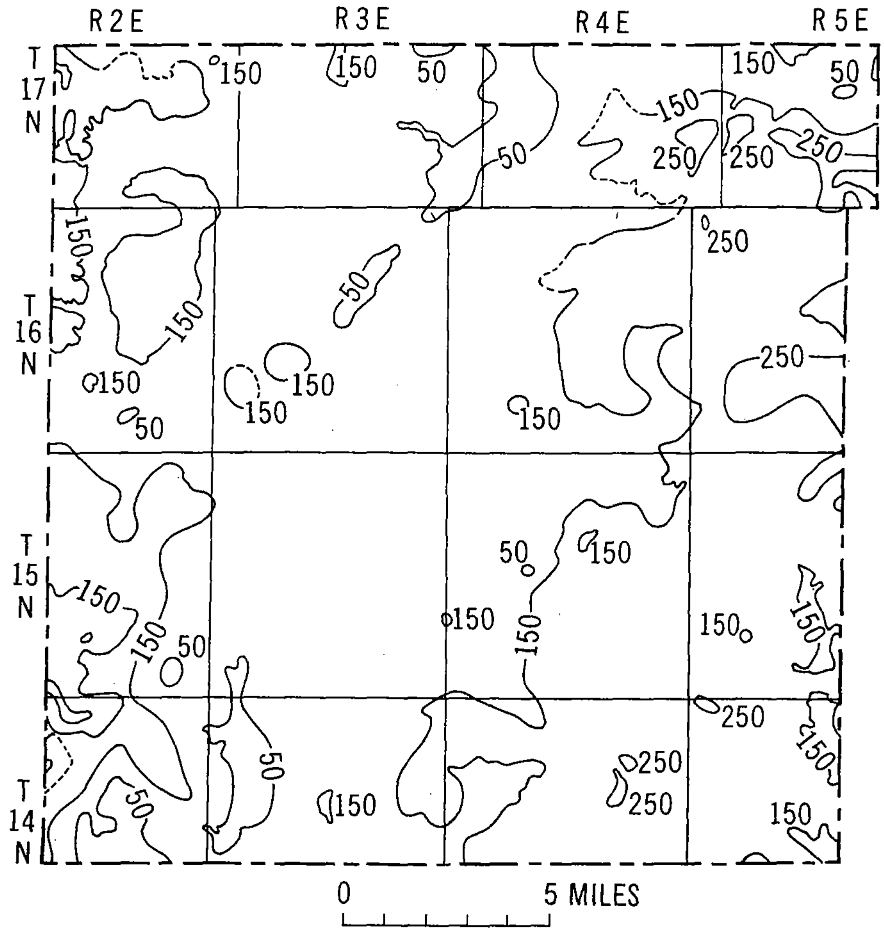

Figure 3. Map showing thickness of unconsolidated materials in Marion County, Indiana. Modified from Harrison, 1963, pl. 3.

Kame -- Kame deposits are concentrated in the southern half of Marion County (Fig. 2) and can be recognized as small hills and ridges that rise abruptly above the surrounding countryside. Kames consist mainly of silt, sand, and gravel that are discontinuous in many places and may abut sharply against clay tills. These deposits were formed by meltwater streams that moved along and within the ice mass and dumped debris in large cracks and crevasses near the ice margin.

Many kames have been used as borrow pits, and a few are currently active on demand. Kames are generally not considered good sand and gravel prospects because the gravels are discontinuous, clay content is high, and reserves are limited. If small quantities of sand and gravel are needed, these deposits may be adequate.

Outwash and Alluvium -- Outwash deposits, generally found as broad plains or as valley fill along the principal drainageways, are more prospective for sand and gravel than are kames or till. Outwash consists mainly of sand and gravel that was deposited by sediment-laden meltwater streams beyond the margin of the glacier. These gravels are important commercial gravel deposits because of the size of the deposits, consistency of particle sizes, and low silt and clay content.

Alluvium results from modern stream deposition and consists partly of reworked underlying outwash sands and gravels and finer silts and clays. Alluvial deposits generally do not contain extensive units of coarse gravels; however, alluvial deposits, because they are generally less than 20 feet thick, can be easily mined with underlying outwash material.

Gravel Gradation and Quality -- The principal source of gravels in Marion County is the White River valley. Sieve analyses of sand and gravel samples from this area generally indicate 25 to 50 per cent gravel. The size of outwash sediments varies considerably along the White River, but decreases markedly from north to south (Fig. 4).

Most gravels from Marion County tested for quality as coarse aggregates pass Indiana State Highway Commission specifications for grade A aggregate. The results of recent testing are summarized in Table 1.

Potential Sites

A study by the Indiana Geological Survey (Harrison 1963, p. 55) found that the terrace and suballuvial outwash deposits along the valleys of White River, Eagle Creek, Fall Creek, Buck Creek, and the East Fork of Whitelick River were the major sources of gravel in Marion County. It was estimated that 1,265,000,000 tons of recoverable pit-run materials are available (Harrison, 1963, p. 57). Land use changes since 1963 and proposed reservoirs have reduced

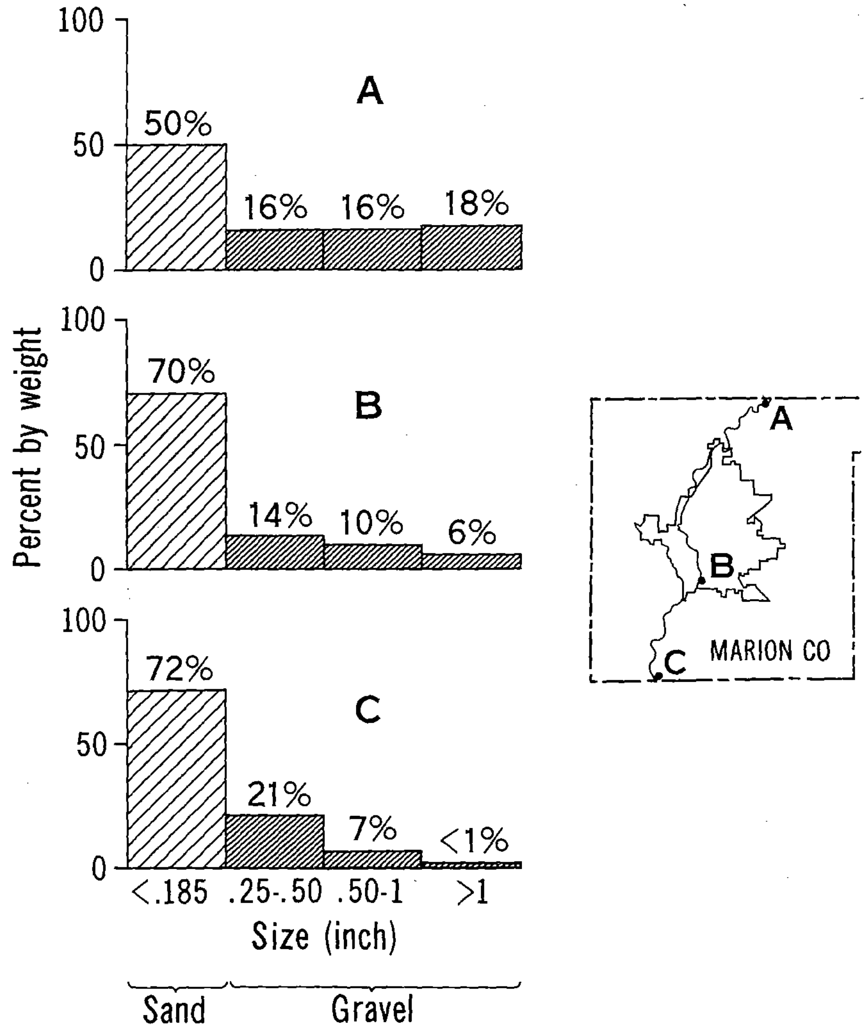

Figure 4. Histograms showing size distribution of sand and gravel in Marion County.

TABLE I

SUMMARY OF 14 PHYSICAL TESTS OF EIGHT GRAVEL DEPOSITS IN MARION COUNTY DURING 1965 AND 1966[1]

Physical Test	Size of Gravel[2]	Range Minimum	Range Maximum	Average	Standard Deviation[3]
Specific	2U	2.70	2.75	2.73	0.02
Gravity	5L	2.72	2.76	2.74	0.01
Per Cent Abrasion	2U	20.4	34.7	24.7	6.48
Loss	5L	--	--	--	--
Per Cent Soundness	5L	--	--	--	--
Loss	5L	--	--	--	--
Per Cent Soft	2U	0.8	2.6	1.5	0.06
Materials	5L	0.0	2.9	1.8	0.56
Per Cent Chert 2.45	2U	0.0	2.7	0.63	0.69
Specific Gravity	5L	0.0	1.7	0.8	0.14
Per Cent	2U	0.8	1.6	1.3	0.22
Absorption	5L	1.3	2.0	1.5	0.22

1 Data provided by State Highway Department.

2 Sizes specified by State Highway Commission of Indiana (1963, p. 516): "2U" corresponds to corase aggregate ranging in size from 1/2" to 2-1/2"; "5L" corresponds to coarse aggregate that ranges in size from about 1/2" to 1-1/2".

3 The standard deviation is a statistical measure of the variability of the sample from the mean value. About 68 per cent of samples fall within $\pm$ one standard deviation of the average value.

the recoverable amount of sand and gravel by more than 50 per cent. Areas that are now most prospective for gravel are (Fig. 2):

a) White River valley north of 73rd Street to county line,

b) White River valley south of Troy Avenue to county line,

c) Buck Creek valley from east county line to south county line, and

d) Fall Creek valley north of 56th Street to Geist Reservoir.

These areas are suggested for commercial gravel plants within the present compatible land use areas of the county. However, special permits by the Marion County Planning Commission allow extraction of aggregate in any zoning district pursuant to a State Highway Department contract for state or federal highway construction.

All of Marion County will ultimately be developed into an urban area, and gravel producers will be forced to look to adjacent counties for gravel from outwash deposits along the White River valley in southern Hamilton County, and to a lesser degree from the valleys of Eagle Creek in Boone County and Fall Creek in Hamilton and Madison Counties. Southern Marion County can look to deposits along the White River valley in northern Johnson County and to deposits along Sugar Creek in eastern Johnson and western Shelby Counties (Carr, 1966).

CRUSHED STONE

Present Supply

At the present time the Indianapolis area is being supplied with crushed stone produced in Owen, Putnam, Hamilton, Madison, Bartholomew, and Shelby Counties (Fig. 1). Stone shipped in by rail and road is stock-piled in the southeastern, northeastern, and western fringes of the metropolitan area. The cost of transporting coarse crushed stone aggregate more than 31 miles to the Indianapolis market increases the price of some sizes by about 82 per cent of the fob price. The lack of competition from nearby crushed stone plants has also allowed the price of gravel to increase slightly above that normally charged in stone-producing areas.

Stratigraphy

As previously described, Marion County is covered by about 15 to 400 feet of different types of unconsolidated glacial deposits that have effectively masked the bedrock. However, cores, chip samples, electric logs, driller's records, and seismic data available at the Indiana Geological Survey, Bloomington, provide sufficient information to assess the geologic feasibility of producing crushed stone in most of the area.

The Indiana Geological Survey has taken cores in sec. 20, T. 17 N., R. 4 E., and sec. 18, T. 16N., R. 4 E. Chemical analyses and a split from each core are kept on open file along with samples and logs of several wells drilled by industry in the county. Numerous additional logs of water wells are available at the Division of Water, Department of Natural Resources, Indianapolis.

Most of Marion County was surveyed with refraction seismic equipment (Saenger, 1958) and the results were published by the Indiana Geological Survey (Harrison, 1963). These data represent the most detailed investigation of geologic conditions in Marion County. It should be noted, however, that the seismic data were obtained with in-line, single-ended shots; thus bedrock topography and lithologic variations in the overburden could influence the calculation of overburden thickness and identification of the rock type.

Mississippian Rocks -- Mississippian shales, thin limestones, and some sandstones of the Borden Group underlie the western part of the county (Figs. 5 and 6), but these rocks are generally not prospective for crushed stone aggregate. The only production known to us came from a small quarry that exploited an argillaceous bioherm in southeastern Montgomery County. This quarry produced mainly agricultural limestone, but also supplied a small amount of crushed stone to a local market. The quarry ceased operation in 1951.

Devonian Rocks -- The Devonian rocks consist of as much as 122 feet of the New Albany Shale that overlies as much as 102 feet of the North Vernon Limestone, Jeffersonville Limestone, and Geneva Dolomite in the central part of Marion County. The North Vernon Limestone contains approximately 34 feet of gray to buff fine-grained skeletal limestone with some argillaceous zones and minor amounts of chert and phosphatic nodules. The North Vernon appears to be a purer limestone in the Indianapolis area than it is in the outcrop area of southern Indiana, but its value as a source of aggregate has yet to be determined. The Jeffersonville Limestone consists of a maximum of 45 feet of tan to buff medium-to fine-grained partly skeletal limestone with a few argillaceous laminae and as many as three 1-inch sand zones at the base. A gray calcareous dolomite forms the lower 4 to 8 feet of the unit and directly overlies the Geneva Dolomite. The Geneva is typically dark-brown carbonaceous dolomite that attains a maximum thickness of approximately 23 feet in the Indianapolis area. Jeffersonville and Geneva rocks, commonly used for aggregate in southern Indiana, are shipped into Indianapolis by truck from Shelby and Bartholomew Counties.

Silurian Rocks -- Silurian rocks of the Wabash Formation underlie the Geneva and subcrop in eastern Marion County. The Wabash consists of three distinct lithologic units, all of which are present in the Indianapolis area. Commonly, the youngest unit is the Liston Creek Limestone Member, a 30-to 60-foot unit that is generally thin bedded and very cherty. Two quarries produce aggregate from the Liston Creek in Grant and Hamilton Counties, but the quality of the material depends mostly on the specific gravity of the

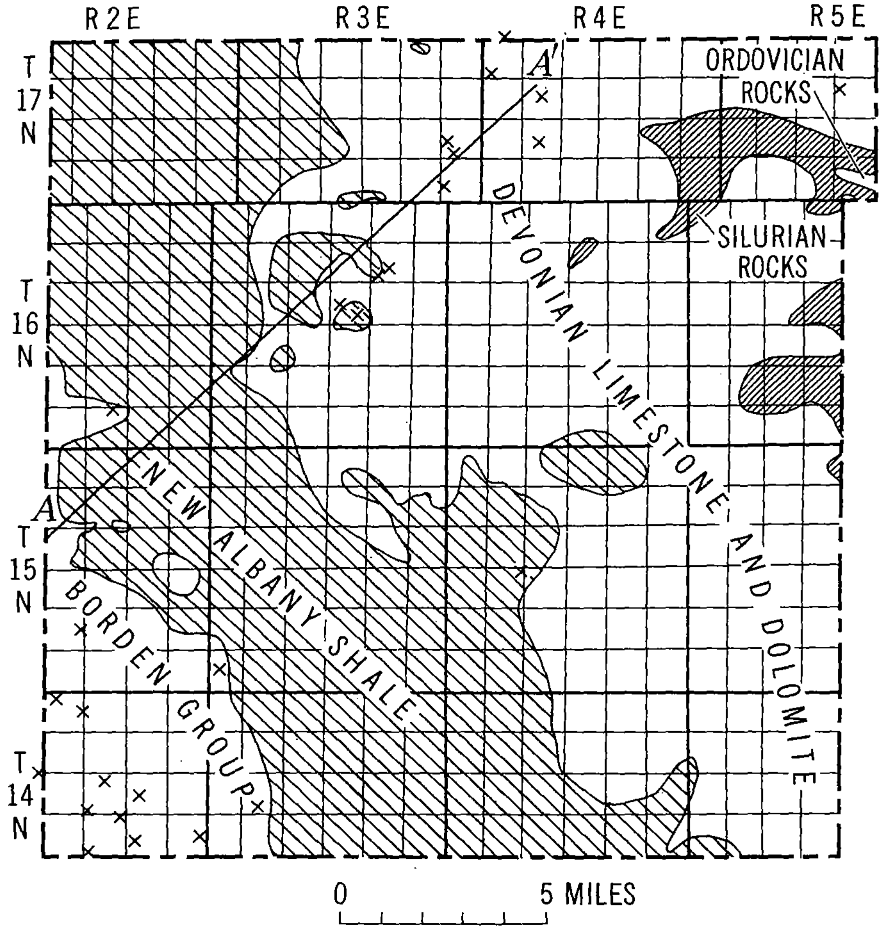

Figure 5. Map showing bedrock geology of Marion County. Modified from Harrison, 1963, pl. 2.

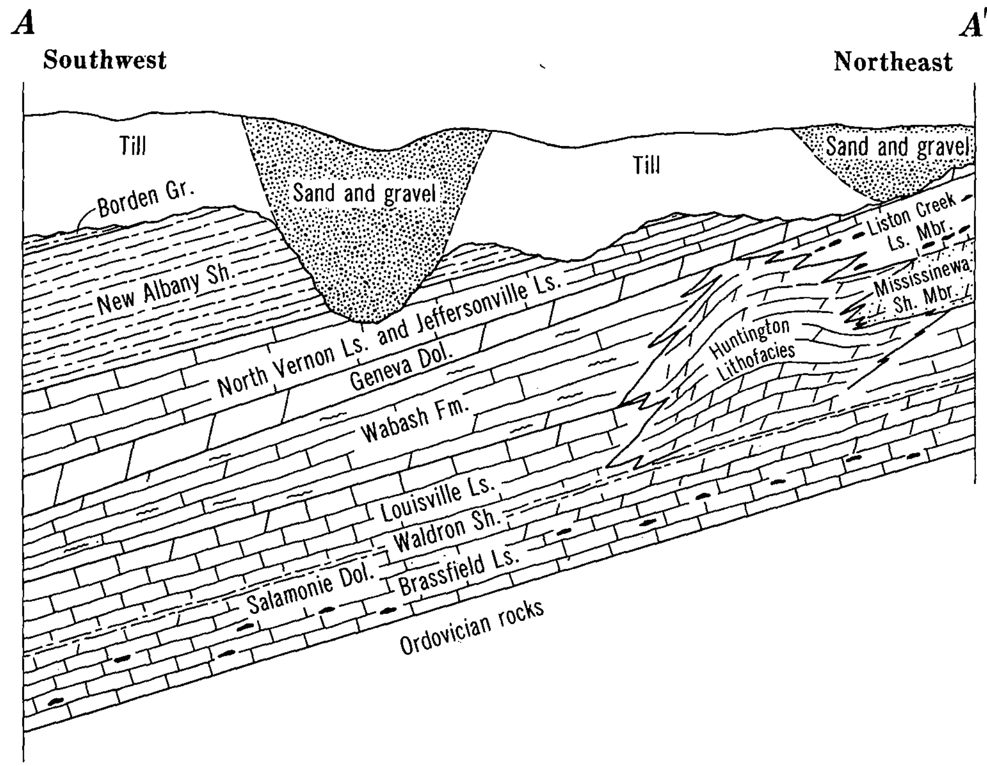

Figure 6. Generalized southwest-northeast cross section showing glacial and bedrock geology of Marion County.

chert. Beneath the Liston Creek is the Mississinewa Shale Member, which, where typically developed, is a silty argillaceous dolomite. In Marion County the Mississinewa has a maximum thickness of approximately 60 feet. It has never been successfully used as a source of aggregate in the outcrop area, but in the Indianapolis region it appears to be a much purer carbonate rock and may warrant further investigation. Both the typical Liston Creek and the Mississinewa lithologies may be replaced laterally by the Huntington Lithofacies, which is generally a cream-colored biohermal dolomite. Numerous quarry operations exploit rocks of the Huntington lithology in northern Indiana, and Huntington dolomite provides a part of the crushed stone aggregate currently being shipped into Indianapolis from Madison County. Esarey and Bierberman (1948, p. 36) recognized reef-like deposits in the Indianapolis area and correlated them with the Waldron Shale and Salamonie Dolomite, but subsequent coring showed the reef facies are actually younger and should be correlated with the Wabash Formation and the underlying Louisville Limestone (French, in press).

Although pre-Wabash rocks have not been found at the bedrock surface in any wells yet drilled in the area, the seismic data and structural conditions indicate that older Silurian and Ordovician units may actually subcrop beneath the glacial deposits in northeastern Marion County (Saenger, 1958). The Louisville Limestone, Waldron Shale, Salamonie Dolomite, and Brassfield Limestone together exceed 130 feet in thickness and form the remainder of the Silurian rocks in this area (French, in press). Carbonate rocks from these units are used as a source of aggregate in many parts of eastern and southern Indiana. The upper part of the Ordovician rocks consist of intercalated limestone and shale and cannot be considered prospective.

Potential Sites

The density and growth rate of the population and the lack of Grade A coarse aggregate production within 30 miles of the city center provide the economic stimuli necessary for the opening of either a quarry or an underground mine in the Indianapolis area.

Open-Pit Quarrying -- Seismic refraction data gathered in 1957 (Saenger, 1958) and later published by the Indiana Geological Survey (Harrison, 1963) indicate several areas where the glacial overburden is relatively thin. Harrison (1963, p. 58) mentioned unconsolidated deposits only 15 feet thick in sec. 36, T. 17 N., R. 3 E., and in sec. 15, T. 16 N., R. 3 E. Both areas are in the valley of the White River where water problems might be unmanageable, and both are in or very near zoned acreage.

Harrison also pointed out sites in secs. 7 and 18, T. 17 N., R. 4 E., with as little as 28 and 34 feet of glacial overburden. This general area was proved independently by the American Aggregates Corp. which found bedrock at a depth of about 30 feet in the new sand

and gravel pit in sec. 9, just north of the Marion-Hamilton county line. Seismic information and projection of well control data, on the basis of a regional dip of 18 to 23 feet per mile S. 57 W., indicate that the bedrock surface is probably composed of Devonian carbonate rocks or possibly the upper part of the Wabash Formation. Harrison suggested a second area of interest in sec. 22, T. 17 N., R. 5 E., where the seismic data showed only 39 feet of drift. Again the seismic information and regional stratigraphic and structural control indicate a bedrock surface of Devonian rocks. A major source of concern in this area, however, is near-by Geist Reservoir, the surface of which is 4 feet above the bedrock surface.

