PROCEEDINGS OF THE FOURTEENTH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Held

March 22, 1963

at the

A. & M. College of Texas

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College Station, Texas

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ACKNOWLEDGEMENT

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It is hoped the technical papers published in these proceedings will have useful information for the readers.

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APPLICATIONS OF AGRICULTURAL SOIL SURVEYS TO HIGHWAY GEOLOGY

By ADRIAN PELZNER
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Introduction

Until about the beginning of horseless-carriage days, the location of highways in the United States was easily accomplished. The highway builder merely improved existing coach trails. The principal function of the coach trail was to connect centers of population. Little thought was given to the ultimate destination of the traveler. The main job of the road was to get the traveler to a town by nightfall so that he could eat, rest and continue his trip in the morning. Thus, latter day motorists, riding on improved coach trails, are often required to travel in a southerly direction for many miles to reach a destination which may have been due west of their starting point.

Locating a highway facility today can be a bewildering complex assignment. The location engineer must not only choose an alinement that will provide the most service for the most people, but he must also consider many other factors. Some of the other factors he has to consider are:

1. Vertical and horizontal sight distances.
2. Cost of right-of-way.
3. Economic losses and benefits.
4. Balance of earthworks.
5. Cost of construction.
6. Surface and subsurface soil and rock conditions.

This paper is primarily concerned with the soil and rock condition factor in connection with highway route locations.

Soil and Rock Conditions

The various state highway departments and other agencies concerned with highway route location are giving much consideration to soil engineering and highway geology. These two disciplines augment and supplement each other. To merely consider only the engineering properties of soils, and ignore the geological agents of transformation, transportation and deposition, often has caused engineering disasters. When the location engineer has been provided with a geological report for several different highway alinements, he can intelligently evaluate the soil and rock condition factor. For example, if he knows that a soil deposit is eolian, such as sand dunes, it will follow that the soil material will be uniform in size at any one location and the particles will be cohesionless or nonplastic. Thus, armed with this knowledge he can (1) be provided with preliminary information on the engineering properties of the soil deposit and the degree of variation of these properties both vertically and horizontally and (2) plan the type of subsurface exploration that is required. Therefore, it is quite apparent that, in connection with the location of highways, a knowledge of the geology of the area is fundamental to the study of engineering behavior of soils.

Sources of Information

A highway geologist when acquiring knowledge of the geology of an area through which a highway is to pass should walk the area, make field observations and take samples for laboratory study. However, prior to any of these steps he will study the geological literature of the area. Fortunately, in this country there is available much information concerning surface and subsurface conditions. Some of these sources of information are:

1. Topographic maps - United States Geological Survey.
2. Geological maps and reports - State Geological Survey and United States Geological Survey.
3. Master's and Doctor's theses - Universities.
4. Files of agencies concerned with construction - state highway departments, Departments of public works, etc.
5. Aerial photographs - United States Geological Survey, United States Department of Agriculture, etc.
6. Soil survey bulletins - United States Department of Agriculture.

The soil survey bulletins are listed last since the remainder of this paper will be concerned with them. They are by no means the least important source of information.

Soil Survey Bulletins

In many ways a well written, modern soil survey bulletin gives the highway geologist the most information concerning surface and subsurface conditions for the least investment of time, money and effort. The soils that are described in the bulletins are classified into the pedological system. Consequently, the soils engineer or highway geologist who uses the soil survey bulletin should know something of pedology. Pedology is the science that deals with soil as a natural body. The pedologist studies the origin and development of a soil, determines its physical and chemical properties, assigns a pedological classification and delineates it on a soil map. A basic principle in pedology is that if the factors of climate, time, topography and parent material are the same, the soils developed or influenced by these factors will be the same. This prinicple is particularly significant to the highway geologist because the soil throughout a mapped soil unit has similar engineering properties and therefore its engineering behavior can be predicted.

The soil survey bulletins contain sufficient background information on pedology to permit the highway geologist to extract the desired engineering information. Such terms as soil profile, series, type and horizon are defined. The formation, classification and morphology of the soils are described.

Cooperative Program of Soil Conservation Service, Bureau of Public Roads and State Highway Departments

The Bureau of Public Roads recognized the value of soil survey information to the highway engineer and geologist and, in 1951, entered into a cooperative agreement with the U. S. Department of Agriculture to perform engineering tests on samples submitted by the Department of Agriculture or other agricultural agencies cooperating with USDA in the National Soil Survey.