Underground Mining -- Although the cost of producing crushed stone is considerably greater from an underground mine than it is from an open quarry, the size of the market area and the lack of nearby competition warrant its consideration. Because of the long road and rail hauls from Owen, Putnam, Madison, Bartholomew, and Shelby Counties, some of the coarser aggregates are currently retailing for $2.45 to $2.65 per ton in the Indianapolis area. The direct cost of producing aggregate from one under-ground limestone mine has been estimated at $1.211 per ton as compared with the $0.98 estimated average cost of open-pit methods (Evans and Eilertsen, 1957, p. 41; Tyler, 1964, p. 182). Gillson and others (1960, p. 185) revised Evans' earlier estimate of $1.211 per ton direct costs for underground mining to $1.975 per ton direct cost that includes a 10-per cent gross return before taxes.

The most promising sites for underground mining in Marion County are in areas where an adequate cover of the New Albany Shale overlies a full section of Devonian limestone and dolomite. The relatively impermeable shale would prevent excessive groundwater from moving into the mine from overlying glacial deposits. Location of a mine west of the updip limit of the shale should also decrease the probability of secondary porosity in the carbonate rocks.

A complete section of Devonian strata beneath the New Albany Shale includes a maximum of about 79 feet of North Vernon and Jeffersonville limestone and 23 feet of Geneva dolomite. The chemical composition of these rocks in the Indianapolis area is shown in Table 2, and the range of physical characteristics of samples from active quarries in southern Indiana is shown in Table 3. The uppermost Silurian units in the area do not commonly provide good aggregate except where the typical Liston Creek and Mississinewa lithologies have been replaced by the Huntington reefs. Reef type rocks have been recognized in the Silurian section immediately beneath the city in Ts. 15 and 16 N., R. 3 E. (Esarey and Bieberman, 1948) but have not been recognized in surrounding areas where mineral extraction might be feasible.

If an underground operation is to be considered in the Indianapolis area, it appears reasonable to assume that it should be west of the updip limits of the New Albany Shale and in an area underlain by

TABLE 2

CHEMICAL COMPOSITION OF SOME DEVONIAN CARBONATE ROCKS FROM THE INDIANAPOLIS AREA

Unit	$CaCO_3$	$MgCO_3$	SiO_2	Al_2O_3	Fe_2O_3
	Location: sec. 7, T. 17 N., R. 3 E.				
North Vernon and	82.8	9.2	5.9	0.7	1.0
Jeffersonville Limestones	93.5	2.2	3.5	0.2	0.3
Geneva Dolomite	52.0	38.6	7.4	0.8	0.8
	Location: sec. 18, T. 16 N., R. 3 E.				
North Vernon and	76.5	16.2	5.3	0.6	0.9
Jeffersonville Limestones	92.4	2.2	4.3	0.4	0.5
Geneva Dolomite	52.2	39.1	6.6	1.1	0.6
Geneva Dolomite	49.7	34.7	14.7	0.2	0.7

TABLE 3

RANGE OF PHYSICAL CHARACTERISTICS OF SOME DEVONIAN AND SILURIAN CARBONATE ROCKS IN INDIANA[1]

	Number of Samples and Locations[2]	Specific Gravity	Abrasion Loss	Soundness Loss	Adsorption
North Vernon Limestone	15 (9)	2.5-2.72	24.7-45.5	6.5-45.5	0.6-6.4
Jeffersonville Limestone	32 (11)	2.2-2.73	22.3-44.0	0.9-30.5	0.4-7.7
Geneva Dolomite	10 (4)	2.3-2.68	27.9-37.6	1.9-12.0	2.3-5.0
Huntington Lithofacies (Wabash Formation)	39 (11)	2.5-2.77	19.7-39.5	0.9-20.2	0.4-2.8
Liston Creek Limestone Member ..	11 (3)	2.5-2.78	23.2-28.8	2.6-48.0	1.1-3.5
Louisville Limestone	25 (10)	2.4-2.78	22.9-37.1	3.5-34.4	0.7-4.8
Salamonie Dolomite	15 (5)	2.4-2.71	32.1-52.1	0.5-23.7	1.3-3.9
(Laurel Member)	29 (9)	2.5-2.80	23.5-36.8	1.0-43.6	0.5-2.7
Brassfield Limestone	2 (1)	2.6-2.76	28.3-35.6	6.2-11.2	0.7-2.6

[1] Data compiled from Indiana Highway Commission records.

[2] Locations shown in parentheses.

till. Such a location would minimize the possibility of excessive water and maximize the thickness of the exploitable rock section. The problem of competition from quarries shipping stone from Putnam and Owen Counties appears to reduce further the choice of location to the north and northwest side of Marion County. One possible site that is already zoned for heavy industry (Metropolitan Planning Dept., 1965) and is accessible by road and rail is in sec. 23, T. 17 N., R 2 E.

WATER

Groundwater is one of the more significant deterrents to either openpit or underground operations in Marion County. In a detailed report on the water resources of the Indianapolis area, published in 1955, Roberts, Widman, and Brown considered (p. 30) the Devonian and Silurian carbonate rocks to be one hydrologic unit. Water in both was found to be under artesian pressure with variable yields, and the permeability was considered variable but generally not high. Recharge of the Devonian and Silurian reservoir rocks is at a maximum in the White River and Fall Creek valleys, where permeable unconsolidated deposits directly overlie the limestone and dolomite. Least recharge occurs in the uplands where the bedrock is overlain by clayey glacial drift. Most of the surficial deposits in Marion County are composed of clay-till with only a few thin interspersed beds of sand and gravel. Groundwater problems in this type of material are minor. The outwash gravel deposits in the stream valleys are by far the greatest source of water in the area. In the White River valley, Roberts, Widman, and Brown (1955, p. 32) recognized an upper, readily depletable sand underlain by clay and two to four water-bearing gravels that may function as a single hydrologic unit. The water is under artesian pressure, but apparently the unit is easily rechargeable and is the greatest source of supply in the Indianapolis area.

Many wells in the Indianapolis area are capable of being pumped at 500 to 700 gallons per minute. In the stream valleys, where recharge is more rapid, 1, 000-gpm capacity is not uncommon and as much as 3, 000 gpm has been recorded. The largest capacity well in the state is near the White River and is capable of producing about 10, 000 gpm from the unconsolidated deposits.

Most of the water in the bedrock is found in the upper 25 feet of partly eroded carbonate rocks where secondary porosity has been developed. Bedrock wells generally average 25 to 50 gpm where the limestone is still capped with shale and about 500 to 700 gpm where the limestone or dolomite is directly overlain by glacial material.

SUMMARY

Marion County is one of the largest markets for aggregate in Indiana. Population growth, urbanization, and land use restriction are rapidly diminishing the number of available crushed stone and sand and gravel deposits within the county. The comprehensive land use plan for Marion County, adopted May 12, 1965, allows for mineral extraction in certain areas, but each site is approved on an individual basis. It appears probably that only those sites lying outside the city limits or in very low-density residential or industrial zones are likely to receive permits in the foreseeable future.

Several sites within the major stream valleys and in very low-density residential areas have been suggested for potential sand and gravel production. Additional sites have been recommended in adjacent counties. If the few figures published on cost of production are realistic, an underground mine in northwestern Marion County might conceivably be an economic venture. Potential quarry sites appear to be limited to the northern and northeastern part of the county.

ACKNOWLEDGMENTS

We thank the Testing Laboratory of the Indiana Highway Department for data it supplied on the physical properties of gravel and crushed stone. All chemical analyses were performed by the Geochemistry Section of the Indiana Geological Survey. Estimates of average water well capacities were provided by the Division of Water, Department of Natural Resources.

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Shallow Geophysical Exploration by the Michigan Department of State Highways

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ABSTRACT

Michigan uses the Barnes layer method and Moore cumulative curve method of resistivity interpretation. The mechanics, application, advantages, and limitations of each method are discussed and examples given. The two methods generally complement each other. Use and interpretation of the refraction seismic method are also discussed. Michigan's two kinds of resistivity instruments and their advantages and disadvantages are described. The Department has single- and multi-trace seismic equipment, used for various purposes. A variety of boring equipment is available for correlation borings and soil and rock sampling. Geophysical surveys are conducted on proposed roadway cut sections and borrow areas, the latter including dry and underwater borrow. In addition, surveys are made to aid in solving special problems, such as bedrock strength evaluations and materials inventories for land appraisal and right-of-way litigations.

INTRODUCTION

The State of Michigan was among the pioneers in the development of soils engineering. This included the adaptation of the agricultural pedological soils system to highway design and construction. While mapping and classification of the agricultural soils series yield considerable information, this system needs to be supplemented in specific areas where deeper, detailed subsoil information is required.

Realizing this, the Department entered the field of geophysics in 1949, with the purchase of an earth resistivity instrument. It was found that while the resistivity method showed considerable promise, methods of interpretation then current did not consistently yield the desired detailed subsurface data. This eventually led to the development of the Barnes layer method of interpretation (1), which is now used throughout the world.

The resistivity method proved very successful, but certain limitations inherent in it led the Department to investigate and in 1958 to purchase a small single-trace engineering-type seismograph. This equipment was to supplement and complement the earth resistivity method. Several years later, a multi-trace seismograph was acquired, greatly expanding the versatility and capabilities of the Department's Geophysical Section.

At present, the Geophysical Section includes one professional engineer, eight geophysicists, three technician-class employees, and eight laboratory aide-class employees. Two resistivity field parties and one seismic field party are maintained in the field on statewide surveys throughout the year. Additional field parties and personnel are added for the summer months as needed. A truck-mounted power auger is also assigned to the Section. This auger is capable of collecting subsoil and rock samples under practically any subsurface condition.

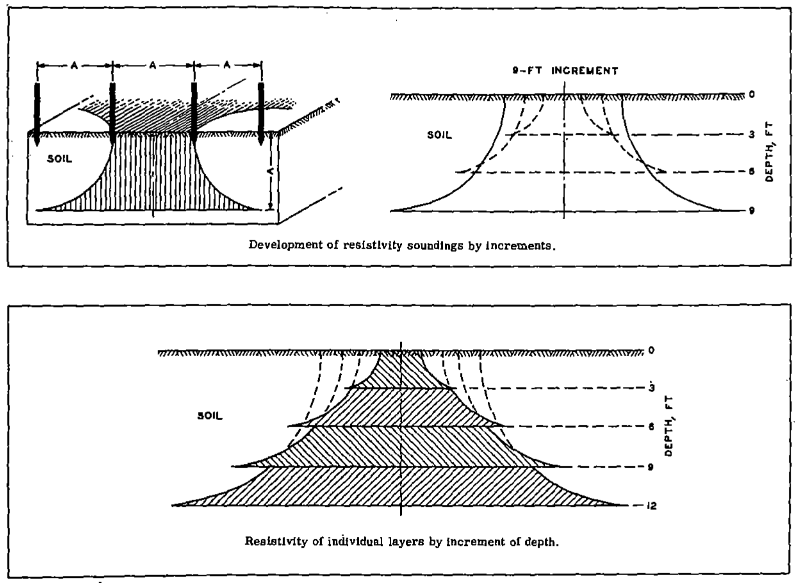
GEOPHYSICAL METHODS OF INTERPRETATION

Earth Resistivity

The Department uses two methods of resistivity interpretation -- the Barnes layer method and the Moore cumulative curve method. Both use the same basic field resistivity data obtained from the Wenner electrode configuration (2), consisting of four electrodes driven into the ground along a straight line at equal intervals. The resistivity sounding consists of expanding the electrode spacing in increments about a fixed point. A subsoil resistivity reading is taken at each increment, resulting in a detailed electrical log of the subsoil.

Typical resistivity soundings are illustrated in Figure 1. The depth of soil being measured at each increment reading is equal to the electrode spacing. Each increment reading includes the entire soil mass from the ground surface to each particular depth increment being measured. Therefore, each successive resistivity measurement includes all the soil measured in the previous reading, plus an additional new amount on the sides and bottom. As successive measurements are taken, each additional, new portion of soil added on the bottom diminishes in relation to the total soil mass being measured. Therefore, a minor change in soil texture near the surface can cause as great or greater change in the resistivity values as can a major change in soil texture at greater depth. This condition is the masking effect inherent in resistivity, and constitutes one of the major problems in resistivity interpretation.

The Barnes layer method of resistivity interpretation was devised to minimize this masking effect. By manipulation of Ohm's law for parallel circuits and a modification of Wenner's formula for computing resistivity, it mathematically strips away the overlying soil layers previously measured, and computes the average apparent resistivity for that volume of soil added by each successive increment.



Development of resistivity soundings by increments.

Resistivity of individual layers by increment of depth.

Figure 1. Typical resistivity soundings.

8/6/64		Sta. 4/19 Survey ℄			(29)
	1/Ra	1/RL	A	ρL	
-3 ○	.002900	.002900	3	197,600	
	.006100	.003200	6	179,100	
	.012000	.005900	9	97,100	
○	.023600	.011600	12	49,400	
-2	.035700	.012100	15	47,300	
○	.040400	.004700	20	203,200	
	.050100	.009700	25	98,500	
	.075000	.024900	30	38,400	

Figure 2. Resistivity field notes ($1/R_L$ and ρ_L computed).

The Barnes layer method has been described in the following manner (3). Assuming 3-ft increments, the first increment measures the resistivity of a volume of soil 3 ft in depth and is the resistivity layer value for that increment. The 6-ft increment measuring a volume of soil 6 ft in depth includes that soil mass previously measured by the 3-ft increment, plus an additional 3-ft layer of soil. This can be compared with two resistors in a parallel circuit where the conductance of one resistor is known (the 6-ft increment), and the conductance of the second resistor is unknown (the layer conductance between 3 and 6 ft in depth). Thus, it is possible to solve for the unknown conductance by the following formula (1):

$$\frac{1}{R_n} = \frac{1}{\bar{R}_n} - \frac{1}{\bar{\bar{R}}_{n-1}}$$

where

$\frac{1}{R_n}$ = layer conductance of a given increment, in mhos

$\frac{1}{\bar{R}_n}$ = total conductance between the ground surface and the bottom of the given increment, in mhos

$\frac{1}{\bar{\bar{R}}_{n-1}}$ = total conductance between the ground surface and the bottom of the increment directly above the given increment, in mhos.

The resistivity layer value (or Barnes layer value) for any given increment can then be computed by the modified Wenner formula:

$$\rho_L = \frac{191\ A_L}{\frac{1}{R_n}}$$

where

ρ_L = layer resistivity, in ohm-centimeters

191 = a constant for converting feet to centimeters, including the factor of π

A_L = thickness of any given layer or increment, in feet

$\frac{1}{R_n}$ = layer conductance of any given increment n, in mhos.

The Barnes layer values are used to construct cross-sections from profile-contours. These are pictorial graphs of resistivity traverses depicting arbitrary resistivity layer values as contours which are plotted in relation to the actual ground surface. Other pertinent information is also shown, including stationing, elevations, proposed grade, water table, index correlation boring logs, and laboratory test information.

The development of a resistivity cross-section (Fig. 2 through 6), involves the following steps:

1. The field notes are reduced to layer values for each sounding.

2. The layer values for each sounding are plotted on rho layer contact charts.

3. The depths to arbitrary layer values are measured from the rho layer contact chart and plotted on a cross-section sheet.

4. Profile-contours are drawn by connecting equal layer value plots.

5. Subsurface interpretations are made using correlation borings and knowledge of the geology of the area.

In a given area, it will generally be found that ranges of layer values will correlate with various textural soil classes. The higher layer values will correlate with sand and gravel, while the lower values will correlate with silt and clay. It will also be found that as the profile-contours are drawn, they will outline definite subsurface structures indicative of the interbedded nature of the subsoils. In many cases, the profile-contours can be plotted several ways, yielding different interpretations of subsurface structure. It is essential that the interpretations of the structures outlined by the profile-contours be geologically acceptable for the given area. Actual correlations of the layer values to soil textural types are then made with correlation borings.

The Barnes layer values are very sensitive to changes in subsurface environment. When these changes relate to soil texture, an extremely representative subsurface picture can be constructed. However, when the changes relate to electrolyte concentration in groundwater, another method of interpretation must be used. Under such conditions the Moore cumulative curve method (4) has proved to be valuable. This is because the cumulative curve method is related to rate of change of the resistivity values independent of the actual resistivity values. A resistivity cross-section is developed from cumulative curves (Figs. 7 through 10) as follows:

1. Field notes are reduced to average apparent resistivity and cumulative resistivity values by using the basic Wenner formula (2):

$$\rho_a = \frac{191\,AE}{I}$$

where

ρ_a = average apparent resistivity, in ohm-centimeters

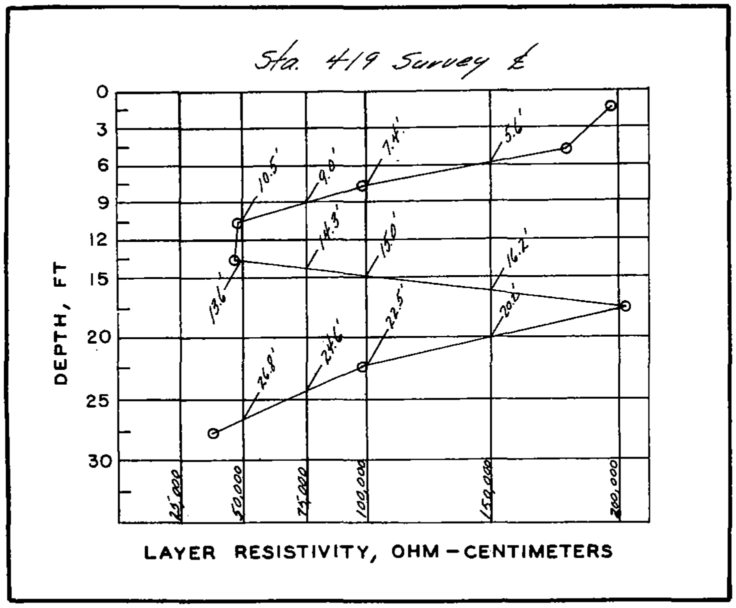


Figure 4. Computer printout page.

RESISTIVITY 12020 PAGE NO. 8

CONTROL SECTION 72023 PIT NO. 1

STATION	READING	DEPTH	RHO	RHODEPTH	INTERCEPT	INTERCEPT DEPTH
419	0.002900	3.	197,570	1.5		
419	0.006100	6.	178,776	4.5		
					150,000	5.6
					100,000	7.4
419	0.012000	9.	97,066	7.5		
					75,000	9.0
					50,000	10.5
419	0.023600	12.	49,392	10.5		
419	0.035700	15.	47,352	13.5		
					50,000	13.6
					75,000	14.3
					100,000	15.0
					150,000	16.2
419	0.040400	20.	203,128	17.5		
					150,000	20.2
					100,000	22.5
419	0.050100	25.	98,365	22.5		
					75,000	24.6
					50,000	26.8
419	0.075000	30.	38,353	27.5		

Figure 3. Resistivity layer contact chart.

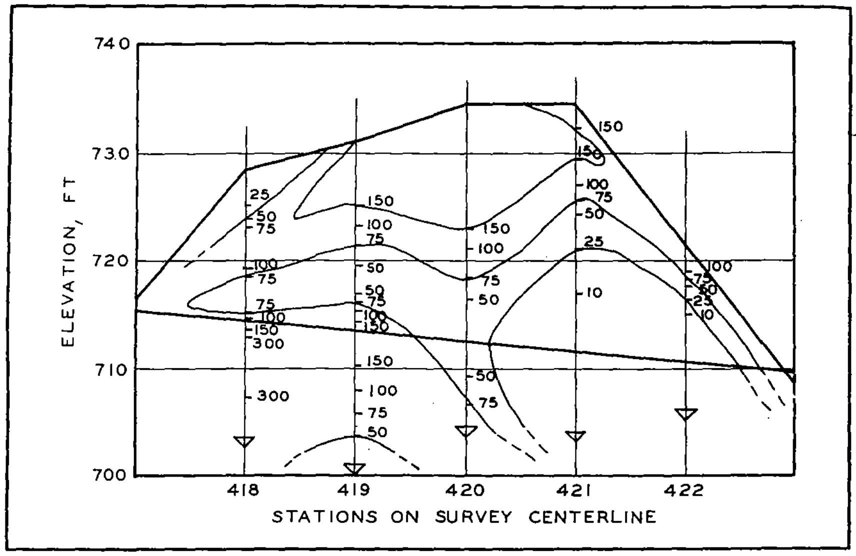


Figure 5. Partially contoured cross-section (Barnes layer method).

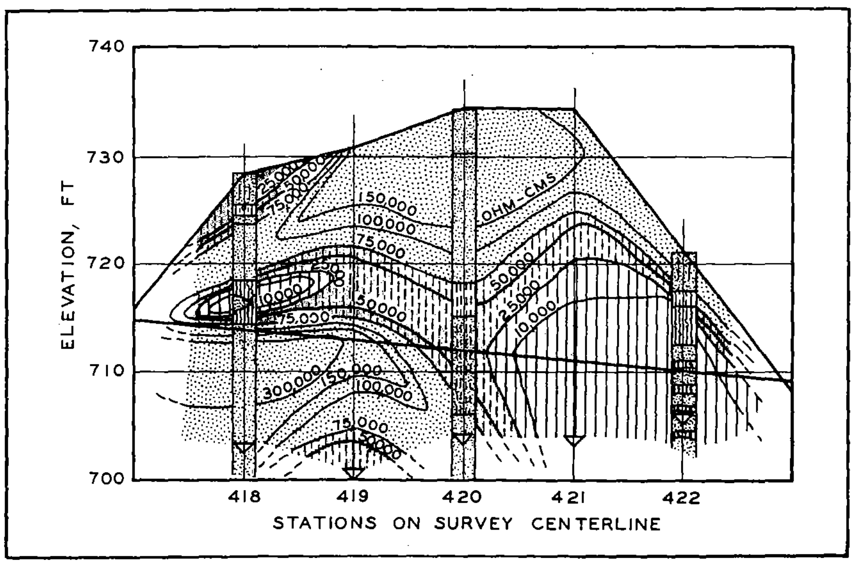


Figure 6. Completed cross-section (Barnes layer method).

191 = a constant for converting feet to centimeters, including the factor of π

A = electrode spacing, in feet

E = potential fall in volts across the inner two electrodes

I = current in amperes carried through the soil mass as introduced through the outer electrodes.

2. The average apparent resistivity and cumulative resistivity values are plotted on a graph using a common abcissa.

3. Depth points are picked from the cumulative curve based on comparison with asymoptotic curve inflections on the average apparent resistivity curve.

4. Cumulative curve points are plotted on the cross-section and interpretations are made using index correlation borings and knowledge of the geology of the area.

The Moore cumulative curve method is particularly applicable in areas containing a sand and gravel deposit overlying clay and having a shallow water table usually containing electrolytes. The geological distribution of the materials is known and the problem is accurate outlining of the materials so that valid quantities can be computed.

In that the cumulative curve method is dependent upon relative rate of change of the resistivity values, the more points obtained across a critical subsurface contact, the more accurately the contact can be delineated. Therefore, when greater accuracy is desired, it is good practice to increase the number of increments in the sounding, thus permitting more detailed depth plots.

With computers available to reduce the resistivity field notes, it is becoming general practice to have both the Barnes layer and the Moore cumulative curve methods reduced so that either or both can be used.

It has been found that the two methods complement each other. An example of this is shown in Figure 11. On Line E, good correlation is obtained with profile-contours from the Barnes layer method. On Line D, however, correlation with profile-contours is negative, and good correlation is obtained with the Moore cumulative curve. Good boring control is essential for correlation of this kind.

This loss in correlation with the Barnes layer method is usually due to the effects of dissolved electrolytes in the ground water. A striking example of this condition appears in Figure 12, where the subsoils consist of a gravel body capped and underlaid by clay. A gravel pit excavation at one end penetrates the clay cap and has removed gravel to the water table. The water table has a definite gradient from right to left.

1/14/65	R Forward	Station 14-6 R Reverse		A	43
	892.6	893.0		3	
	347.2	347.2		6	
	161.4	161.2		9	
	90.91	90.91		12	
	62.50	62.50		15	
	45.45	45.45		18	
	39.21	39.21		21	
	32.46	32.46		24	
	28.28	28.20		27	
	24.69	24.69		30	
	22.07	22.07		33	
	19.60	19.60		36	
	17.85	17.85		39	
	16.52	16.52		42	
	9.01	9.01		45	
	6.17	6.17		48	

Figure 7. Resistivity field notes

RESISTIVITY 12021

CONTROL SECTION 12034 B PIT NO. 2

STATION	FORWARD	REVERSE	AVERAGE	RHO	CUMULATION	DEPTH
14-G	892.6	893.0	892.8	511,574	511,574	3.0
14-G	347.2	347.2	347.2	397,684	909,258	6.0
14-G	161.4	161.2	161.3	277,275	1,186,533	9.0
14-G	90.91	90.91	90.91	208,366	1,394,899	12.0
14-G	62.50	62.50	62.50	179,063	1,573,962	15.0
14-G	45.45	45.45	45.45	156,257	1,730,219	18.0
14-G	39.21	39.21	39.21	157,271	1,887,490	21.0
14-G	32.46	32.46	32.46	148,797	2,036,287	24.0
14-G	28.28	28.20	28.24	145,634	2,181,921	27.0
14-G	24.69	24.69	24.69	141,474	2,323,395	30.0
14-G	22.07	22.07	22.07	139,107	2,462,502	33.0
14-G	19.60	19.60	19.60	134,770	2,597,272	36.0
14-G	17.85	17.85	17.85	132,965	2,730,237	39.0
14-G	16.52	16.52	16.52	132,523	2,862,760	42.0
14-G	9.01	9.01	9.01	77,441	2,940,201	45.0
14-G	6.17	6.17	6.17	56,567	2,996,768	48.0

Figure 8. Computer printout page (Moore cumulative curve).

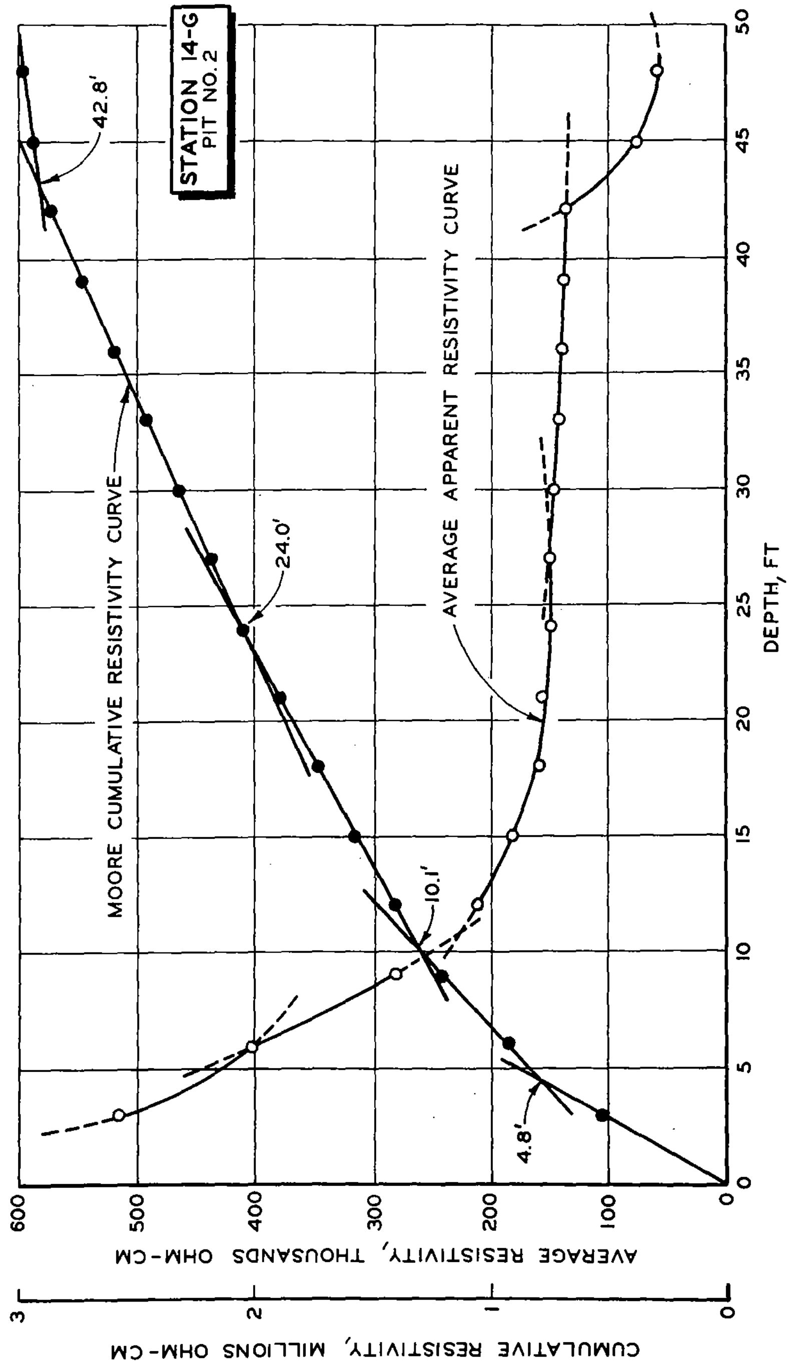


Figure 9. Average resistivity and Moore cumulative resistivity.

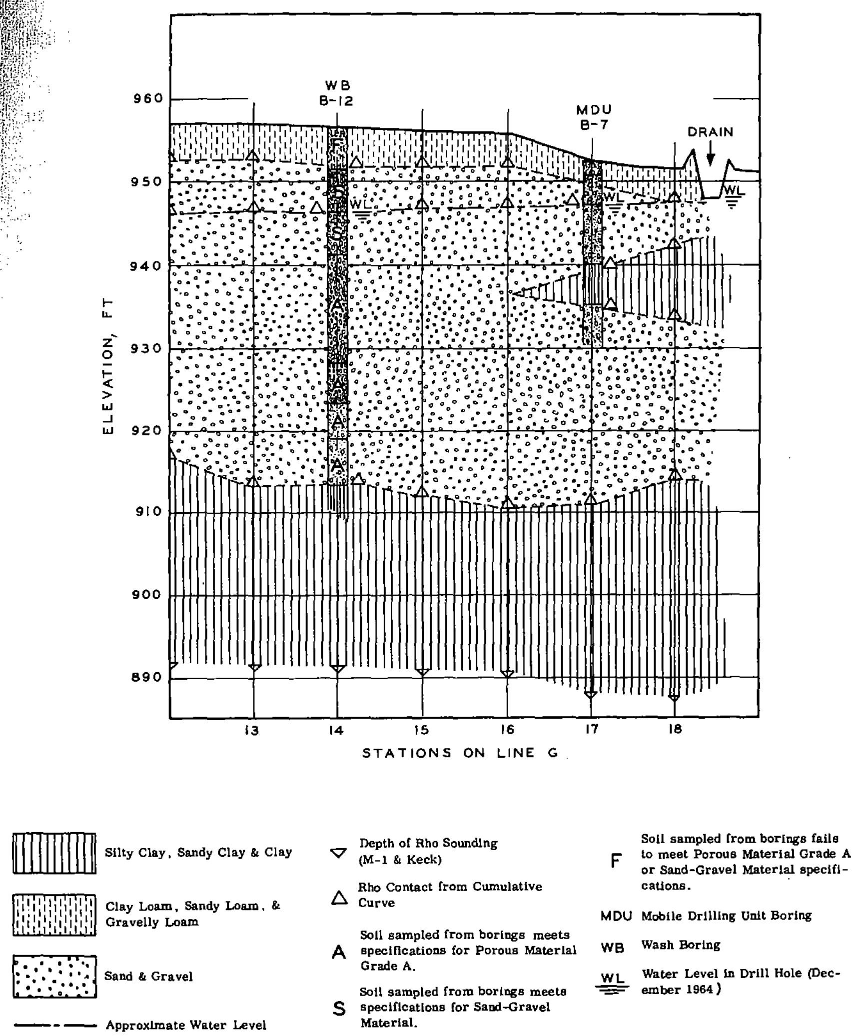


Figure 10. Completed cross-section.

The profile-contours from the Barnes layer method show a high resistivity zone correlating with the gravel body, but the larger values pinch out in the vicinity of Station 7C. The 37, 500 ohm-cm profile-contour correlates well with the clay cap, but only checks in a general way with the underlying clay. The failure of the profile-contours to extend much past Station 7C is due to electrolytes entering the groundwater of the exposed gravel pit excavation and migrating downstream. This condition has been found to be common whereever surface excavations intersect the water table. An awareness of the condition and good boring control will usually solve the problem.

Refraction Seismology

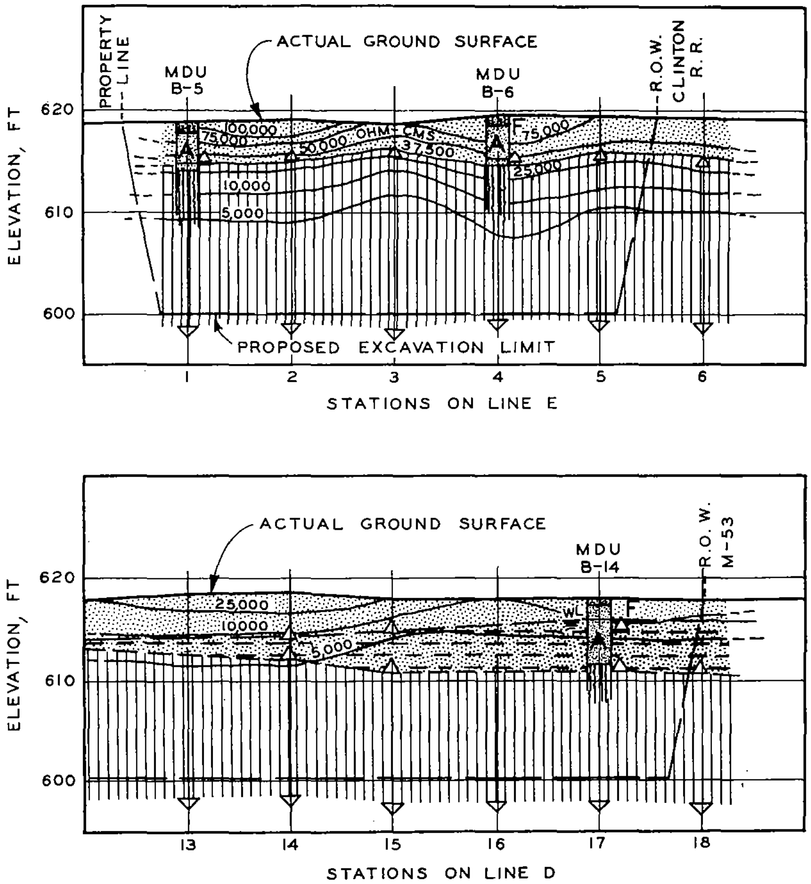
While the earth resistivity method yields considerable information, like all geophysical methods it does have limitations. With the advent of the small engineering-type seismographs, the Department entered the field of seismology. At first, the method was used primarily for location of bedrock and later for outlining various zones in the glacial drift. It has been found that the refraction seismic method not only complements the earth resistivity method, but also supplements it with unique data such as velocity information giving valuable insight into some engineering properties of the soil and rock.

The Department is developing applications of the refraction seismic method to highway engineering. Since most of Michigan's highway design and construction is in glacial drift, various applications of the refraction seismic method in unconsolidated glacial drifts are being investigated. Very little work is found to have been done in the past in this field.

The entire range of glacial sediments has been investigated. Good seismic anomalies and correlations have been obtained in materials displaying the extreme ends of the glacial sediments, such as soft organic sediments and partly indurated Pre-Wisconsin drift. Velocity studies in relation to construction and engineering properties also are being made in typical Wisconsin drift clays.

Three types of seismic soundings are being utilized -- the conventional reverse sounding, the profile sounding, and a modified arc or broadside sounding. In the conventional reverse sounding, the geophones are spaced horizontally at a distance of two to five times the depth being investigated. A shot is fired at each end of the spread and the data obtained are plotted as conventional time-distance curves. The time-distance curves are then interpreted, and the velocities and thicknesses of the various subsoil layers are computed. The conventional refraction seismic formulas have been programmed on the computer, thus eliminating considerable conventional calculations. The theory and mathematical treatment of this type of survey are well covered by textbooks (5).

It has been noted that certain types of bedrock and some glacial soils exhibit seismic velocity anisotropy. That is, the velocity of



Silty Clay

Wet to Saturated Sand

Sand

F — Soil sampled from borings fails to meet specifications for Porous Material Grade A Modified or Sand-Gravel Material.

A — Soil sampled from borings meets specifications for Porous Material Grade A Modified.

▽ — Depth of Rho Sounding

△ — Contact from Cumulative Curve

WL — Water Level in Drill Hole (October 1963)

MDU — Mobile Drilling Unit Boring

Figure 11. Comparison of Barnes Layer Method and Moore Cumulative Curve Method.

CROSS-SECTION FROM RESISTIVITY PROFILE-CONTOURS AND CUMULATIVE CURVES

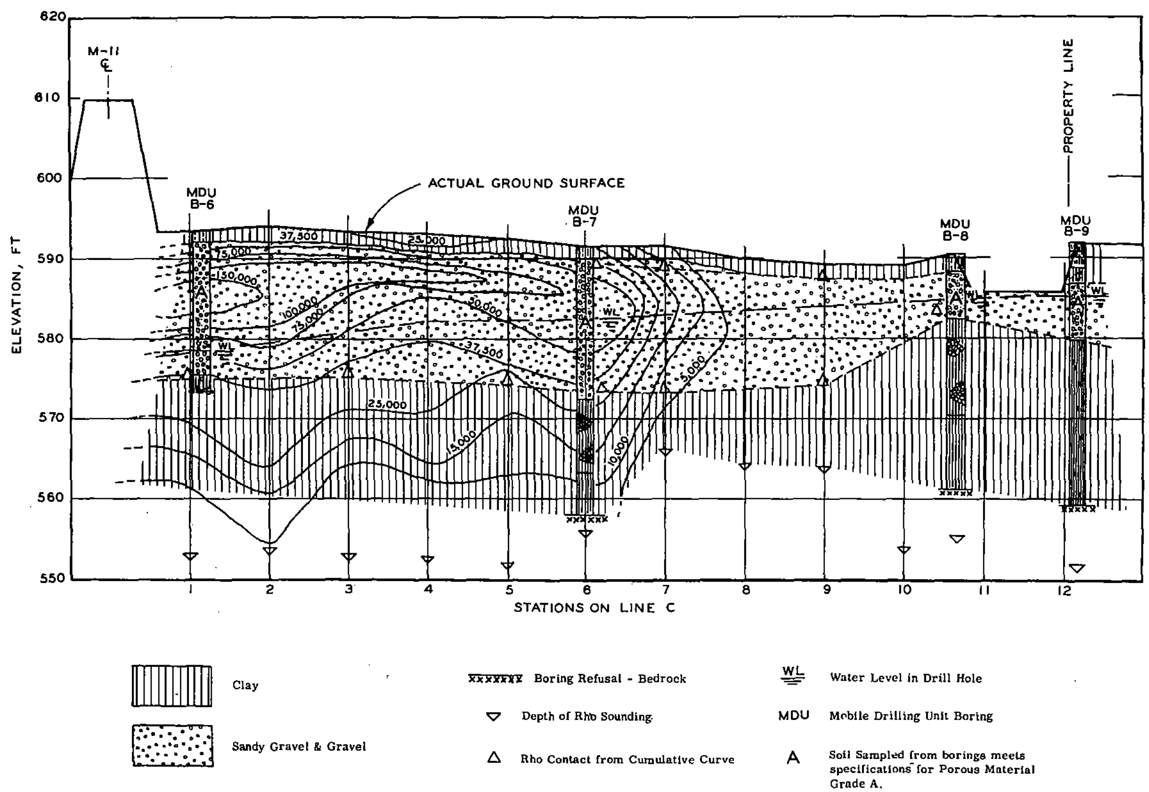


Figure 12. Effects of infiltration of electrolytes in ground water on Barnes layer values.

the material in one direction is different than its velocity in another direction. In some instances, this velocity difference has been significant enough to affect the results of the survey. It is now routine field practice to conduct an isotropy study of a given area and determine the better direction to run the geophone spreads in order to achieve optimum results.

The seismic profile method of subsurface exploration is based upon the time deviation patterns of actual recorded travel times from the estimated normal travel times associated with a bedrock refracting horizon. It has been used successfully as a reconnaissance tool for obtaining relative depths and surface configuration of bedrock. Several years ago, striking seismic profile delay patterns or anomalies were obtained over organic deposits. Studies were then conducted to develop profiling techniques and interpretations for application to highway engineering.

Organic soils yield large positive time anomalies. The magnitude of this positive time anomaly (delay time) will yield information as to the relative outline of the swamp. If the seismic velocities of the organic soils are known, an estimated profile of the swamp can be constructed.

Figure 13 shows the results of a seismic profile survey. The upper graph is the seismic profile delay curve. The heavy straight line serving as the abcissa of this graph is the apparent time-distance plot of a deep, high-velocity, refracting horizon and shows how the seismic record would appear if the delay anomaly were absent. The total seismic time delay anomaly is shown on the graph by the larger circled points.

The cross-section on northbound centerline (lower half, Fig. 13), shows the outline of the Iyopawa Swamp as determined by conventional methods. This swamp consists of muck underlaid by marly sedimentary peat. Average seismic velocity of these organic soils was measured at each station by pushing an electric seismic blasting cap to the bottom of the swamp and measuring the time it took the elastic wave to reach the surface. At some stations a measurement was also made at the bottom of the muck layer, permitting separate computation of seismic velocities for the marly sedimentary peat and the muck. These uphole measurements are quickly made with the portable MD-1 seismograph and a single geophone.

Seismic velocities varied considerably in each material, as shown on the cross-section. It is believed that these velocities may give some insight as to the nature and workability of the subsurface organic materials. The higher velocity zones would represent more compact materials while the lower velocity areas would represent softer, less compact materials.

Seismic uphole times are subtracted from the total time delay anomaly resulting in the plot of the smaller circled points shown on the seismic profile delay curve. If the organic materials outlined

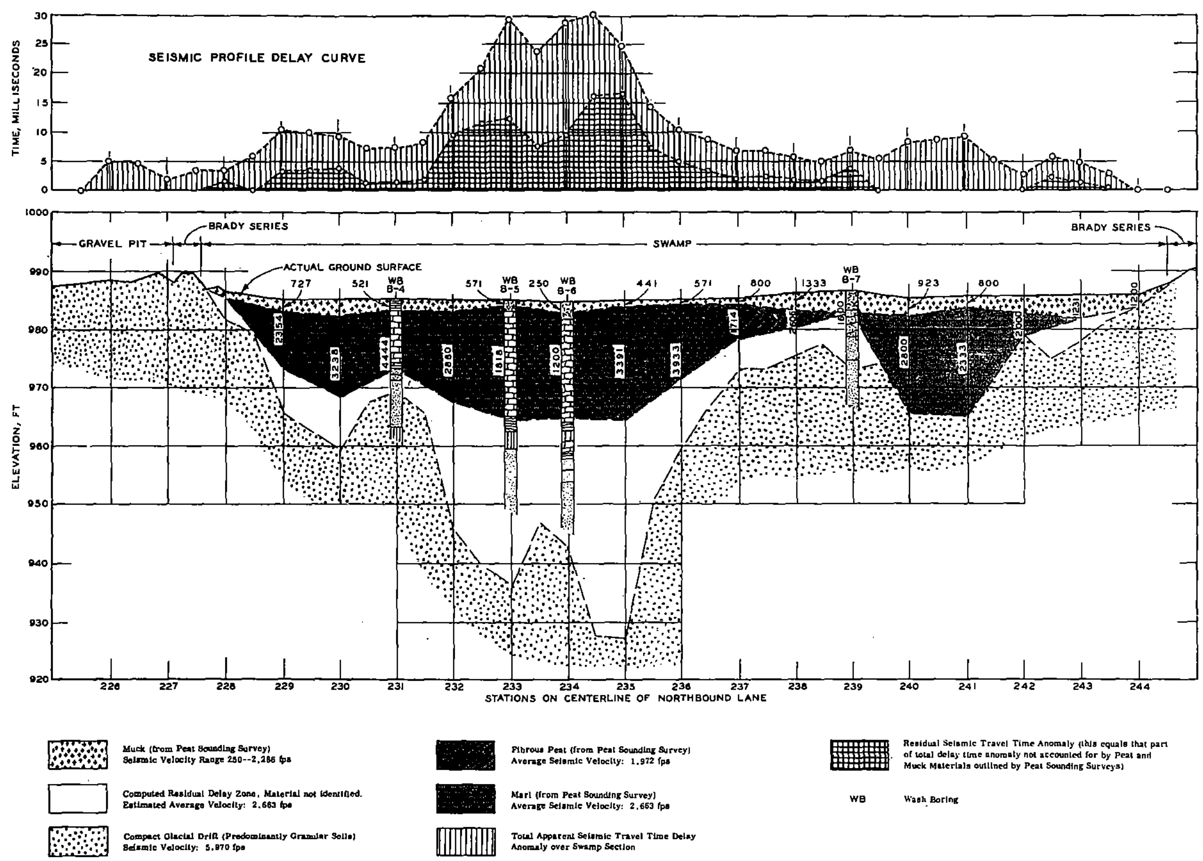


Figure 13. Seismic profile survey of Iyopawa swamp.

by the conventional survey have produced the entire seismic delay anomaly, the plot of the seismic uphole measurements will fall on or below the abcissa of the graph, such as at Stations 228+50, 242+00, and 244+50. However, if the plot does not fall on the abcissa, the time left over (residual time) represents additional low velocity material below the swamp.