The test data are reported in the county soil survey bulletins and serve as a basis for engineering interpretations and estimates. Having the engineering test data, a logical expansion was to prepare an engineering interpretations section for the county soil survey report. Currently, most of these engineering sections are prepared by Soil Conservation Service personnel of the U. S. Department of Agriculture with some assistance from highway engineers. The Bureau of Public Roads assists Soil Conservation Service in editing the engineering section of the report.

There are approximately 3,500 counties in the United States. Although samples are not being submitted for all these counties at the same time it was evident from the beginning of the cooperative program that the Bureau's testing facilities would be severely overtaxed if the Bureau of Public Roads was to perform engineering tests on all the soil samples collected in these county soil surveys. Because of this and other factors the Bureau has encouraged state highway departments to enter into three-way cooperative arrangements with the Soil Conservation Service and the Bureau of Public Roads whereby the state highway department would do the testing and assist in the writing of the engineering sections. This type of arrangement serves the double purpose of giving the state highway department confidence in the reported data and encouraging the State highway department to use the information contained in soil survey bulletins for choosing route locations, planning pre-construction soil surveys and indicating potential construction material sources.

Since the inception of the cooperative program there have been over 10,000 samples taken for engineering tests in 392 counties in 47 states and Puerto Rico. Thirty-four state highway departments are cooperating in this program. Over 100 soil survey bulletins have been published containing an engineering section, and, as of January 1, 1963, 71 were being processed for publication.

The Texas Highway Department was one of the first state highway departments to enter into cooperative arrangements with Soil Conservation Service and the Bureau of Public Roads. Their participation in the National Soil Survey program dates back to 1957. As of January 1, 1963, the Texas Highway Department has supplied engineering test data and additional information for the engineering sections of 21 county soil survey bulletins. The information contained in one of these bulletins (Lamb County, Texas) is discussed in the following sections.

Soil Survey Bulletin for Lamb County, Texas

The type of information contained in a modern soil survey bulletin is depicted by the following listing from the Table of Contents of the Lamb County soil survey bulletin:

1. General nature of the county.
2. General soil areas.
3. Description of the soils.
4. Wind erosion and its control.
5. Use and management of the soils.
6. Engineering applications.
7. Formation, classification and morphology of the soils.
8. Additional facts about the county.
9. Glossary.
10. Literature cited.
11. Guide to mapping units, capability units and range sites.

A highway geologist or soils engineer when using the Lamb County bulletin to extract information on soil and rock conditions in the county and in planning a detailed soil survey for a proposed highway alinement would be most interested in the "Engineering Applications" section and the information in the geology section. The geology of the area is one of the items included under the general heading "Additional facts about the county."

Geology Section

The geology section reviews the geologic history of the area starting with the formation of the Permian Red Beds from marine sediments deposited from a shallow inland sea. The High Plains rose above the level of this sea and streams flowing over the exposed Permian Red Beds eroded fine-textured materials and redeposited them in flood plains. These materials formed the Triassic Red Beds. The area was again inundated and sand, clay and limestone were deposited over most of the area forming Cretaceous deposits. These were later mostly eroded away after the Rocky Mountain uplift.

As the Rocky Mountains eroded, gravelly, coarse material was deposited in alluvial fans. The finer materials were transported and spread farther to the east. The Ogalalla formation developed from these deposits of outwash. The text describes the deposition of the Ogalalla formation as the outstanding geologic event in the history of Lamb County since it is the main source of irrigation water in the county.

The materials from which the soils of the county developed were deposited during the Pliocene epoch and were later reworked, mostly by wind, in the Pleistocene epoch. This wind shifted and sorted the surface materials.

Although the geologic history, as described in the bulletin, is rather brief, it certainly helps the user of the report to understand the nature of the geologic agents that have been at work in the county. Certainly the information contained in the engineering applications section can be better understood when the geologic history of the area is comprehended.

Engineering Applications Section

The principal subsections of the Engineering Applications Section of the Lamb County report will be described in some detail.

Engineering Classification Systems

The engineering classification systems that are used in the soil survey bulletin are based on the gradation and plasticity a soil material possesses. In general, if a soil is non-plastic or very slightly plastic and is coarse textured it is best suited for engineering purposes.

In the unified classification system, used by the U. S. Corps of Engineers, Bureau of Reclamation and other engineering agencies, pairs of letters are used to designate the soil groups. The first letter of the symbols used in this report are S, M, and C standing for sand, silt and clay, respectively, while the second letter of the symbols are P, L, and H meaning poorly graded, low and high plasticity, respectively.

In the AASHO system, a method standardized by the American Association of State Highway Officials, the soil materials are classified from A-1, gravelly soils of high bearing capacity, to A-7, clay soils of low bearing capacity when wet.

Soil Test Data and Engineering Properties of the Soils

Twenty-one soil series have been mapped in this county. The areal extent of five of these series comprise approximately 65 percent of Lamb County. Five soil types have been selected as examples from the table in the bulletin that shows the estimates of properties that are significant to engineering of all the soils that are mapped in the county. Portions of this table of estimated properties are shown in Table 1. The data given in the "Percentage passing sieve" column are estimates which are based on actual test data, when available, or test data for similar soils from other counties. The shrink-swell potential of a soil, reported in the last column, is the property of the soil material to change volume when subjected to changes in moisture. A soil that is rated high in shrink-swell potential is of significance to engineers since it can cause severe damage to the engineering structures built on it.

The Amarillo comprises 45 percent of the land area of the county. Other than its being erodible, it does not have properties that would cause engineering problems.

The information of significance concerning the Arvana soil is the presence of rocklike caliche at a shallow depth, as indicated in the description column. The text advises that the soils in Lamb County do not provide a source of sand or gravel and that bedrock is not likely to be encountered. Therefore, the Arvana and a few other soils in the county are very significant in that they indicate local areas of potential aggregate supply.

The Brownfield is similar to the Amarillo in that it does not have properties that would cause engineering problems other than its erodibility.

The information in the description column for the Randall along with the estimated classifications of A-7-6 and CL warn the engineer and geologist concerned with highway location of potential trouble. They should certainly avoid locating a highway in the Randall if at all possible.

The information concerning the eolian nature of the Tivoli is of particular significance to location engineers and geologists. Deep shifting sand dunes present design, construction and maintenance problems. This soil should also be avoided whenever possible.

The soil survey bulletin for Lamb County also contains a table of test data prepared by the Bureau of Public Roads from test reports supplied by the Texas Highway Department. This table is of interest since it reports actual test data for some of the

Table 1. — Brief Description of Soils and Estimated Properties Significant to Engineering, Lamb County, Texas

Soil Type	Description	Depth from Surface Inches	Classification Unified	Classification AASHO	Percentage passing sieve No. 4	No. 10	No. 200	Shrink-swell Potential
Amarillo Loamy Fine Sand	Loamy fine sand over moderately permeable, well drained sandy clay loam; developed on unconsolidated alluvial and eolian sandy sediments.	0-14	SM	A-2-4	100	100	15-20	Low
		14-45	SC	A-6	100	100	30-40	Low
Arvana Fine Sandy Loam	Moderately permeable, well drained fine sandy loam over sandy clay loam on nearly level to gently sloping areas; developed on thin eolian mantle over rocklike caliche.	0-6	SM-SC	A-4	100	100	40-50	Low
		6-30	CL	A-6	100	100	50-60	Mod.
Brownfield Fine Sand	Fine sand over well drained moderately permeable sandy clay loam; occurs on broad gently undulating sandhill areas; developed from sandy earths that appear to be eolian.	0-20	SP-SM	A-2-4	100	100	5-10	Low
		20-50	SC	A-6	100	100	35-45	Mod.
Randall Clay	Dense clay to depth of 6 feet; occurs in intermittent lakebeds; receives runoff from adjoining areas and is submerged for long periods; parent material is calcareous clay.	0-10	CL	A-7-6	100	100	60-70	High
		10-26	CL	A-7-6	100	100	75-90	High
		26-50	CL	A-7-6	100	100	80-90	High
		50-70	CL	A-7-6	100	100	80-90	High
Tivoli Fine Sand	Windblown deposits of noncalcareous fine sands 6 to 75 ft. high; occurs in east-west strip 2 to 6 miles wide that bisects county.	0-72	SP-SM	A-3	100	99-100	2-10	Low

soils that are mapped in the county. It gives the test data for specific layers or horizons of the typical modal profile of the reported soil as well as the variants or non-modal profiles of the soil. For the Lamb County report the table gives test data on gradation, liquid limit, plasticity index and volume change. The engineering classifications (unified and AASHO) are also given in the table of test data.

This table of test data enables the user of the report to gain some insight into the physical properties of the mapped soils for which actual test data has been reported. It also enables the user to understand the range of properties that can be expected for the mapped soil.