By assuming that the underlying residual time layer has the same average seismic velocity as the overlying soil immediately above the residual layer, and knowing the length in milliseconds of the residual time anomaly, it is possible to compute and plot the outline and extent of the residual delay zone. This zone appears on the cross-section and is located immediately below the marly sedimentary peat zone outlined by the peat sounding survey.

It is believed that the outline of this residual delay zone represents the broad outline of a kettle or drainageway that has become filled. The actual materials are unknown, but are probably a combination of mineral and organic materials. However, the residual delay zone represents unknown material that should be checked by borings. The bottom shape of this zone gives information as to potential deep pockets and can be used as a guide for boring locations and depths.

Subsequent wash borings confirmed the previously mentioned seismic interpretations that the residual delay zone represented fill material in a glacial drainage channel or kettle. Although no buried organic zones of consequence were encountered in this particular swamp, basic assumptions and interpretations made pursuant to the seismic profiling method proved valid.

A second method of depicting potential buried organic areas is to run a series of parallel refraction seismic profile traverses across the area in question and plot the residual anomaly times as delay time contours. The delay time contours depict the structural outlines of the organic sediments. The individual contour values should be related to the thickness of these materials; therefore, the areas outlined by the higher contour values should contain thicker organic sediments than those areas outlined by the lower delay values. It is estimated that the areas lying outside the delay zones contain negligible organic sediments.

In Figure 14, the delay anomaly contours outline definite structural zones. A north-south channel runs across the area between Stations 149 and 150. The edge of an easterly trending channel or meander is located at Station 147 and between Stations 151 and 156, 50-ft left of the westbound lane. These areas could be potential trouble zones for slides if a large fill was placed along the westbound lane. Another channel or meander lies to the right of median centerline between Stations 155 and 157.

This type of survey can be conducted rapidly and the structural delay anomaly contours can be plotted immediately in the field. The

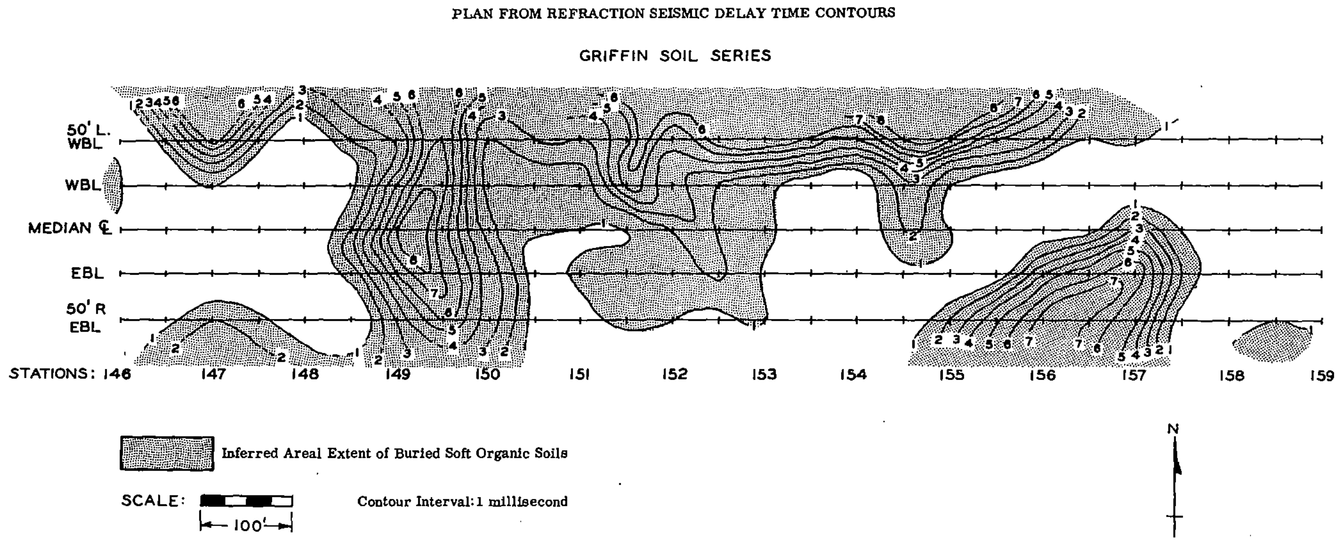


Figure 14. Subsurface structure from delay anomaly contours.

contour plotting yields considerable preliminary insight as to the structure, areal extent, and relative thicknesses of buried organic materials in large glacial drainageways. It can eliminate many areas as void of significant soft and organic sediments and point out these major zones where boring efforts can be concentrated to yield most information. As such, the seismic profile method should be used as reconnaissance information to guide the optimum locations of later, detailed borings.

FIELD SURVEY EQUIPMENT

Earth Resistivity Equipment

The Michigan Department of State Highways uses two types of resistivity instruments, each having individual advantages -- an alternating-current instrument and direct-current instrument (Fig. 15).

The alternating-current instrument is the Soiltest ER-2 Earth Resistivity Meter manufactured by their Geophysical Specialities Division in Wayzata, Minn. It is a light, rugged, accurate instrument, easy to use and rapid to operate. It requires no leveling or calibration and the scale is calibrated to read $2\pi R$. Its ease of handling and single reading per increment of depth make it a good production instrument. Its limitations are lack of power and penetration in dry materials, and limited scale reading in the high ranges.

The direct-current instrument is designed and assembled by William Keck Associates of Okemos, Mich. It is a rugged, precision piece of equipment. It requires calibration for each sounding and two readings per increment of depth. The scale is calibrated in resistance (ohms). Its principal disadvantage is its relative slowness of operation for production work. Its advantages are ability to operate under the more difficult resistivity field conditions and its great depth penetration.

It has become routine practice to provide both instruments to each resistivity field survey party. The geophysicist party chief uses the particular instrument he believes will achieve the best results.

Seismic Equipment

The Department has two seismographs -- a small engineering-type and a multi-trace type (Fig. 15).

The small type is the Model MD-1 Engineering Seismograph manufactured by Soiltest Ind., Geophysical Specialities Division, Wayzata, Minn. The instrument is well designed and built. It can time to the nearest 1/4 millisecond and recently an accessory quartz

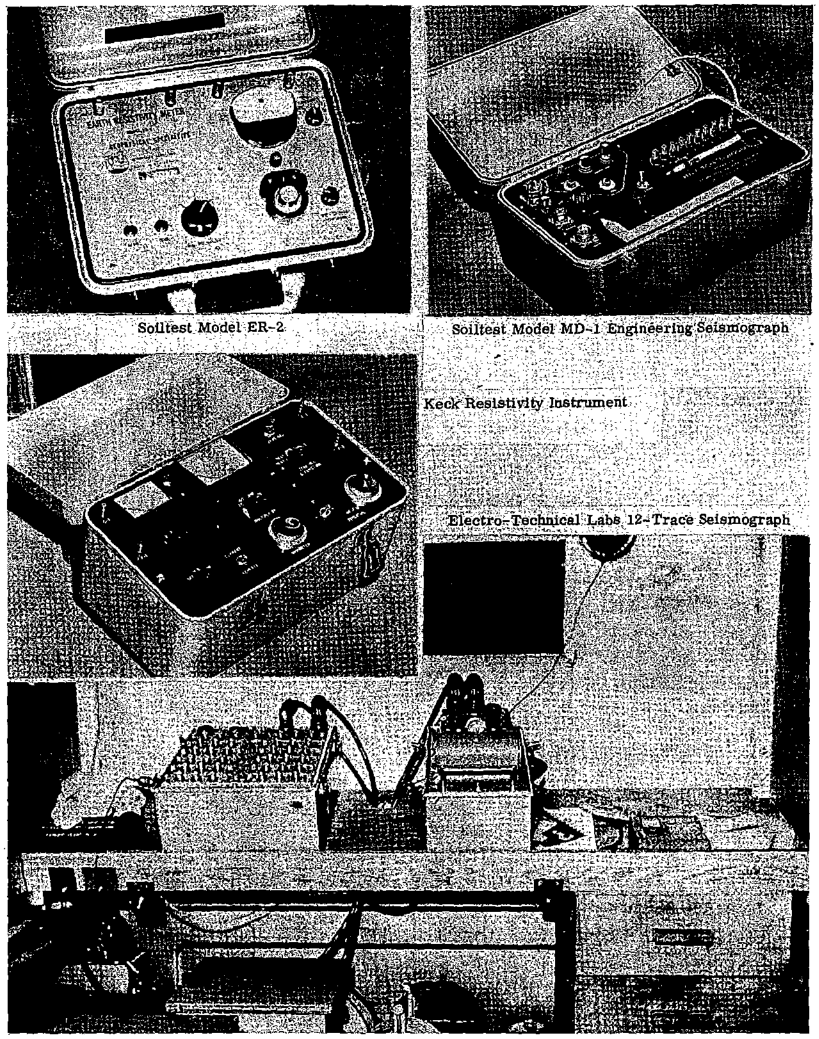


Figure 15. Resistivity and seismic equipment.

crystal has been made available, enabling it to time to the nearest 1/8 millisecond. The elastic wave for this instrument is generated in the ground by striking a steel plate with a sledge hammer, or by an electric blasting cap and primacord. A horizontal spread of 100 to 150 ft is generally optimum with this instrument; therefore, its depth potential is limited. The MD-1 is a good, dependable instrument when used within its limitations. The Department uses if for shallow exploration and for obtaining uphole velocities of various materials.

The multi-trace instrument is an Electro-Tech 12-Trace Seismograph manufactured by Electro-Technical Labs Inc. of Houston, Texas. It is a truck-mounted unit capable of doing any extensive refraction survey required by the Department. The elastic wave is generated in the ground by electric caps, primacord, canned ammonium nitrate, and occasionally dynamite. The seismic traces are recorded on photographic paper.

Boring Equipment

The key to accurate classification of subsoil and rock materials and to detailed geophysical interpretation is adequate boring control. The Department has a variety of boring equipment assigned throughout the state and available for correlation and sampling purposes.

The Geophysical Section is assigned a B-40 Explorer Auger manufactured by Mobile Drilling Inc. of Indianapolis, Ind. This rig (Fig. 16) is capable of drilling conventional continuous-flight auger borings, hollow-stem auger borings, wash borings with split spoon samples, and rock core borings. The boring unit's principal limitations are its size and weight which limit its use in rugged terrain.

For the more rugged and wooded terrain of the Upper Penisula, a Mobile B-40 Auger mounted on a D-4 Caterpillar Tractor is available. This rig uses 6-in. diam continuous-flight augers, and is capable of reaching almost any desired boring location.

Conventional wash boring equipment with trained crews also is available upon request. This is generally used for sampling and subsoil correlation in large underwater borrow areas. In areas not accessible to power equipment, hand auger borings are conducted and in some cases test pitting.

SURVEY REPORTS

Geographical Surveys of Roadway Cut Sections

It has become standard Departmental practice to conduct geophysical surveys on all new proposed roadway cut sections 12 ft or more in depth. Resistivity soundings are made on each station (100-ft interval) across the cut to a depth of 8 to 10 ft below proposed

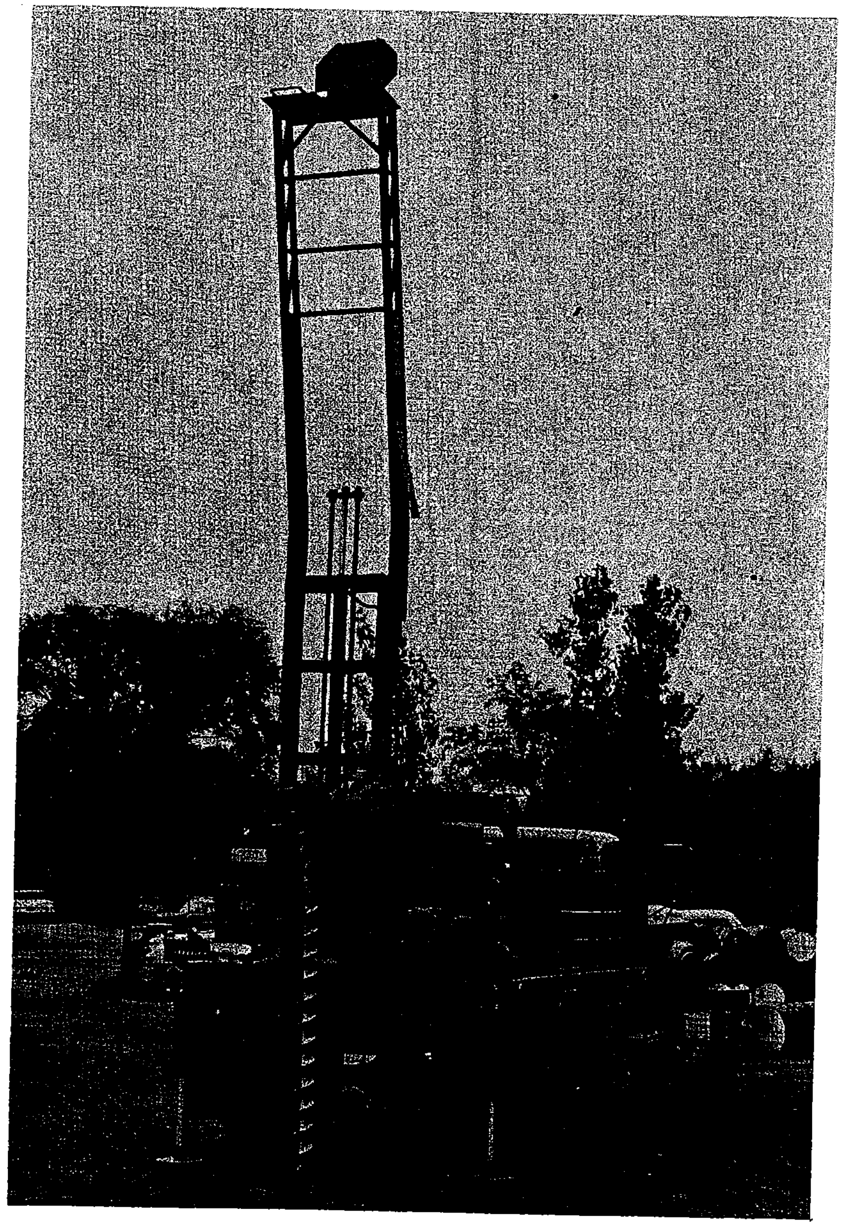


Figure 16. Mobile B-40 explorer auger.

grade. Depending upon the situation, a single traverse or a series of parallel resistivity traverses will be run. Seismic soundings will also be conducted if bedrock or Pre-Wisconsin till might be present. Correlation borings are made and representative soil samples collected for laboratory testing for classification and specification use.

The end product of the survey is a cross-section along the surveyed geophysical traverse depicting as accurate and detailed a picture of the subsoils as possible. Soil classifications are based on the correlation boring logs. Highway specification uses of the various materials are based on laboratory test results for the representative soil samples collected during boring operations. Other information included on the cross-sections are stationing, elevations, proposed grade location, correlation boring logs, water table, and laboratory specification test results.

The cross-sections, boring logs, and laboratory test reports are included in a formal written report covering the results of the survey and correlating the information for highway engineering use. These reports are sent to various Divisions throughout the Department.

Design personnel use them as an aid in laying grades, determining drainage and road base requirements, and as a general material inventory of the soil in the cut. The Soils Division uses them as a guide during construction and for borrow information. They are an aid in appraisal by the Right of Way Division. The reports are available to all bidding contractors and a complete set of survey reports for the particular project is furnished to the contractor awarded the bid.

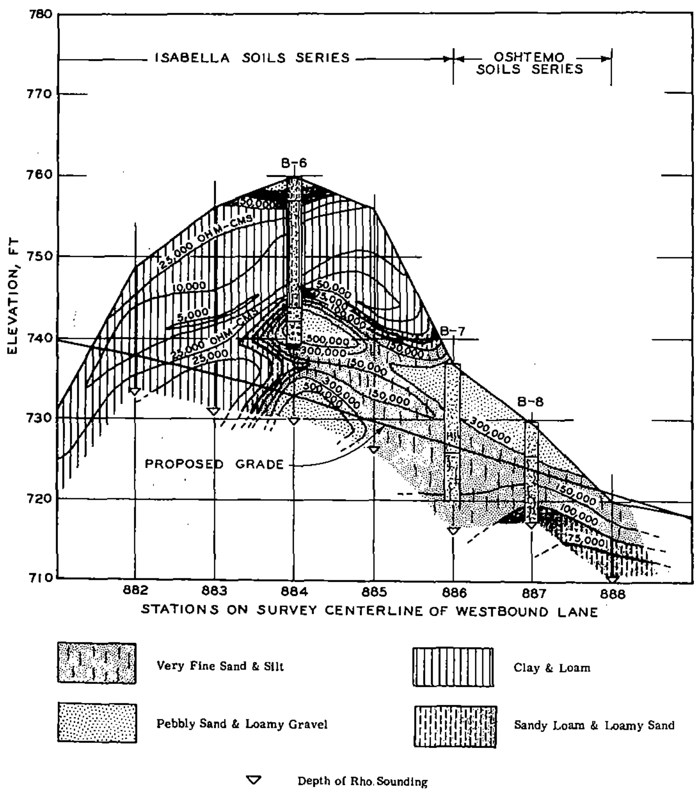
Figure 17 shows a cross-section from resistivity profile-contours on I 96 near Grand Rapids. The subsoils consist of clay interbedded with sand. The correlation between the predicted and actual subsoil conditions is evident. It should be noted that the vertical exaggeration on the cross-section sheet is 10 times.

Figure 18 shows a cross-section from resistivity profile-contours and seismic discontinuities from a cut section on US 131BR in Kalamazoo. The cross-section shows the presence of a buried marl deposit with ground water behind it. The picture shows the open cut. The marl extended about halfway up the backslope, but shows in the picture only at the bottom where it is wet. A considerable flow of water was anticipated and encountered. One water well was affected and had to be deepened. This situation could have been troublesome, but the advance information from the survey allowed it to be planned and designed prior to construction.

Geophysical Survey of Borrow Areas

Most of the Department's geophysical surveys are conducted on proposed borrow areas, which are locations adjacent or close to the

CROSS-SECTION FROM PROFILE-CONTOURS



Actual cut face.

STATION 884

Close-up of cut face.

Figure 17. Comparison of predicted and actual subsoil conditions in a roadway cut section.

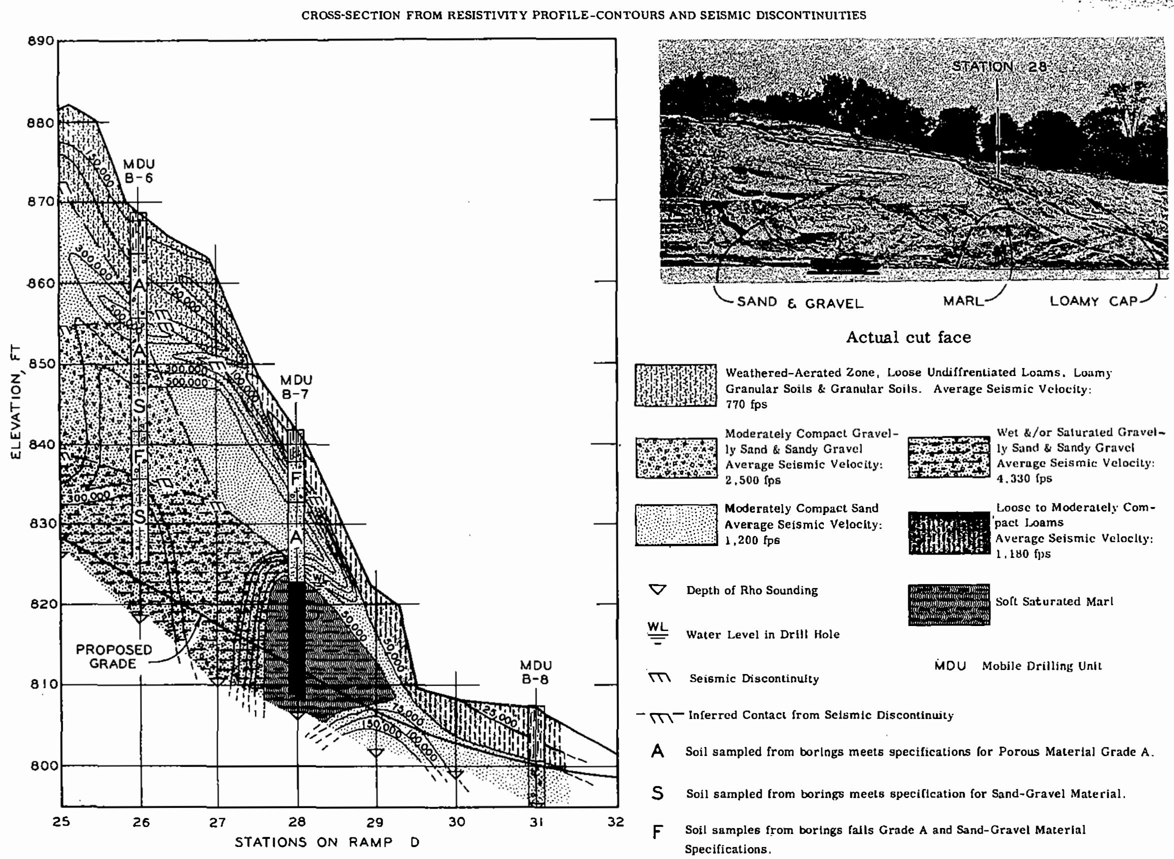


Figure 18. Comparison of predicted and actual subsoil conditions in a roadway cut section.

roadway location where specific materials are to be removed for use in building the road. These materials are often sand and gravel which must meet certain test specifications. They are generally used for specific purposes such as subbase, swamp backfill, bridge abutment backfill, etc.

It is Departmental practice to purchase necessary borrow areas along with the right-of-way. It is mandatory then that detailed subsoil knowledge of all the proposed borrow areas be available to the engineers before purchase. A large borrow area yielding non-specification material can upset the planning, continuity, and costs of a project.

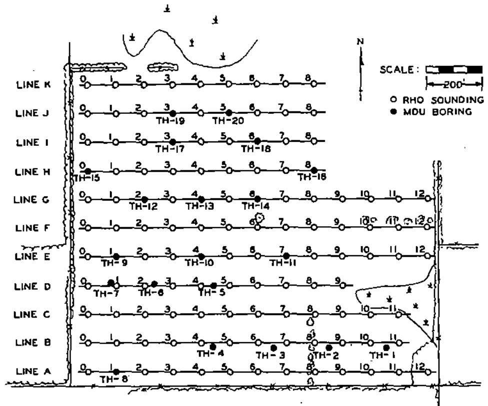
Throughout the years, general surveying techniques have been developed for surveying potential borrow areas. For large areas a series of parallel geophysical traverses are established. Stations are at 100-ft intervals and the traverses are usually 100-ft apart, resulting in a 100-ft grid of the area. Elevations are made, a proposed base of excavation established, and the area surveyed by resistivity with a sounding at each station. Seismic soundings will also be run if needed. Correlation borings are then made, with representative soil samples taken and submitted to the laboratory for testing.

After the various data have been assembled, cross-sections are drawn and interpretations made. Textural classes of the various subsoil materials are correlated with specification use, and volumes of these materials are computed by the average-end-area method. A formal report is prepared including a detailed written description of the subsoil condition, volumes of the different classes of materials encountered, a detailed log of field borings, copies of the laboratory test reports, and the cross-sections including a general location plan of the surveyed area. These reports are distributed to various Divisions for use in design, appraisal, planning, and construction.

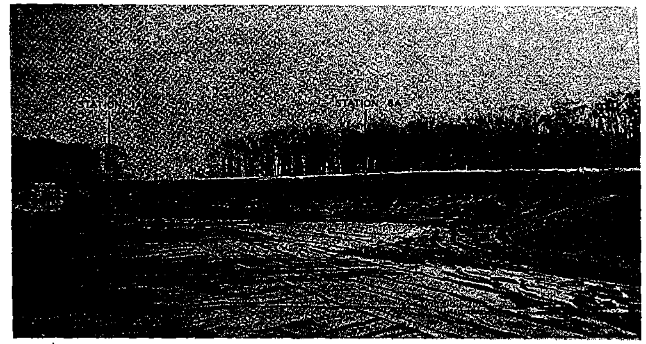
The proposed borrow areas can be divided into two major classes -- dry and underwater. The dry areas consist of various ice-contact features such as kames, eskers, crevasse fillings, outwash, etc. They are characterized by rapid vertical and horizontal changes in texture, and are generally surveyed by resistivity using the Barnes layer method.

Figure 19 shows a borrow area used on I 94 near Ann Arbor. This is a sand and gravel outwash or valley train overridden by ice which deposited a thick cap of till clay. The clay cap was stripped and used for embankment construction while the sand and gravel were excavated to the water table and used as subbase and bridge abutment backfill.

Underwater borrow areas are generally located in high water table areas such as glacial lake plains, spillways, and river flood plains. They are usually characterized by gradational changes in texture. These areas are generally surveyed by resistivity and



General location plan



Actual cut face along line A.

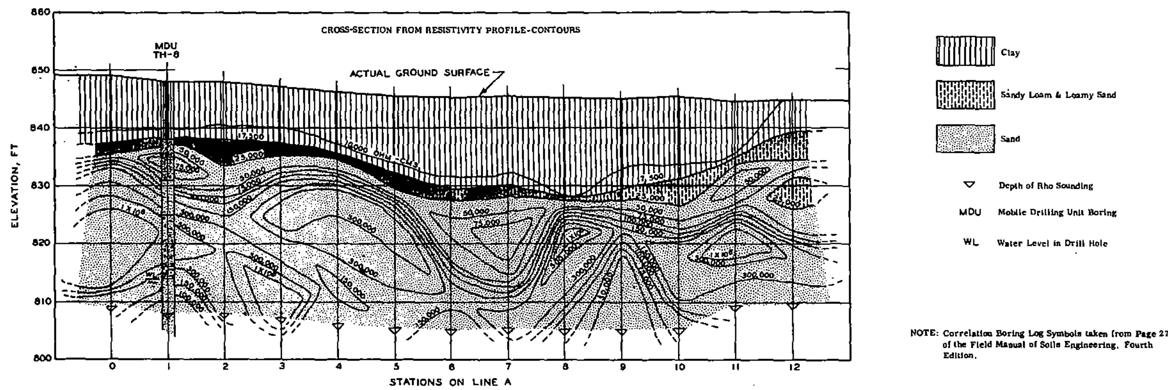


Figure 19. Location, photo, and cross-section along line A.

usually interpreted by the Moore cumulative method. Wash borings and split spoon samples are required for correlation and sampling.

SPECIAL SURVEYS

In addition to roadway cut section and borrow area surveys, occasional subsurface problems arise requiring the use of geophysics.

The main piers of the I 75 Rouge River Bridge in Detroit are set on four 54-ft diam caissons, sunk through 80 ft of clay, and founded and keyed into the limestone bedrock. It was found that this bedrock was glacially scoured, jointed, and probably faulted. It was vital to evaluate the top bed as a structural unit. Seismic soundings were conducted at the bottom of the caissons directly on top of the rock with the Minnetech MD-1 Seismograph. One sounding is shown in progress in Figure 20. The velocities obtained ranged between 11,500 and 15,000 fps, indicating competent rock. These unique velocity data gave mass measurements of the rock bed that could not be obtained from drill rock cores and visual observation of the rock surface.

When the right-of-way of a new roadway relocation passes over land containing commercial sand and gravel, appraisals must be made to determine the fair market price of the take. In cases involving valuable land and possible court litigations, geophysical surveys are often requested to determine the vertical and horizontal extent and volume of the deposit. These surveys sometimes are extensive and involve not only the part being taken by the Department but also the entire property. This is necessary because the appraisers have to know the total quantities of commercial materials before the take and the quantity left after the take. If the case goes to court, Geophysical Section personnel may testify as to the method of survey, validity of interpretations, and findings of the survey. The courts have accepted the survey results and many settlements have been concluded on the basis of the volumes and other information given.

CONCLUSION

The geophysical methods used by the Michigan Department of State Highways have been very successful. Each method of interpretation and piece of equipment, however, does have its limitations and the Geophysical Section is constantly striving to perfect and improve them. Equipment, field techniques, interpretation, and presentation of survey results are constantly undergoing modification. New equipment, methods of interpretation, and data treatment are continually being evaluated.

The field of shallow subsurface geophysical exploration for engineering purposes is a dynamic and ever-changing one. While much of the basic data reduction can be done routinely by data processing

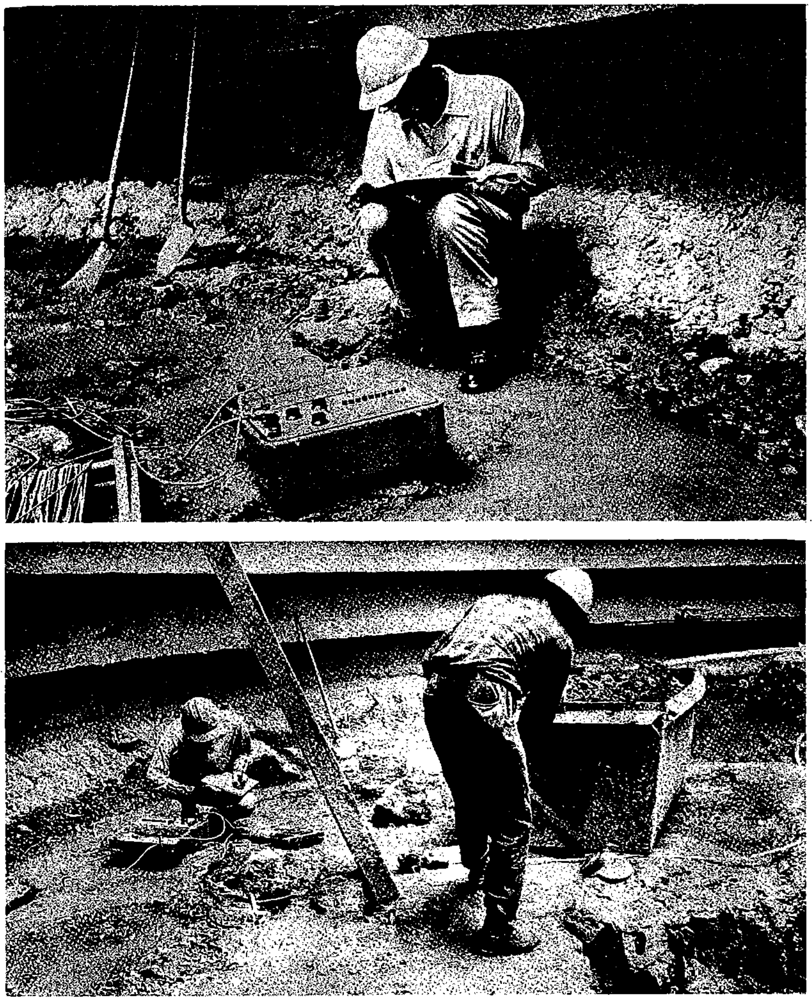


Figure 20. Plotting refraction seismic data of bedrock soundings (above), and conducting refraction seismic soundings (below) at bottom of Rouge River bridge caisson.

equipment, correlation and interpretation of results is still an art. As such, the success of a geophysical survey depends upon the experience, ability, and imagination of the personnel conducting the surveys and interpreting the results.

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Panel Discussion on Preliminary Exploration for Highways with Emphasis on Local Problems

PANEL MEMBERS:

Dr. C. W. Lovell, School of Civil Engineering, Moderator
Mr. David Novick, Westinhoff and Novick, Chicago
Mr. Gordon R. Benson, Illinois Division of Highways
Mr. Kenneth M. Miller, Ohio Department of Highways
Mr. C. F. Hotler, Indiana State Highway Commission

Dr. C. W. Lovell, Jr.: Gentlemen, if you can roster 45 minutes or 1 hour of fortitude, we will finish the final portion of this symposium which is a discussion period. I have noted that you have been a little restrained in your questioning throughout the formal papers. I certainly hope that during this period you will let yourselves go a little bit and ask questions and, if questions are asked which the panel doesn't seem to be handling too well, perhaps you can even help with the answers to the questions.

I am the moderator; my name is Bill Lovell, and I am here because I am available, I'm on the staff. The other members of the panel are here by virtue of their qualifications in this area. To my immediate right, the man nearest to me, is David Novick, who is a partner of Westenhoff & Novick, Chicago. The next gentleman is Kenneth M. Miller, Assistant Engineer, Pavement and Soils, Ohio Department of Highways, Columbus. The next gentleman is Gordon R. Benson, Chairman, State Soils Committee, Illinois Division of Highways, Springfield. The last gentleman on the end is Cal F. Hotler, Assistant Chief of Materials and Tests, Indiana State Highway Commission, Indianapolis.

We are continuing on this topic of applications and as you can see from the geographic distribution of the panel members, we are going to sort of zero in on the problems of subsurface investigations for highways in the States of Illinois, Indiana, and Ohio, and possibly somewhat adjacent areas. We hope that as things are said here which are intended as further input to this matter of applications, that you will respond with comments and questions.

I would like to start out by commenting on this matter of what one might do before he went out into the field and started "stomping around" as it was phrased this morning. We heard a number of suggestions then as to what might be done. Here at Purdue we got interested in this problem and we made a little study of it. We used as our criterion, the matter of economics. In other words, if there was something which should be done in the way of an office survey or study, preliminary to going out, making holes and taking samples, and so forth, it would have to be economically justified.

We considered a subsurface investigation of the standard variety, that is, a study of the things you normally would do but pretty well spelled out and yet not particularly peculiar to the piece of terrain on which you are working. They were just good solid rules for any piece of terrain. We considered the cost of subsurface investigation by this method. We said, let's approach it from a different angle. Let's consider that this is a unique job, it's like no other. It really should have a set of job specifications, that is, specifications that are drawn up specifically for this particular job and set up to gain insight into what one might do to fit the subsurface investigation to the peculiarities of that particular facility-environment combination. One would, of course, refer to existing airphoto coverage, the normal black and white coverage. Then with a little experience and perhaps, something which I haven't mentioned before, a little L. S. D., you might be able to do a very good job of interpreting these photographs as far as soil distribution is concerned. Then, also, review some readily available geologic report for the area, and from this sort of background information lay out a subsurface investigation that seems uniquely fitted to the particular job at hand.

Next assume that the same operations were to be carried out using each type of approach; for the standardized approach and for the job-oriented or job-designed approach. The same unit cost would apply. Figure the cost to see if when you design the subsurface investigation for a particular job, whether it costs more, costs the same, or costs less, than if it followed a standardized procedure.

I think the results were encouraging in that it was found that for a variety of topography, geology, and soil textures, that it was in no case more expensive to do it in the latter, job oriented way, that is to make reference to nominal airphoto coverage, to make a study of prominent geologic features, and also to utilize pedologic information if that information were available. The reduction in cost was of the order of 10 to 20 per cent, as opposed to the standardized approach. A minimal saving was accomplished where the topography was rough, when the geology was complex, and where the soils were inherently unfavorable. This means that one couldn't really hope to save very much money by the office survey method in these situations because of the inherent complexities of the job.

Let me next call on Mr. Benson to make some further comments, and if you do have questions or comments of your own at any time, please stick up your hand, identify yourself, and we will be very glad to give you the floor ... Mr. Benson.

Mr. Gordon Benson: We, of the Illinois Division of Highways, recognized some years ago, that the staff was going to have to be supplemented by consulting engineers in order to accomplish the amount of work that was at hand. Until that time we had had no definite format established as a guideline for making a soil survey. So we sat down, so to speak, to try and delinate the things which we were going to put together as guidelines for these consulting engineers who would be working with us. Consequently we came up with certain specifications which I would like to lean on a little bit here.

When we were developing these specifications we recognized in a hurry that if we were talking about something as big as the State of Illinois, and something as diverse as it is in geology, that it is pretty difficult to put on paper or to draw up a specification which is always completely sensible. We tried to establish a minimum set of rules which would be supplemented, as necessary, by the men in the field, which would give a reasonable and, we hope, thorough picture of the soil conditions as they existed.

In the cut sections we asked that the borings be spaced along the center line of survey of a single lane pavement at 100 foot intervals, and be about 6 feet below the top of the subgrade. If it is going to be a divided improvement, such as our interstate program has, we space the holes 100 ft. apart but in alternate pavements. In other words, under each alternate pavement they would be 200 feet apart, but staggered 100 feet right or 100 feet left off the center line of survey. If, however, they were spread more than 100 feet apart, between the two center lines, it would require a full soil survey on one of those pavements. For unusual conditions, if our terrain was extremely rough so that the pavement grades or the ground lines might vary substantially, we would want two complete soil surveys run in.

In all other locations other than cuts, we space the holes out to 300 feet apart and take them down to 6 feet below the subgrade or 2/3 of the height of the proposed embankment. I got quite a kick out of it when Cal Hotler tried to figure this out during his paper before because he came to the same conclusion as we did right here. It shows that great minds run kind of alike.

We have since had to temper this a little bit recognizing that we have problems that arise involving stability and settlement. Let's see if I can find some more of our remarks on slope stability. We require that in areas where roadway embankments greater than 15 feet in height are to be constructed, the foundation borings shall penetrate the subsoil to a minimum depth equal to 2/3 the height of fill of the proposed embankment, or to bedrock, which ever is encountered first. If necessary the boring shall be advanced to a greater depth so as to terminate in at least 5 feet of material having minimal compressive strength value of 1 ton per sq. ft. or encounter 5 feet of granular material having a standard penetration test value of 12 or greater. We are running these stability analyses that we talked about using the results of our standard penetration spoon samples, and recognizing that these are somewhat disturbed, we ask that if the consultant who was working with these comes up with a safety factor that is less than 1. 75, that they contact the district engineer. He will then either ask them to go further with their sampling with a more detailed type of refined sampling or else go out and do it ourselves.

Personnel wise, as far as our soil surveys go, we have a staff in Illinois of perhaps 50 engineers and geologists, counting those in the Bureau, in the 10 districts that are involved all the time on soils investigation. Our equipment -- we have 10 B52 level drills, we have a Joy 22HD, which we purchased back in 1955. We have one B61 Mobile drill, have one CME55. For the shallower work for example, our shallow line borings, we have at least 10 auger-type units such as B27's or B40's or Acker Hill Billies, and we have one little trailer-mounted Concor unit. We manage to keep this equipment busy all the time plus actually hiring quite a staff of consultants to help us out.

Lovell: Questions or comments, Gentlemen?

Audience: Are the people in Illinois still coring through their hollow-stemmed drilling rigs as they did at one time? Coring through hollow-stemmed augers, the flight augers used for boring, I believe.

Benson: We just use the hollow stem as a casing. Yes, that's the B61 that we bought. We acquired the larger sized augers so that we can work with NX tools through the auger and also can take some 3 inch diameter Shelby Tube samples.

Audience: Are there any problems using this auger or from getting samples with it?

Benson: No. The only problem that we have is that in the core sometimes it is a little difficult to seal the bottom of the auger off against the rock and you tend to lose some water down at the bottom and this can be a problem.

Lovell: Would you stand up and identify yourself, please, when you ask the question? Thank you.

Woody McGraw, Kentucky Department of Highways: Have you considered the possibility of the stress distribution below the fills depending on the width of the fills rather than the height when you mentioned this 2/3 H factor which you stated was arbitrary? I think you were talking about the stress distribution before and assumed you were thinking of the matter of settlement under a fill and the influence the weight of the fill has on the foundation that it is sitting on. Would the width of the fill be more of a factor than the height for consolidation? For stability problems?

Benson: Yes, we do. I'll give you an example -- a consideration of a soil problem that we faced with a 30 foot high embankment at Peoria, or just south of Peoria where Interstate 474 will cross over the Illinois River. We are out in the alluvial flood plain and in making some foundation borings for the abutments and some of the approach piers, we got out of granular material and into quite a soft clay with moisture contents up around 30, 40 and 50 per cent. I think our strengths were about 1/2 ton per square foot where we ran into that material at a depth of about 75 feet and didn't go out of it until we got down to 125 feet where we encountered bedrock. Because of that situation we elected in that area to extend the bridge because we knew there would be a long-term settlement. There was no way to handle it, and where we can, we put our structures on steel H beams so some of our fill borings go pretty deep.

Lovell: It seems to me that both the dimensions are pertinent, both the width and the height, and if one attempts to calculate the stress distribution in making an embankment he might very well choose a parameter of half the width over the height as a parameter in this determination. So I think both points are well taken, but the width and the height are important.

Lovell: Yes, Sir:

Burrel Witlow: I'd like to make a comment on this, on the settlement of fills. This has to do with inquirying of other engineers as to what problems might be encountered in an area. I can think of an example. We had an office in Norfolk, Virginia. We were looking at a crossing by a railroad across a swamp for about 500 feet. They had a box culvert as the only drainage construction there and apparently they had fills out, approach fills on each side. We immediately checked with the railroad personnel as to how much difficulty or trouble they had with this area and they remarked on several occasions that they had had no trouble. Our calculations showed consistently that we would get about 4 or 5 feet of settlement in about a 20 foot fill. Finally, after the project was about ready to start, we checked with a fellow who had worked for the railroad in this area. He said, "I don't know what you mean by trouble but we do have a little trouble -- we jack the railroad up about a foot and a half every year. Nothing other than that; you know, we built that on a trestle all the way across there; when the top timbers rotted off at the water line we cut them off two feet below the water and built a platform, and that railroad is sitting on a platform with fill on top of it." He said, "I think they had about a 65 foot pile under there that no one knew about."

Benson: I might add one little item which was brought out there, the item of box culverts under embankments where we are going to get a lot of settlement. We fought with that baby a long time and primarily, I would say, due to the results of listening to the proceedings given by one of the members at our meeting down at Texas A & M, a fellow from Missouri, we got a solution. For a couple of years we have adopted the idea of a segmented box-culvert which we put in camber in short sections so that it could take these suspected large settlements. In some of the areas we have to scratch our head a little bit, in real flat country such as Ron Berliant was talking about this morning, where the inflow and outflow ends are going to be about the same elevation. You don't want to pond water too much. Actually we are building some boxes overwidth and if we are talking about 24 inches of settlement, we'll stick our tongue in our cheeks and camber for maybe 18 inches and figure -- well, let it have a belly in it like this. We will put the extra width in that way. I think this is one answer, rather than putting it on piles because limited skin friction gets to be a pretty big item there.

Lovell: All right. Mr. Novick, will you give us some of your ideas, please?

Mr. David Novick: Yes, thank you, Bill. I'd like to talk a little further about Interstate Route 255 in the East St. Louis area that Al Sanborn and Ron Berliant talked about earlier this morning. This was a project where we started with the specifications that Gordon Benson referred to as a base and then modified them rather drastically because of the special nature of this project. We first were involved in I 255 in the preparation of the route relocation report. In this capacity we were functioning as highway design engineers, rather than soil engineers. We recognized early that there would be major soils problems. We recommended as part of the route location study that a rather elaborate program of investigations be undertaken for the final design.

I know, throughout my career, I have always grumbled to myself and my associates about the confining effect of having to work within a standard specification where the problems are special. Here we had an opportunity to introduce many new soil mechanics concepts and to it at the very start of the job. We were pleased that the Illinois Division of Highways generally accepted these recommendations and added their own suggestions. Jointly we developed this program. One of the special features was the introduction of pilot borings at all bridges at a very early stage in the design.

I am sure many of you have had this experience of finding a problem at a bridge site and then being advised that it is too late to do anything about it because the bridge plans are 75 per cent complete and this doesn't make it too popular with the bridge engineers. It is usually difficult to get these changes made. So we provided these pilot borings in the roadway exploration stage and this gave us some information on the bridge problem very early. As a result, we were able to make recommendations for flattening end span slopes at the structures prior to the submission of these bridges for approval so that the bridges could then be approved reflecting the soils problem that existed at each site.

Another very special problem in this project was the existence of buried organic deposits and these deposits made it very difficult to judge where the swamp conditions existed at an early stage of the project or by surfical means. You couldn't tell from walking reconnaissance. You could hardly tell from aerial maps. You could only tell by intuition, and then drilling relatively deep holes. We found many deep and thick organic deposits, sometimes below 20 or 30 feet of relatively firm soil.

The varied deposits had another intriguing aspect to them. Since they were so deep, it was virtually impossible to consider excavation of them. You could hardly justify excavation of 20 feet of good soil to get to organic material below. So this alternative was usually ruled out and the sand-drain alternative appeared to be most economical in a number of cases. The sites were especially conducive to sand-drains because there was no real stability problem. The upper firm soil prevented the probability of any displacement-type failures and the problem became one of accelerating settlement.