Engineering Interpretation of the Soils

Table 2 shows some of the engineering interpretations for the five selected soils. The text states that the rating for suitability of the soil material for road subgrade is based on the estimated classifications of the soil materials. Soils that have a plastic clay layer, such as Randall clay, impede internal drainage and have low stability when wet. Such soils are rated as "Poor." The erodible loamy fine sands that have a large percentage passing the No. 200 sieve are rated "Poor to fair," for road subgrade. Fine sands having a somewhat lesser percentage passing the No. 200 sieve are rated "Fair."

The text advises that the suitability of a soil for road fill depends largely on its natural water content and texture. Plastic soils such as Randall clay that have a high moisture content and are difficult to handle, compact and dry are rated "Poor."

Ratings of the soils with respect to the effect of materials and drainage on the vertical alinement of highways are also given in Table 2. The text discussion of the "Materials" column indicates that various detrimental features were considered in arriving at the ratings. Some of these features are the presence of rocklike caliche in the Arvana, erodible fine sand in the Brownfield and Tivoli and plastic clay in the Randall. Similarly, the features that were considered in arriving at the ratings in the "Drainage" column were high water table, seasonal floods and seepage over impermeable strata.

The Amarillo is a loamy fine sand. It is rated as poor to fair for road subgrade. The basis for rating loamy fine sands for road subgrade is given in the text as follows:

> The loamy fine sands, which are very erodible and have a large percentage passing the No. 200 sieve, are rated "Poor to fair." This rating is based on their poor grading and general lack of stability unless confined.

In the Arvana series the rating of "Poor to fair" is given for Road subgrade and Materials. For Road subgrade the rating is given because of the fine texture and high plasticity of the Arvana. The rating of poor to fair is given for Materials because of the rocklike caliche layer at depths of 10 to 36 inches. The presence of hard caliche may require special excavation equipment, which would make excavations more expensive in this material.

The Brownfield is a fine sand and is rated as poor to fair for Materials. This rating has been assigned, according to the text, because cuts made into fine sand expose highly erodible material to the action of wind and water.

The Randall, as previously described, is a wet plastic clay. It has a high shrink-swell potential and is classified as A-7-6 and CL. These undesireable properties and engineering classifications are reflected in the rating of "Poor" which has been assigned to the Randall in all the columns in Table 2.

The Tivoli is a deep, cohesionless eolian sand lacking stability unless confined. For this reason it has been assigned a rating of "Poor" for Road subgrade. The erodible nature of the soil material is the reason for the rating of "Poor" in the Materials column.

Table 2. — Engineering Interpretations of Soils, Lamb County, Texas

Soil	Suitability of soil for - Road subgrade	Road Fill	Vertical Alinement of Highways Materials	Drainage
Amarillo	Poor to fair	Fair	Fair	Good
Arvana	Poor to fair	Fair	Poor to fair	Good
Brownfield	Fair	Fair	Poor to fair	Good
Randall	Poor	Poor	Poor	Poor
Tivoli	Poor	Fair	Poor	Good

Summary and Conclusion

As previously stated, 21 soil series have been mapped in Lamb County. These soil series have been delineated in the bulletin on a detailed soil map which is based on aerial photographs. The type of engineering and geologic information contained in the bulletin has been shown by presenting the information for five of the soils.

Certainly the highway geologist or soils engineer by judiciously extracting engineering and geologic information from the bulletin is in a better position to assist the location engineer in evaluating the surface and subsurface soil and rock conditions. If alternate route locations are being considered for a proposed highway in Lamb County, the geologist or engineer can point out the problem soil areas. For example, if all other large areas of Tivoli and Randall might be rejected in favor of some other alinement because of the detrimental properties these soils have. Conversely, if the evaluation of the location factors dictate the location of a highway through large areas of Tivoli and Randall soils, the design, construction and maintenance problems in connection with these soils can be recognized in advance.

The soil survey bulletin, unfortunately, is not a panacea. There is other soils information that should be known in connection with route locations that cannot be obtained from the soil survey bulletin. For one thing, the soils that are described in the report are usually not described to depths greater than 5 or 6 feet. If the proposed grade line is to be deeper than this depth, not much information can be obtained on the conditions that exist at the proposed grade. Also the bulletin does not give strength information that can be used in the design of pavements or the type of footings necessary for bridge and culvert construction. It is for these reasons that borings must be made, samples taken and laboratory tests performed.