Since the fills were high (it was mentioned this morning, 30 to 40 feet over much of the site) the settlement problem was a big one. By anticipating this kind of problem at an early stage in the design, we feel that you can fully take advantage of soil mechanics studies and get these conclusions utilitized in the design. You can also resort to such techniques as stage construction wherein you build portions of the embankment ahead of other proportions, and utilize time as a soil mechanics device. There is really no reason why you can't do it -- build a portion of the embankment a full year ahead of the rest of the highway and thereby gain this valuable time for preloading. Similarly at bridge sites where there is a problem, those bridges can be taken out of the normal staging process and built ahead of the others and in this way also gain valuable preloading time.

Lovell: I am a little bit puzzled about the use of sand-drains and organic materials. It was my impression that most of the settlement in organic materials came from secondary effects. Of course, sand-drains accelerate the primary, of which there is little. I was also of the impression that permeability in the organic material was sufficiently high. There was no very substantial reason to worry about accelerating the primary. Apparently I am a little off base on this and would appreciate being enlightened.

Novick: Well, I wouldn't say that you are off base, Bill. I would agree exactly with what you have said for materials that you would call peat. We have had similar experiences. The materials we are referring to are not in the peat category but what we might call organic silts or organic clays. We found that by loading our laboratory consolidation specimens in very small increments, we can develop consolidation curves which approach the conventional for inorganic soils and we suspect that these organic materials are loaded in the laboratory conventionally, (what I mean by conventionally is in double load increments) that this process may destroy the structure of the soil and in this way change the properties through disturbance. So we find that before performing the laboratory tests in very small load increments, we can develop both pressure void ratio curves and time curves which resemble the conventional theory of consolidation. And for such soils, we believe the sand-drain method as applicable.

Lovell: Are there other questions or comments?

Mr. Miller, would you like to give us your contribution, please.

Mr. Kenneth Miller: Well, I have been listening to these gentlemen from Illinois and their problems are much similar to those we encounter in Ohio. In Ohio, we do predominately all of our own investigating, soil boring investigations, as well as testing. We are set up with, briefly, -- the equipment that we have consists of 8 truck-mounted augers, 2 truck-mounted core drills, and 15 trailer or skid-mounted core drills. Then we employ the drive-rod penetration units. We have 14 of these units. It was mentioned this morning by Mr. Sanborn, I believe, that they use a similar device. I don't know if this is their terminology or what they use, but we have been using the equipment for a number of years. In fact, it dates back before the second World War and we have compiled a lot of empirical data since this time. We have made correlations with drive-rod soundings, and Split Spoon standard penetration tests. The rest of the rolling stock we operate and maintain includes 12 water trucks, 19 utility trucks, and possibly 5 bulldozers.

In Ohio when we investigate a new roadway, it is our general practice to investigate the structure sites first. In this way we have our structure problems solved before we get into our real major or any other soil recommendations that we might make.

We also have the specifications for subsurface investigations similar to what was described earlier. Mr. Hotler's comment in his paper this afternoon regarding soil profile information generally is the same as what we follow. Our boring locations are spaced 300 to 500 feet apart instead of 1000 feet apart. We investigate each boring. We take a sample representative of that layer and we test each sample that we obtain. So we do have quite a bit more information regarding gradation, liquid limits, and physical characteristics.

We also use special investigations where we encounter peat bogs and known landslide areas. We take undisturbed samples of all our soil foundation areas and employ the things here mentioned with certain limitations. We make all of our own slope recommendations, and recently we have employed geologists to run the geophysical equipment that we obtained. This consists of the updating of our old resistivity outfit and acquisition of a completely new seismic refraction apparatus. We employ these geophysical techniques, especially in landslide areas, to determine the failure zone of the slipped area and soft soil foundation areas to determine the depth of the peat desposits.

Ron Berliant: You say you investigate structural sites first? Prior to roadway investigation in your highway work?

Miller: Generally, yes. This is in the boring program.

Berliant: If you find any problems in the approach to a bridge column, would you try to solve this problem prior to getting additional information pertaining to adjacent roadway?

Miller: No, we would try to obtain as much information as possible, if we encounter this poor foundation material. In other words, our bridge investigation consist of generally ten core borings and drive-rod soundings at each pier location. That may be slightly misstated there. Two core borings for the structure, and then generally about eight, one on each side of the pier. We take a rod sounding. Now if we encounter poor foundation material we can go back and schedule a boring program and sampling program to determine its extent. This is very useful if we can obtain this information while we are getting the soil profile information.

Berliant: What I am driving at is the treatment for the bridge column. Might it be the same treatment that you might want to use on the normal thickness of the embankment? Therefore, you could do it simultaneously rather than try to treat the bridge column differently from the other.

Miller: That's right.

Lovell: Of course the cuts, large cuts, and major embankments could also be considered as structures that could be preinvestigated along with other structures.

Miller: This is true.

Lovell: Can you still do an excellent job in Ohio of accumulating and correlating your data so that you can do almost anything by one point determination?

Miller: Well, the Bureau has brought this up many times. We still use the one point determination. However, if a consultant is performing the work, they generally will run 5 point determination against a compaction curve. They will run 3 or 5 point liquid limit tests, too.

Lovell: It is interesting that the 3 adjacent states, Illinois, Indiana, and Ohio have about the same standard specification for subsurface investigation. It is entirely possible that if we were to investigate adjacent states that we would find further extension. There would seem to be some logic in Ohio, Indiana, and Illinois, being rather similar in their approach, if they would be willing to standardize an approach because of the similarity in the environment in which they are working, ground conditions, and the climate.

Lovell: Mr. Hotler, would you give us a few words, please?

Hotler: It limits me, doesn't it? Few words?

Lovell: Not really.

Hotler: It sounds like, Gordon, that you got forced into something about like Indiana did. Back in the old days, the size of our soils department kind of varied depending upon the amount of work that was being done, and also depending on the number of qualified personnel that we could hire. We probably had anywhere from one to four soil survey parties working at various times. Now this is back in the pre-World War II days, and even after World War II, it varied, all depending upon the amount of money available for construction. Then this interstate program came along and at that time we were extremely shorthanded; had very few personnel. We did have one soil survey party in the field all the time. But along about 1962, if the date is correct, I think it was about that time -- early in 1962, the Bureau of Public Roads said we must have complete roadway soil surveys on all Interstate projects. We were faced with a situation of not having sufficient people to do the work, not having any equipment at all to do the work but yet we had to do it. So the only choice we had was to have it done by consultants. Well, we couldn't just turn the consultants loose without some guidance. We hurriedly prepared specifications to guide these consultants. We did, I think, look at what Ohio used and Kentucky used, by way of specifications. Probably theirs at that time weren't as elaborate as they are now. We had to put together something for a boring interval. So we said let's have borings every 300 feet on center line if this were a two lane or 24 foot pavement; if we had divided highways, we had them every 300 feet but staggered from one center line to the other, much the same as previously described.

When you are trying to set up guide lines for someone to follow, it is hard to get across without setting up definite limits. When it is in your own mind, you know that you want, and you can exercise your own judgment, but to get your judgment transmitted to someone else's mind so that you are thinking alike, is awfully hard to do. This is where we had to come up with a definite set of rules for making these borings. This is, I believe, the project that Bill Lovell had mentioned earlier in his discussion in making comparisons in cost of soil surveys in the conventional manner as opposed to planning and quite a bit of office work. Now since that time, we brought in a little bit more personnel where we can do a little more of our office work and checking and engineering, and that sort of thing. We have revised our specifications, somewhat, in the manner in which I mentioned earlier in the afternoon.

Just so nobody is mislead, Ken, we do make borings. We like to get one in each cut and each fill as a minimum. If we have long cuts, and long fills, I mean station-wise, then we put additional borings in the cuts and fills at about anywhere from 500 to 800 or maybe even 1000 feet, depending upon the terrain of the particular areas we are in.

Now we do get into a number of problems, (and I think we saw some of the problems yesterday on the field trip) when we find these peat, marl, and muck deposits. If the deposits aren't too deep, hand borings are pretty adequate but you can't go very deep with hand-borings in peats. We have a hand sampler, a piston, a plug-type sampler, which is adequate for shallow depths, too. But if the peat and marls are deep, then this requires machine boring. There's where we get into a problem in swamp areas, how do we get a machine in there? Somebody, Henry Mathis, I believe, from Kentucky, was wanting to know what we did in cases like that. You do the best you can. If you can get a machine in, you do; if you can't, you use a skid-rig; and if you can't do that, you are forced to do what you can by hand.

Now, as an example, of course, what we try to do is to determine the depths and the areal extent of the thing so that the designers can come up with some sort of quantities for removal. As an example, what happens when we don't treat these peat deposits in some manner? For those of you who were on the field trip yesterday, as we passed through Fowler, you may recall that considerable dip in the pavement. I called attention to it for the people in bus #2. This was a deposit of peat and marl some 12 to 15 feet deep with an embankment of approximately 20 feet. We knew it was there but the design progressed and we weren't aware what was going on, so it didn't get taken out and you saw what the result was. We have a flexible pavement there -- concrete bends pretty well. It is in good condition; it has some normal transverse cracking. I haven't looked at it recently but I don't think that it is in very bad condition at all.

Somebody had asked the project engineer about it several years after it was constructed and the dip had developed. It wasn't one of the local people around Fowler. He asked what was wrong and the engineer said, "Well, we built it that way intentionally to slow the traffic down coming through Fowler." Now, I don't know if that's a true story or not.

Then other problems we get into are these saturated soils with subdrains that I talked a little bit about also. If we can locate those during our soil survey, well and good. We will make some kind of recommendations as to what should be done. We can either raise the grade in many cases and help the situation, or we might want to lower the grade to get under this material if we can be sure it is solid underneath within a reasonable depth. We might increase the thickness of the subbase or any number of combinations.

The trouble is, we don't locate very many of them during our soil surveys because of our intervals of boring. If we hit them, we're lucky. So, as a result, most of them come up during construction and then we get a fire department call to come up there to make an investigation and tell them what to do. Well, actually this is beneficial because it is a lot easier to make a complete investigation at that time because the cuts are down to where you don't have to drill nearly as deep and we can get better information.

Now just a word about problems with respect to settlement and foundation stability. We hit these problems, too. I think they are common to most anybody that's building roads. Whenever we find something like this, a suspected condition in a soil survey, we do take undisturbed samples and we do have consolidation testing and strength testing. We determine whether there is or is not a problem. If there is a problem, we can set out to solve it. So based on this information, recommendations will be made. It might be a situation, if it is not very deep, that we can take the compressible material out; or maybe if it is pretty deep, and it is not economical to take it all out, maybe we can take part of it out and partially solve the problem. We might surcharge, as Dave Novick mentioned on some of his work; surcharge to accelerate consolidation.

If it is a stability problem, rather than a settlement problem, we might counterload or put berms up, counter-loading berms, in order to insure stability. We might specify in such special provisions that certain time intervals will elapse after the time of the construction with instrumentation and determine how fast the construction can proceed, or any combination of these things.

I recall a couple of interesting examples of where we have measured some settlements. We had one project involving a bridge approach and a bridge over a stream founded on about 22 feet of medium-soft stiff, silty-clay loam and another 23 feet of soft to medium-stiff silty clay. The height of fill was going to be 22 feet and from consolidation testing the settlement was predicted to be anywhere from between one to two feet, there was quite a bit of variability between samples -- so this is how we arrived at one or two feet. So it was specified that a settlement plate would be installed prior to the construction of the embankment and then the readings made continually as the embankment was placed until such time as the settlements had subsided. So in this particular project, the settlement plate was installed on May 13, 1965, and the embankment construction was started. The embankment was completed rather quickly, by June 4, which would be about a couple of weeks, in which the 22 foot embank embankment was placed. Now this, bear in mind, was the highest part of the embankment and where we put the settlement plate. At that time, total settlement which had been measured was 7.7 inches. Four and a half months later, the settlement was 12.9 inches. In the next four and a half months it settled an additional 0.6 of an inch and gave it a total of 13-1/2 inches. Now this was along in the spring of 1966 when those measurements were taken, so it appeared from the time settlement curve that had been plotted, that about all the settlement was out of it that we were going to get. It was now progressing at a real slow rate, so the thing was paved in the summer of 1966. I drove over the project in December and it seemed really smooth and I didn't believe there had been any additional settlement.

This gives you an example where we had predicted a settlement of one or two feet and we got a little over a foot, or 13-1/2 inches. Now, no doubt, there will be some additional settlement but I don't think it is going to be as noticeable as if we had gone ahead and paved that thing after they got the embankment completed on June 4. If they had paved that within the next week or so, they would have had another 5 to 6 inches of settlement and they'd have had problems.

We have one other project that is under construction now. This one has about 15 feet of soft to medium-stiff silty clay loam and an additional 35 feet of variable silty clay loam and silty clay. The maximum height of fill on this project will be 33 feet. Some 2 to 3 feet of settlement has been predicted. Here again, variabilities in the underlying materials made it difficult to say exactly or to pin it down really close. The prediction was that it would take about 30 years to get 75% of this consolidation so it doesn't do much good to surcharge. When you have that long a period of time, surcharge wouldn't increase consolidation rapidly enough so it was decided that we would put in settlement plates in this one. Seven settlement plates were installed. We only specified two, but somehow or other special provisions came up and we put in a lot more of them. At any rate, the embankment was started July 29, 1966. They got the embankment up in about 2-1/2 months and they completed it October 10, and at that time the deepest part of the fill at the maximum settlement was 7.8 inches after we had the fill up. As of March 27, of this year, or 5-1/2 months after completion of the embankment, we have a total of 12.02 inches. The rate of settlement has been fairly uniform since the completion of the embankment, ranging from anywhere from 0.1 to 0.3 of an inch per week. And the time settlement curves have not indicated at this point that we are getting much flattening out so we don't know just how much longer that thing is going to have to sit. The special provisions had specified a minimum of 9 months waiting time and we have had 5-1/2 months already since the completion of the embankment.

These are just a few examples of how we apply predicted information or information developed from our soil surveys and recommendations and go out and try to see what happens. We are going to try to do a lot more of this and I think this will give us a better insight as to how predictions come out ... That's all I have, Bill.

Lovell: Do we have any additional discussion from the audience? Yes, sir?

Henry Mathis, Kentucky Department of Highways: I would like to direct my question to the panel or to the audience here who has had some experience in setting slopes in design. What criteria do you follow as far as the height of lift and width of bench, etc. ?

Lovell: Who will take that?

Hotler: You mean the cut slopes?

Mathis: Yes, the cut slopes -- rock cut slopes.

Hotler: Well, as far as Indiana is concerned the standard design section for rock slopes show a variable slope anywhere from 1/2 to 3/4 on 1 or maybe 1 on 1, or whatever is decided how the rock might stand. Of course, the harder rock, we feel, will stand on steeper slopes, and the softer materials, such as the shales, won't stand on these steeper slopes. Our standard cross section, typical standard shows, I think, about a 15 feet height bench with a 12 feet width of the bench, staggered up that way. Now we have, as I mentioned earlier, that I 64 job, used some presplitting and we will be doing more of this. In the case of presplitting we will allow, I think, up to the maximum depth that can effectively presplit, about 30 feet. So we will allow, in this case, up to 30 feet depth on the benches. But we will still bench ... What do you think, Bill?

Ken Miller: In Ohio we don't generally require that a bench has to be made midslope and we have cuts on a 1 to 1 slope as high as 100 feet. This is in an alternating shale and sandstone area. Generally our width of benches is 10 feet and we use, as a general rule, 1 to 1-1/2 slopes in poor shale and mudstone rock, and 1 to 1 in the shale, interbedded shale (limestone-shale-sandstone areas), and 1/2 to 1 in sandstones, and sometimes as steep as 1/4 to 1. Wherever we exceed a 1 to 1 slope, we put a bench at the bottom of the roadway ditch.

Mathis: Do you presplit this 1 to 1?

Miller: No, we had difficulties trying to presplit 1 to 1 slopes. We presplit anything steeper than 1 to 1.

Witsack, Purdue: I would just like to make several comments basically on the work that Professor Woods has done and the work from the report that I presented for him this morning. It seems to me that there has been several excellent examples of the basic ideas of physiographic problems just presented in this panel discussion. I have copied down several of them. I heard the Ohio representative say that he had problems similar to those in Illinois; I heard both people from Illinois say that it is quite difficult to get a set of standard specifications for the state of Illinois. The gentleman from Kentucky just asked about some experience dealing with rock slopes. Well, a lot of this, and I don't claim or believe that Professor Woods claims, that the idea of Physiographic Provinces are the ultimate solution, but if you analyzed it and if you kind of forgot for awhile about the idea of political boundaries -- the idea being now, as far as Indiana and Ohio are concerned, why should there be any difference in their subsurface exploration in the glacial country? In reference to the question of the gentleman from Kentucky, there are several sections involved here. We talk about the interior low plateau but Mr. Hotler answered that ideally the basic geology in the surface section of the Highland Rim would go right through Kentucky. Ideally from a geological viewpoint, and the structural geological viewpoint, I think that these are things that people between states should talk about and they should be accepted on a common basis. The panel member from Ohio also answered a question on Kentucky. Kentucky, on the eastern side of the state is in the Appalachian Plateau. It is composed of sandstone and shales, which ideally are the same as those in Ohio. I think the use of physiography or physiographic provinces will really go a long way in this attempt to relate subsurface exploration.

One other point in connection with the standard boring spacing of every 300 feet. I would like to ask the representative from Ohio -- Does he feel that this standard specification would be adequate for both a point on the western and a point on the eastern boundary of Ohio? Do you think this should be rigidly adopted?

Miller: Well, I might be a little more specific on this spacing of borings than just the 300 to 500 foot spacing. All of our borings are located by a reconnaissance engineer or geologist and we would like to see as many borings made as possible, but economically this isn't feasible. Therefore, the geologist or reconnaissance engineer outlines the borings program. It has been our practice that after a review of a number of soil profiles, that they average out both on the western part and the eastern part of the state as between 300 to 500 feet. This is a good average.

This is the way they set their borings. Now this may not be economical, but these are set by men in the field and they feel that these borings represent a particular area. There may be some borings that are repetitious but we try to obtain as much information as possible.

Benson: I wonder if I could make a couple of comments. We, in Illinois, are doing our own soil surveys. Years ago, we used to proceed under the idea that we would make as few borings as possible and try to find out all the information we could. But we have to require a great deal of engineering judgment from the man in the field, and also anticipate when he is going to tighten up on the spacings of the borings so that he doesn't, for instance, reduce the amount of information available by interpreting the specifications using poor judgment. We have seen instances where people blindly go plunging along on the Interstate and punch a hole 300 feet here and 300 feet there, 300 feet over here, and behold, he left a 600 foot bog unbored. And we never would catch it, which was a miscarriage of intent on our part. We want those people to use engineering judgment. We hammer away at our own people in the districts and I think, by and large, the message has been heeded quite well. We don't always have all of the control over some of the consultants that we would like. We turned Dave Novick pretty much loose because we had a great deal of confidence after discussing it with him, and felt that he shared the same feelings in this major undertaking as we did.

But as I say, there is a great deal of engineering judgment required. I don't like to see a standard 300 feet or a standard 100 feet for something when we are talking about say 75% of the state of Illinois, which is pretty well glaciated by both the Illinois and Illinois-Wisconsin drift sheets. If we compare the driftless area in northwestern Illinois with the American bottoms where Dave Novick has been working, or the nonglaciated area down in the southern tip of Illinois you find quite a divergent character of the geology of these different regions. But we had to come up with some kind of a rule of thumb, and we more or less adopted this as a basic format. I suspect that as far as our own participation goes in the original thinking of what was to be used to develop this specification that I was colored by my own experience up in the northern part of the state. We had spacings that got down as close as 100 feet. We were working in the cuts because geologically some of these marine deposits are quite involved and you run into a lot of buried silts, etc. that are locally carrying a lot of water. In the northern part of the state it gets pretty cold, and if you don't go pretty close there you are going to miss some frost-heaves. You get down to the southern end of Illinois and frost is no problem there and they can build their roads out of A48 silts and have no swell at all. This is, as I say, a very loose specification. It is intended to be a minimum specification and one which could be supplemented, as necessary, to cover local complexities. We hope that the engineer who is in the field will exercise his wisdom and supplement them as necessary. Thank you.

Lovell: Yes, sir.

Audience: I would like to know if anyone from any of the States has had any experience with mercury-settlement gauges or the new type of piezonomy just coming out whereby you don't have to have the line completely saturated with water at all times?

Hotler: We have it in Indiana. I can answer that quickly.

Benson: We are going to try at least one installation, I believe, to use the new type of piezonomy. I don't know if there is anything unusual about it. It uses coaxial tubing and you measure the height of the water by the gas pressure. The inside tube, I believe, is 1/8 inch in diameter. It is manufactured in Colorado. It is not too expensive an installation. The only thing is in installing it. The first time you take a reading, you are going to have to allow for an error because you displace the water out of that tube and it has no place to ge except up in the outside tubing and you are introducing an error, which you are going to have to recognize, I believe. We figured in the office about a 12% error the first time around because of the ratio of about 8 to 1 in the volume of the inside tube to the outside one. You are reading the bubble pressure from this gas. After that you have to install a little turn-off valve in the line and you can turn it off and after that your inside pipe will remain essentially water-free. But I think that it looks like a pretty handy tool. I wrote a soils engineer in Canada, I can't think what his name is now, but they had used these devices there and have been quite well satisfied with them.

Novick: We haven't had the experience with these new devices but we had some good luck working with ordinary well-point piezometers. The length of observation is not unduly long if the soils are not especially corrosive. These are very simple, of course. In other words, the more elaborate piezometers are difficult to interpret and are frequently not operating properly.

Benson: The only advantage that we saw in the coaxial tubing piezometer was with very fine grained soils used in a short stretch of embankment where they might be raising the grade in a hurry, and you want to watch the peizometric head to see what is happening there. But if you use a piezometer that is pretty big around for soil that is only so permeable you might have a considerable time lag between where the water wants to get until it has an opportunity to get in there and fill this pipe. We felt that if you went down to where you could use the real small diameter tube and had a pretty good sized head down at the bottom with a pretty good sized stone, you would have an opportunity to get a fast reaction that wouldn't give quite such a dangerous situation under certain circumstances. The one that I am thinking about -- We built a railroad track and I didn't want the darn thing to slide. So we watched it very carefully with this device and had excellent success.

Lovell: Gentlemen, I think this panel has done a wonderful job. Do you agree with me?

West: Thank you, Dr. Lovell. Fine job!

Gentlemen, this concludes the technical session of the 18th Annual Highway Geology Symposium with the reminder that we are having our closing banquet tonight and hope to see all of you there.

Recent Engineering Extension Publications

BULLETIN No. 113. Proceedings of the Forty-ninth Annual Road School. By John F. McLaughlin. (Vol. XLVII, No. 5, September 1963.) Free.

BULLETIN No. 114. Proceedings of Conferences on Land Surveying, 1962 and 1963. By Ken S. Curtis. (Vol. XLVII, No. 6, November 1963.) Two dollars.

BULLETIN No. 115. Proceedings of the Eighteenth Industrial Waste Conference held at Purdue University, April 30, May 1 and 2, 1963. By Don E. Bloodgood. (Vol. XLVIII, No. 3, May 1964.) Ten dollars.

BULLETIN No. 116. Proceedings of the Fiftieth Annual Road School. By John F. McLaughlin. (Vol. XLVIII, No. 4, July 1964.) Free.

BULLETIN No. 117. Proceedings of the Nineteenth Industrial Waste Conference held at Purdue University, May 5, 6, and 7, 1964. Part One and Part Two. By Don E. Bloodgood. (Vol. XLIX, No. 1, (a and b, January 1965.) Five dollars in paperback; ten dollars in clothbound.

BULLETIN No. 118. Proceedings of the Twentieth Industrial Waste Conference held at Purdue University, May 4, 5, and 6, 1965. By Don E. Bloodgood. (Vol. XLIX, No. 4, July 1965.) Ten dollars in clothbound.

BULLETIN No. 119. Proceedings of the Fifty-first Annual Road School. By D. G. Shurig. (Vol. XLIX, No. 5, September 1965.) Free.

BULLETIN No. 120. Proceedings of Conferences on Land Surveying 1964 to 1966. By Ken S. Curtis. (Vol. L, No. 1, January 1960.) Three dollars and fifty cents.

BULLETIN No. 121. Proceedings of the Twenty-first Industrial Waste Conference held at Purdue University, May 3, 4, and 5, 1966. Part One and Part Two. By Don E. Bloodgood. (Vol. L, No. 2, March 1966). Thirteen dollars and fifty cents in clothbound.

BULLETIN No. 122. Combustion of Hydrocarbons -- Property Tables. By R. J. Steffensen, J. T. Agnew, and R. A. Olsen. (Vol. L, No. 3, May 1966.) Two dollars and fifty cents.

BULLETIN No. 123. Proceedings of the Fifty-second Annual Road School. By D. G. Shurig. (Vol. L, No. 4, July 1966.) Free.

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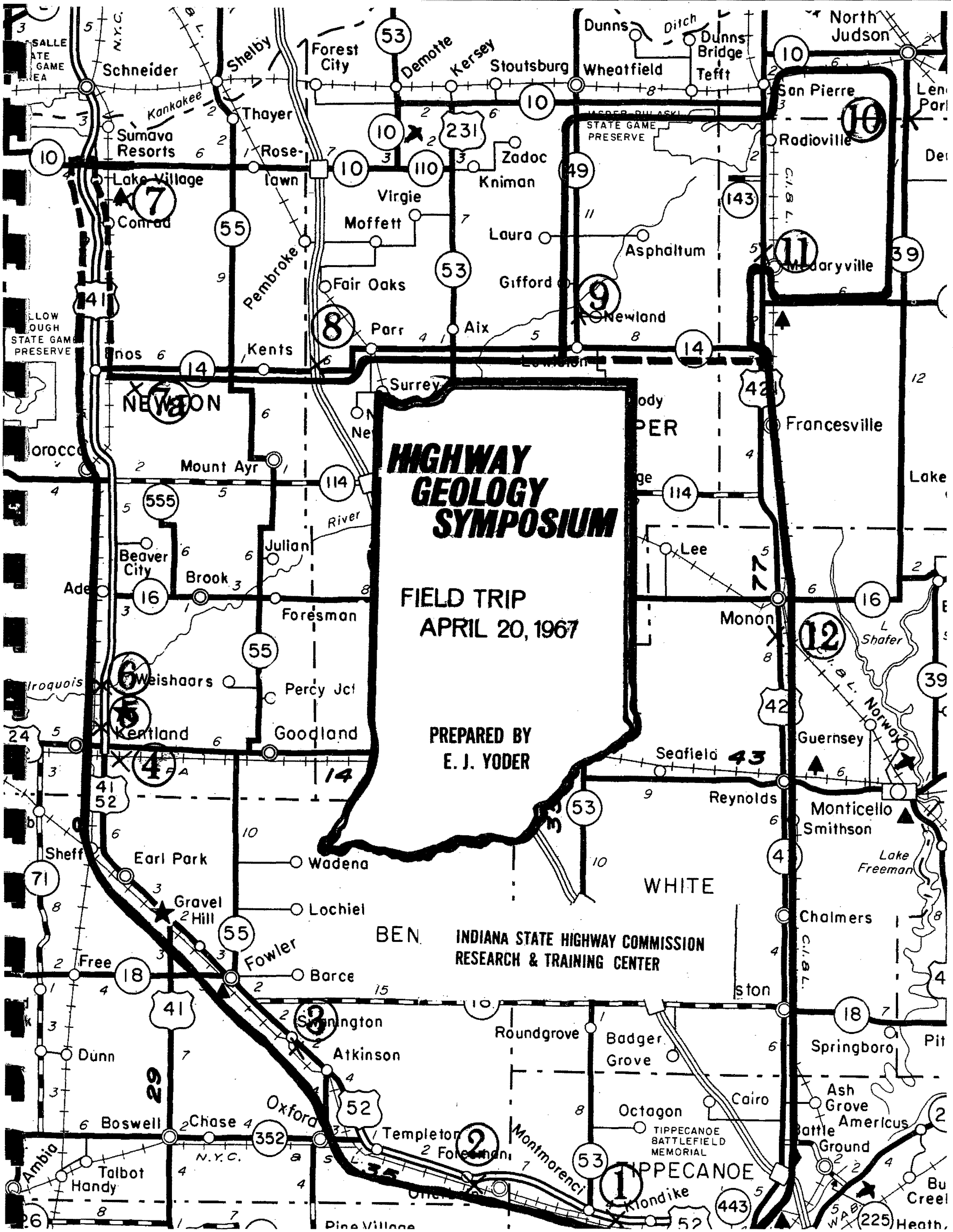
BULLETIN No. 126. Ninth Annual Seminar on Plastics for Tooling, edited by O. D. Lascoe. (Vol. LI, No. 3, May 1967.) Ten dollars.

BULLETIN No. 127. Proceedings of the Eighteenth Annual Highway Geology Symposium. By T. R. West and E. J. Yoder. (Vol. LI, No. 4, July 1967.) Four dollars and fifty cents.



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APRIL 20, 1967
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Brook
Julian
River
Lee
Monon
Foresman
Iroquois
Weishaars
Percy Jct
Kentland
Goodland
Seafield
Guernsey
Reynolds
Monticello
Smithson
Earl Park
Wadena
WHITE
Lake Freeman
Sheff
Gravel Hill
Lochiel
Chalmers
Fowler
Free
Barce
Swanington
Roundgrove
Badger Grove
Springboro
Dunn
Atkinson
Oxford
Templeton
Montmorenci
Octagon
Cairo
Ash Grove
Americus
Battle Ground
TIPPECANOE BATTLEFIELD MEMORIAL
Boswell
Chase
Ambia
Talbot
Handy
TIPPECANOE
Klondike
Pine Village

LOG OF FIELD TRIP

HIGHWAY GEOLOGY

INTRODUCTION

The following pages include a log of mileage readings, a tabulation of miles traveled from the start and a general description of the features that will be observed on the field trip. At frequent intervals during the field trip, the mileage readings will be given so that the participants can tell exactly where the party is at any given time.

Several stops have been planned along the way to observe pavement performance, and in particular, the effect of soils and environment on performance. The field trip also includes a stop at one or more stone quarries.

For the sake of clarity, detailed descriptions are not included in the log. State Highway and Purdue Personnel will discuss various features from time to time, particularly at the stops where time will permit detailed observation of the feature in question. The soils map which is enclosed was prepared by the Joint Highway Research Project, Purdue University.

PURPOSE OF FIELD TRIP

The primary purpose of this field trip is to point out several natural features in the vicinity of Lafayette, Indiana and to study correlations of design and performance with soil conditions, materials and environment. Appropriate pictures have been included where needed to illustrate pertinent points.

The field trip will cross two major physiographic features. Corresponding engineering problems associated with these features will be discussed. The first of these features is the Wisconsin Till Plain; this area is considered to be typical of nearly one half of the State of Indiana. The next feature is the sand area in the Northwest portion of the state which is unique.

It is pertinent to note that it may not be possible to visit each of the stops that are designated on the maps and on the log. Further, two alternate routes are given in the extreme northern portion of the field trip. Whether or not these portions of the field trip will be taken depends in large part upon the time that is involved in the stops during the morning.

HIGHWAY FIELD TRIP, HIGHWAY GEOLOGY SYMPOSIUM

GENERAL DESCRIPTION OF THE GLACIATED AREA

The route of the field trip in the glaciated region follows routes U.S. 52 and U.S. 41 from Lafayette, Indiana to Kentland, Indiana and from there due north. The primary portion of the route in the glaciated area is from miles 0.0 to 39 although some additional drift is encountered later in the trip.

The parent soils in the area are unsorted till of Wisconsin age. The parent soils contain mixtures of clay, silt, sand and gravel. The B Horizon soils are quite plastic and may range from 8 to 24-inches in depth. The area north of Lafayette is very flat and the water table is perched or near the surface during the spring of the year. The low areas contain top soils with relatively high organic content whereas the higher locations are inorganic. The major source of aggregate in the area is from outwash deposits of the Wabash River.

As the route leaves the Wabash valley the vegetation and its influence on soils changes from forest cover to prairie cover.

From mile 39 near Kentland to mile about 45, the route crosses the Iroquois Lacustrine Plain. Here the area is very flat and the soils plastic.

Near Kentland, Indiana, aggregates for construction are primarily crushed rock taken from the quarry at that location. North of Kentland, sand deposits of varying quality are to be found. These sands are used in construction of subbases for concrete pavements and in secondary road construction.

The primary design problem, in so far as concrete pavements is concerned, is pumping of these pavements when they are constructed directly on the in place soils. Therefore, primary consideration is given to use of subbase courses under concrete pavements and, as will be discussed on the field trip, the performance of these pavements is directly associated with type of subbase as well as the drainage conditions in and around the pavement.

The primary problem associated with asphalt pavements is that of spring break-up and freeze-and-thaw. Secondary pavements of the area are nearly always of the flexible type and, in many cases, have not been designed as such but are reconstruction of old county roads. Primary break-up associated with spring thaw occurs in cut areas where adverse ground water conditions are encountered as well as the plastic B Horizon soils.

ROAD LOG

Mileage Reading	Miles	Stop Number	Remarks
	0.0		Corner of Stadium and Northwestern
	1.0		Start of concrete pavement just beyond Police Barracks
	2.0		Klondike, pavement is re-surfaced in spots
	3.0	1	Constructed 1947 Pavement 9"-7"-9" PCC Subbase 6" Gravel Subbase Drainage None, Trench Construction

Mileage Reading	Miles	Stop No.	Remarks
			This pavement has a long history of pumping. Defects noted are transverse cracks and a large number of restraint cracks. Figure 1 shows a view of a restraint crack and Figure 2 shows a view of a joint and resulting spall that is due in part to infiltration. Figure 4 shows the distribution of cracking for this and other pavements. Looking northwest at this location, water is nearly always found running from the pavement due to the fact that the subbase was constructed in a trench without sub-drains. The next stop will contrast this situation where drainage was provided in the subbase. The soils at this location, and up to mile 51 are drift of Wisconsin age. These soils have typical soil profile development.

Figure 1. Restraint cracks.

Mileage Reading	Miles	Stop No.	Remarks

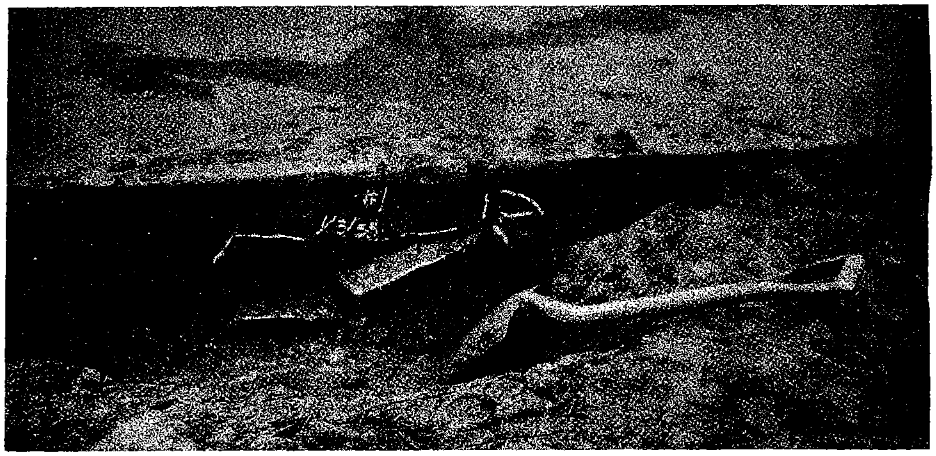


Figure 2. Joint spall resulting from second stage blowing.

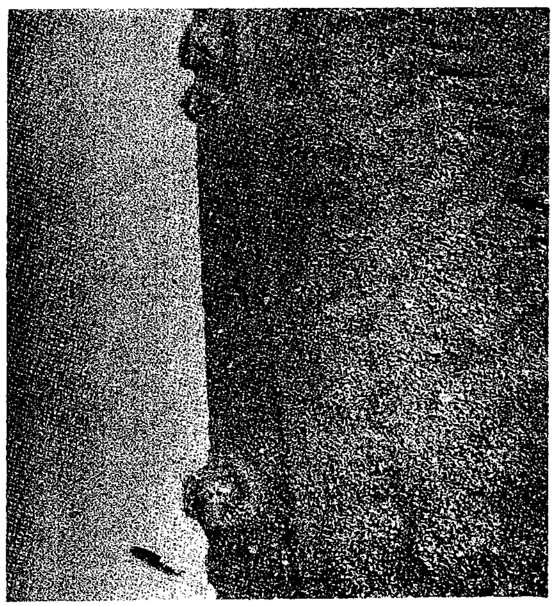


Figure 3. Second stage blowhole (pumping) on a gravel base.

Mileage Reading	Miles	Stop No.	Remarks
			This pavement from the start at mile 1.0 to mile 7.5 has shown performance typical as at this location. Pumping has been on the decrease since about 1960 but evidence is still seen after periods of heavy rains.
	6.7		Junction S. R. 53

Mileage Reading	Miles	Stop No.	Remarks
	7.5		Start of 1953 Pavement Constructed: 1953 Pavement: 9"-9"-9" PCC Subbase: 4" inside - 7" outside gravel Subbase Drainage: Through shoulder, except subdrains at special locations

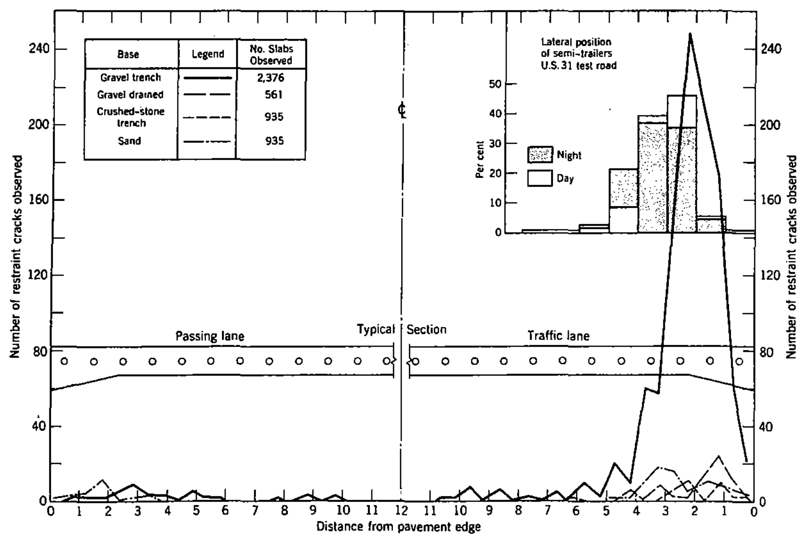


Figure 4. Lateral Position of Restraint Cracks on Several Indiana Highways

	11.4	2	At this location, it is noted that the performance is much better. The reasons are probably many including, thicker pavement, less age, but most important, the subbase is drained. The pavement showed high amount of pumping soon after it was opened to traffic. Pumping, however, has been slight since about 1958.

<table>
<tr><th>Mileage Reading</th><th>Miles</th><th>Stop No.</th><th>Remarks</th></tr>
<tr><td></td><td></td><td>2(cont'd)</td><td>Some restraint cracking is noted but this has not progressed to a serious stage. Just north of this location, nearly every joint was a pumper for the first several years of traffic. Approximately 100 samples were taken from this stretch of highway and the pavement represented by Stops No. 1, 5 and 6. The percent of subbase material passing the No. 200 mesh sieve (average) is as follows.
<table>
<tr><th>Base Type</th><th>No Pumping</th><th>1st Stage Pumping</th><th>2nd Stage Pumping</th></tr>
<tr><td>Gravel</td><td>7</td><td>12</td><td>12</td></tr>
<tr><td>Stone</td><td>10</td><td>14</td><td>16</td></tr>
<tr><td>Sand</td><td>17</td><td>19</td><td>19</td></tr>
</table>
</td></tr>
<tr><td></td><td>14.2</td><td></td><td>Start of old pavement in south-bound lane. This old pavement was built directly on the native subgrade in 1928 and is a 9-7-9 pavement. It has a long history of pumping, but has been resurfaced and under-sealed. Pumping on the pavement is now minimal but can still be detected after heavy rains, especially in the spring.</td></tr>
<tr><td></td><td>16.5</td><td></td><td>Town of Templeton</td></tr>
<tr><td></td><td>16.9</td><td></td><td>Junction S.R. 352</td></tr>
<tr><td></td><td>17.8</td><td></td><td>End four lane pavement. This pavement is the same as described above. It is a high accident road and the state is constructing two additional lanes.</td></tr>
</table>

Mileage Reading	Miles	Stop No.	Remarks
	22.1	3	Stop at the batch plant for the new construction of this road. The new pavement is 9 inches thick, has a subbase of gravel, subbase is 5"-8" inches thick and is drained
	22.3		Town of Swanington
	25.3		City of Fowler
	29.5		End new pavement
	30.1		Junction U.S. 41. This pavement is the same as the older pavement described above. Pumping is still noted near this intersection.
	32.2		Town of Earl Park
	38.2		Start Pavement with stone base Constructed 1949-52 Pavement 9"-8"-9" PCC Base 6" Crushed Stone Drainage None, Trench Construction
		4	Kentland Stone Quarry
	39.2	5	This pavement has a long history of pumping, but, very little 2nd stage pumping has ever been seen. Crack interval here is about 10 to 15 feet. The slabs are faulted as are many of the cracks. Some cracks are faulted enough to suspect torn steel. There is nearly always water under this pavement. The water table is generally high and the soils are clayey. Water is no doubt trapped in the subbase.

Mileage Reading	Miles	Stop No.	Remarks
		5 (cont'd)	Figure 5 shows a view of a trench dug at a site just north of this Stop and Figure 6 shows a 1st stage pump on the opposite lane just north of this Stop.

Figure 5. Water running from open-graded subbase course, near Stop No. 5

Figure 6. First stage blowhole on a crushed stone subbase, U.S. 41, Kentland Indiana

Mileage Reading	Miles	Stop No.	Remarks
	41.7		Start 1953 drained sections Constructed 1953 Pavement 9"-9"-9" PCC Subbase 4" inside - 7" outside crushed stone Subbase Drainage Through shoulder except in special locations This pavement has much better performance than that at Stop No. 5. Particular attention is directed to the fact that the stone subbase here (as well as at Stop No. 5) has considerably more fines than the gravel subbases at Stops 1 and 2.
	42.4		Drain outlet
	42.8		Bridge
	43.3	6	Drain outlet for the subbase. This is 1953 pavement as above and water is nearly always seen at this outlet.

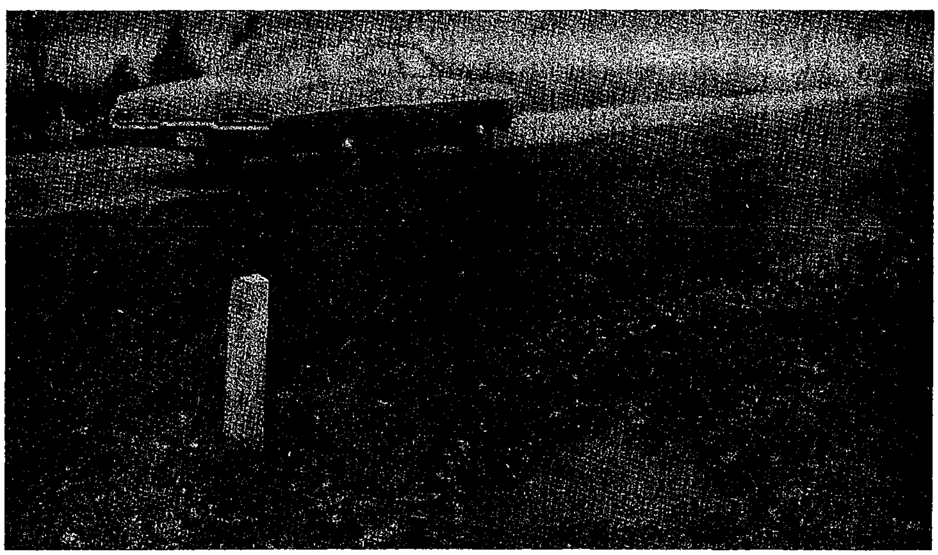


Figure 7. View of Drain at Stop No. 6

Mileage Reading	Miles	Stop No.	Remarks
	45.4		Junction S.R. 16
	49.4		Start 1954 pavement. This is a 9"-9"-9" inch pavement, stone subbase which is drained. The performance has been excellent.
	50.5		Junction S.R. 114. This marks the general transition ~~into the~~ Kankakee Sand Plain from here north.
	51.7		Railroad overpass, sand dunes on right.
	52.8		Sand and gravel pit, start 1962 pavement 9"-9"-9" pavement, stone subbase.
	55.9		Town of Enos, junction S.R. 14

GENERAL DESCRIPTION OF THE KANKAKEE SAND BASIN

Continuing from mile 50.5 (Junction of U.S. 41 and S.R. 114) the area under consideration lies generally within the Kankakee Lacustrine Section. However, from mile 68.5 to near the junction of S.R. 14 and U.S. 421 the area under consideration is covered with Wisconsin Till.

The major portion of Indiana has been covered once or more by the glaciers and their deposits of drift. The northern part of the state lies fully within the bounds of the last advance of the glaciers so that the soils and topography are controlled by glacial deposits and subsequent development. In the area under question, the northern boundry of the Kankakee Sand Basin is marked by the Valparaiso Moraine and the southern border by the Marsailles Moraine. The area is marked on the attached soils map as water deposited sands.

The Kankakee Sand Basin is associated with glacial drainage and ponding along the St. Joseph, Kankakee, Tippecanoe and Iroquois Rivers. The area consists of a sand lacustrine plain which is underlain with Wisconsin Drift.

A large portion of the Kankakee Sand Basin was, thirty years ago, a swamp under several feet of water much of the year. The area has been famous historically as a wild foul and game gathering place and, incidentally, the state still maintains a game reserve in the area. At the time that the area was primarily swampy, many ridges and islands of sand developed. These sand islands and ridges, at the present time, are readily noted on the field trip and numerous sand dunes are seen in the area.

Over the past thirty to fifty years a great deal of effort has gone into draining the area to make it usable for farming. Therefore, it will be noted on the field trip that many drainage ditches are crossed; some of these ditches may be as much as 20--25 feet in depth. As a result of the drainage offered by the ditches, much of the area is now usable and is rich fram land.

Except for the muck and peat deposits which are seen in the area, the area is covered largely with sands. The sand ridges and dunes may attain a height from 10-30 feet. Several moraines cross the area and offer some gravels for construction.

The predominating feature of the area is the fact that the area is relatively flat and drainage is to the west with a very low gradient. The area consists of sands on till accompanied by a high water table. During the sping of the year, much of the land is still under water in spite of the extensive drainage conditions.

The present sands in the basin are generally quite organic in the low spots approaching the condition of sandy muck. These organic sands accompanied by the high water table offer several unique problems in so far as highway construction across the area is concerned.

Materials of construction in the region are scarce except for the stone quarries near Kentland, Francesville and Monon. Several sand and gravel pits are also found. Performance of pavements of the area is strikingly influenced by topographic position, corresponding depth of water table and location of organic deposits. Performance of the pavements is generally good where the roads are elevated and depth of water table is relatively deep but very poor where the highway is located on the lower organic deposits. Poor foundations conditions obviously exist where peat and muck are encountered.

This portion of the field trip is intended to demonstrate the effect of water table and poor foundation conditions on pavement performance. The contrasting performance of pavements in the high areas compared to those in the low areas is striking and evident. Further, particular attention is directed to the contrasting soil conditions of the Kankakee Sand Basin and the adjacent drift area just south of the sand basin. Most of the pavements seen on this leg of the trip are in the state Secondary System.

ROUTE OF ALTERNATE NO. 1

Mileage Reading	Miles	Stop No.	Remarks
	55.9		Town of Enos and junction SR 14
	4.8		Start 1963 pavement 9"-9"-9" pavement, crushed stone subbase, 4" and 7" thick.
	7.6		Lake village, sand dunes
	9.1		Junction S.R. 10, turn right on S.R. 10
	10.9		Start asphalt pavement, note poor performance in low areas.
	11.5		Ditch crossing
	11.8		Sand dune, note good performance
	11.9		Road decends to organic sand, high water table, longitudinal cracking and consolidation in spots.
	12.4		Intersection of S.R. 10 and Meridian County Road. S.R. 10 parallels a drainage ditch and offers a good example of the effect of high water table on performance. Turn South (right) on Meridian County Road.
	13.4	7	Drainage ditch at right illustrates the soil profile and high water table at this location.
	14.4		Road goes up into sand dunes. Road in excellent condition.
	15.0		Road decends onto organic sand. Numerous chuck holes and cracking is noted.
	15.4		Ditch.

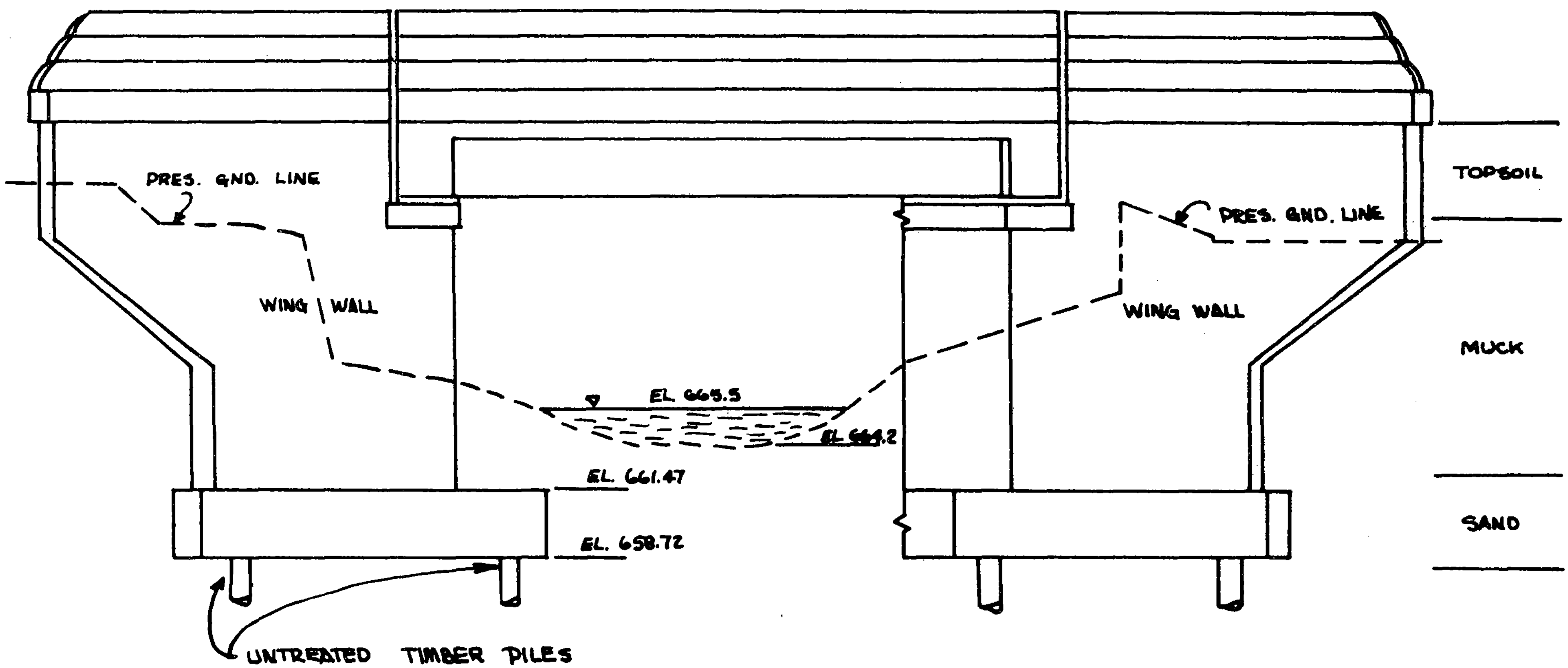
Mileage Reading	Miles	Stop No.	Remarks
	16.4		End of Meridian Road, turn east on County Road 600 N
	17.4		Turn south on County Road 100 E
	17.6		Ditch
	17.9		Ditch
	18.8		Ditch at the edge of a sand dune
	20.4		Ditch
	20.8		Ditch. Turn west at County Road 250 N and 100 E
	21.8		Turn south on County Road Meridian at junction of Meridian and County Road 250 N
	23.3		Junction S.R. 14. Turn east on S.R. 14

Mileage Reading	Miles	Stop No.	Remarks
	55.9		Town of Enos, Junction S.R. 14 and U.S. 41 Turn east on S.R. 14 Constructed 1935 Surface 2" Asphalt Concrete Base 6" Crushed Stone Note: Breakup just beyond 1st cross road where the road is on grade. The soil is organic.
	58.0	7 (a)	Pavement as above, resurfaced in 1955 with 1" A.C.
	58.9		Junction Meridian Road Pavement as at mile 58.0 Note: General poor performance. Road is at grade on organic sand.
	60.3		The soils of this area are highly organic and very poor pavement performance is noted.
	61.0		The sands in this area are elevated and better pavement performance is noted.
	62.0		Junction S.R. 55 Constructed 1937 Surface 2" Asphalt Concrete Base 8" Crushed Stone The thicker base and slightly elevated grade and better ditches have resulted in better performance than the road west of S.R. 55

Mileage Reading	Miles	Stop No.	Remarks
	65.2	8	Intersection of S.R. 14 and Interstate 65. At this location construction of a bridge over I 65 is under way. The embankment is entirely of sands of the area. Starting here the following designs on S.R. 14 Constructed 1937 Surface 2" Asphalt Concrete Base 7" Crushed Stone From here east the road drainage (ditches) is is poorer and breakup is again noted in the low areas. Contrast the performance in the sand area with that in the drift area beyond mile 69.
	67.0		S.R. 14 jogs north. Note lack of ditches along the road and corresponding poorer performance.
	67.6		Poor pavement performance is noted where the pavement is low. S. R. 14 turns east.
	68.5		Town of Parr. At this location the road leaves the Kankakee Sand Basin and is on an area of thin sand on top of drift. Spring break up is generally noted in the low spots and in some of the slight cuts.
	71.6		Cut. The soil at this location is bouldery and covered with sand.
	72.2		Spring break in cut just before Junction.
	72.3		Junction S.R. 53. Performance of the pavement beyond here is quite good. Design as follows.

Mileage Reading	Miles	Stop No.	Remarks
			Constructed 1937 Surface 2" Asphalt Concrete Base 7" Crushed Stone Resurfaced in 1961 with 1" A.C. Total now 3" A.C. + 7" Base
	73.1		One mile stretch of resurface, 1941, with 2" bituminous coated aggregate.
	73.5		Ditch on the left hand side parallels the road. Water in this ditch flows west.
	74.5		The road at this location ascends the drift material. Performance at this location is typically that of the Wisconsin drift, generally poor in cuts.
	76.0		Low organic soil with some minor break up.
	77.7		Junction of S.R. 14 & S.R. 49 Turn north on S.R. 49 Constructed 1946 Surface 1" Asphalt Concrete Base 9" Gravel Note the muck on the left.
	78.7		Road at this location goes over the muck. The road has been resurfaced and general poor performance has been noted at this location. Muck is noted on the right side of the road.
	78.9		Junction of S.R. 49 and County Road 225N. Looking east on County Road 225N general poor condition of this road is noted. Excellent exposures of muck are seen in a ditch on the north side of County Road 225N.

Mileage Reading	Miles	Stop No.	Remarks
	80.2	9	Bridge over Oliver Ditch. The general profile is shown in Figure 8 on the next page. Here some muck and marl is present under the approach slabs. This is contrasted to the peat trestle at Stop No. 11. Note the slabs are badly cracked. The road in both directions is undulating.
	82.5		Pavement changes here Constructed 1951 Surface 1" Surface Treatment Base 8" Gravel Resurfaced 1951 with 3" A.C. and in 1953 with 1" A.C. Total is now: Surface 5" Asphalt Base 8" Gravel Patching was carried out in 1965.
	83.2		Ditch construction on the right. Road goes up. Performance improves because of thicker pavement and better drainage.
	84.9		Road to Jasper - Pulaski State Reserve
	86.3		Bridge, same design as at Stop No. 11
	87.8		Junction S.R. 10 and S.R. 49. Turn right on S.R. 10. The condition of S.R. 10 from here to Junction of 421 offers a classic example of the effect of high water table and soil condition on performance of secondary pavements. For the next eight miles excellent performance is noted when the highway is located at elevated grade in the sand dunes and very poor performance is noted when the road is

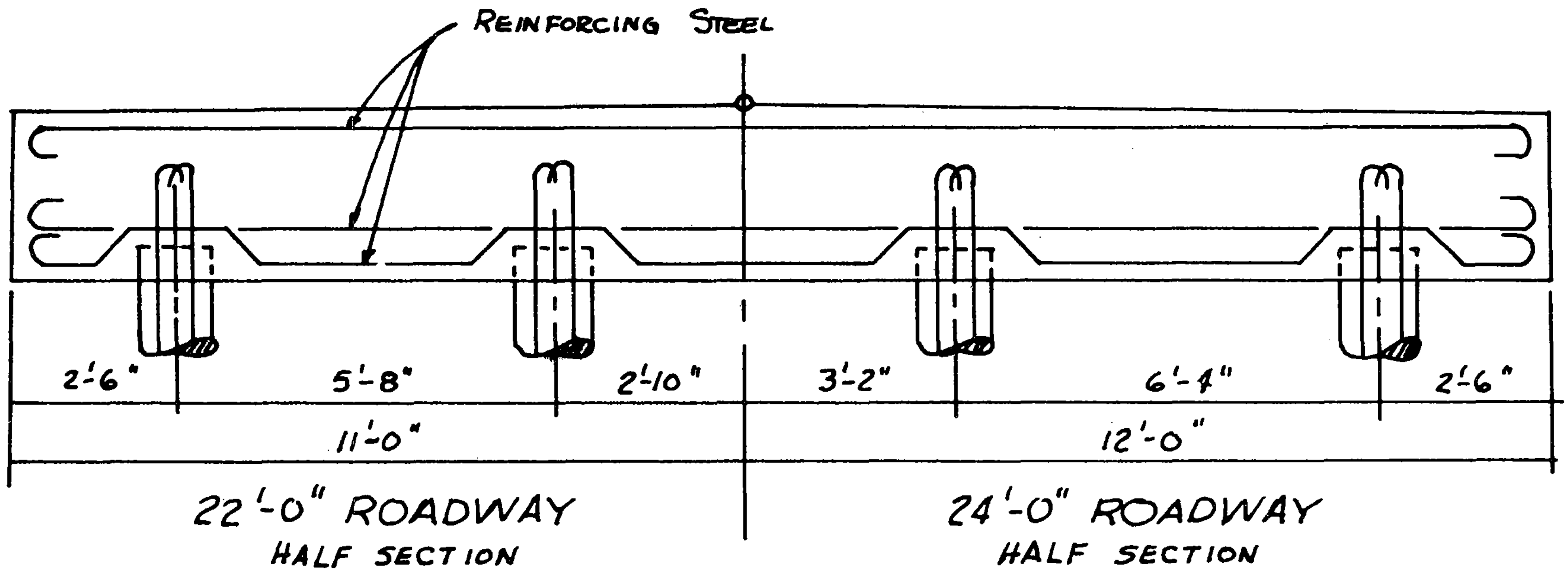


LONGITUDINAL SECTION BRIDGE OVER OLIVER CREEK S.R. 49, NO. JCT. 14

FIG. 8

Mileage Reading	Miles	Stop No.	Remarks
			located down on organic sands with high water table. The design is as follows: Constructed 1930 Surface Surface Treatment Base 7" Crushed Stone Resurfaced in 1937. Thickness now Surface 2" Asphalt Base 7" Crushed Stone
	95.7		Junction S.R. 10 and U.S. 421. Turn north on U.S. 421
	97.7		Junction S.R. 10. Turn east on S.R. 10 Constructed 1930 Surface 2" Bituminous coated aggregate Base 8" Crushed Stone Resurfaced in 1943, 2" and in 1953, 2 1/2". Thickness now: Surface 6 1/2 Asphalt Base 8" Crushed Stone Performance in general is good. Note that the effect of topographic position and soil has been minimized because of stronger pavement and good drainage. Moderate to severe bleeding of the surface is noted.
	103.7	10	Junction S.R. 39 and S.R. 49. Turn south on S.R. 39. Constructed 1948 Surface 1" Asphalt Base 6" Soil-Cement The next several miles are over a large area of muck. A soil-cement base was used. Note the transverse cracks. The general condition of the road is good. Muck. Road has been resurfaced Muck.

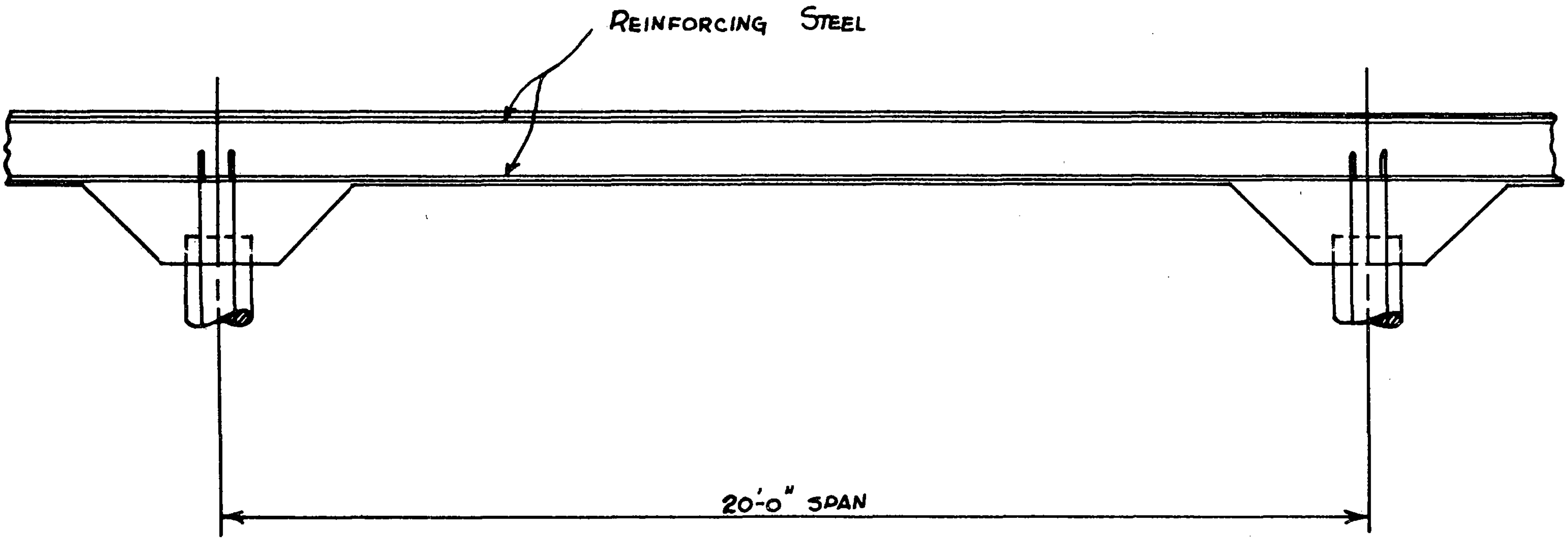
Mileage Reading	Miles	Stop No.	Remarks
	109.0		End of soil-cement. Design here is: Constructed 1945 Surface Base 6" Gravel This pavement is in excellent condition due to general elevated poistion and good drainage.
	114.7		Junction S.R. 39 and S.R. 14 Constructed 1938 Surface 2" Asphalt Base 8" Crushed Stone Resurfaced in 1963, Thickness now Surface 3 1/2" Asphalt Base 8" Crushed Stone Road is in excellent condition
	120.7		Junction U.S. 421 and S.R. 14. Turn north to to inspect Peat and Muck Trestle
	122.7	11	Peat and Muck Trestle. Here the state has built a trestle to preclude settlement of the bridge approaches. The designs for this are shown in Figure 9, 10, and 11.
	124.7		Junction of U.S. 421 and S.R. 14, go south on U.S. 421 Two designs were used on U.S. 421 and S.R. 43 from Medaryville to Lafayette. The flexible design is: Constructed 1931 Surface 2" Rock Asphalt Base 12" Gravel Resurfaced 1951, 1964. Total now: Base 12" Gravel Surface 5 1/2 Asphalt



A TRANSVERSE SECTION OF CONTINUOUS SLAB

MUCK TRESTLE U.S. 421, MEDARYVILLE, IND.

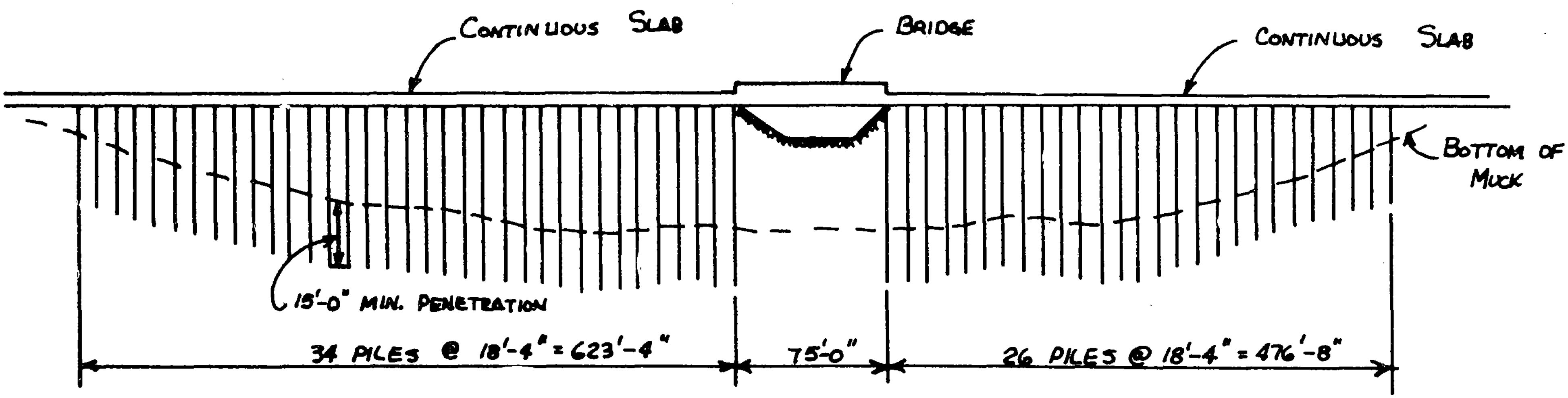
FIG. 9



A LONGITUDINAL SECTION OF CONTINUOUS SLAB

MUCK TRESTLE, U.S. 421, MEDARYVILLE, IND.

FIG. 10



LONGITUDINAL SECTION OF MUCK TRESTLE

U.S. 421, MEDARYVILLE, INDIANA

FIG. 11

Mileage Reading	Miles	Stop No.	Remarks
			The rigid design is: Constructed 1930 Pavement 9"-7"-9" P.C.C. Resurfaced in 1960 with 2" Binder and 1" Surface. The rigid pavement here has never shown appreciable pumping. On the other hand, much of the orginal gravel road was left under the pavement.
	131.7		Francesville Stone Quarry
	133.7		Junction with S.R. 114
	139.4	12	Monon Stone Quarry
	169.4		Lafayette