The important thing a soil survey report, such as the Lamb County report, can do is to familiarize the highway engineer and geologist with the various soils along a proposed highway location. It can point up the problem soil areas. Intelligent use of the information contained in a county report enables the engineer to plan a sampling program to provide necessary laboratory test data for design purposes while avoiding the expense and delay of unnecessary sampling and testing. The county report can also be very useful in the search for construction materials such as aggregate, sand and borrow.

In conclusion, as stated earlier, a well written modern soil survey bulletin gives the engineer or geologist the most information concerning surface and subsurface conditions for the least investment of time, money and effort. It is for this reason that the Bureau of Public Roads supports the program of the Department of Agriculture and has entered into the cooperative arrangements to assist in the preparation of engineering information for the reports.

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CORRELATION OF CULVERT PERFORMANCE AND SOIL CONDITIONS

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Introduction

The Missouri State Highway Department, in common with other similar organizations, is now using higher standards in designing highways than were formerly considered adequate. Straighter alignment, flatter grades, deeper cuts, and higher fills on softer foundation material are all parts of the general problem facing the highway engineer.

A small segment of the overall picture in Missouri has to do with the fact that standard culvert design has not always proved to be sufficient to resist the stresses imposed by these advanced design procedures. Signs of culvert distress were first noticed six or seven years ago, not long after construction of some of the higher fills on soft foundation soil. Reports of joint openings of 2 feet, cracks of 6 inches, and 50% reduction of effective flow due to sag, were common. To sift fact from rumor, it was decided to make a thorough investigation of culvert performance.

During the summer of 1962, we made an extensive study to find out how large the problem was, concentrating the major part of our efforts in the known critical areas. Two hundred thirty-nine culverts were included in the investigation, of which 169 were concrete boxes of various sizes, 67 were concrete pipes and 3 were corrugated metal pipes. A few interesting facts will be mentioned later concerning the metal pipes, but in general they are not considered of importance to this study because of their small number.

Each of the culverts in the study was checked for distress signs, as well as location, size, fill height, fill slopes, fill materials, foundation material and approximate age. Culvert performance on three specific highways has been periodically investigated during the past several years. Structures on these routes were included in the study, and will be separately discussed later in this report.

Information derived from this investigation enables us to suggest several remedial measures, to be detailed later. This report is primarily concerned with culvert performance, and presents information on:

a. Geology of Missouri
b. Definition of culvert distress.
c. Summer study and results.
d. Detailed studies.
e. Corrugated metal pipe performance.
f. Conclusions.
g. Design approach.

Geology of Missouri

Ozarks

The Ozarks region, in general, covers the central and eastern part of the State south of the Missouri River, except for the flatlands or "Bootheel" area in the southeast. The topography of this region ranges from gently rolling hills to the mountainous areas around Fredericktown and Branson. The streams are deeply entrenched and the valleys narrow and steep. Culvert locations in the Ozarks region are thought to be safe from any foundation failures, since the streambeds are usually firm or incompressible material, generally composed of ledge rock, shale, gravel, or residual soils containing rock fragments. Investigation of culvert sites usually consists only of visual inspection by the District Geologist. If a box culvert is being planned for a location and ledge rock is found in the streambed, the District Geologist checks the quality and uniformity of the rock for the possibility of leaving the floor out of the box.

Box culvert sites are drilled with a three inch power auger to determine depth to rock. If found at an economical depth (two or three feet) extensive augering is done to check the uniformity of the rockline by drilling holes on ten foot centers along each wall of the proposed structure. If the geologist has any question as to the suitability of the rock for a culvert floor, one or two cores, 2-1/8" in diameter by 5' long, are cut and examined.

Western Plains

The Old or Western Plains region forms the western part of the state, south of the Missouri River. This area is generally rather level, with a few rolling hills. Streams and valleys are wider than in the Ozarks and streambeds are generally ledge rock, shale, residual soils or shallow alluvial deposits.

Culvert site investigations are slightly more involved here than in the Ozarks, although the District Geologist usually investigates proposed culvert sites as was described for the Ozark region. The exception occurs when alluvial deposits are encountered in the valleys.

The silty, alluvial valleys are augered to check the depth and consistency of the compressible material, to locate the water table, and any drainable sand or gravel layers that may be present. If it is the opinion of the geologist that a problem exists, other measures, described in the final section of this report, are adopted.

MAJOR GEOLOGIC REGIONS OF MISSOURI

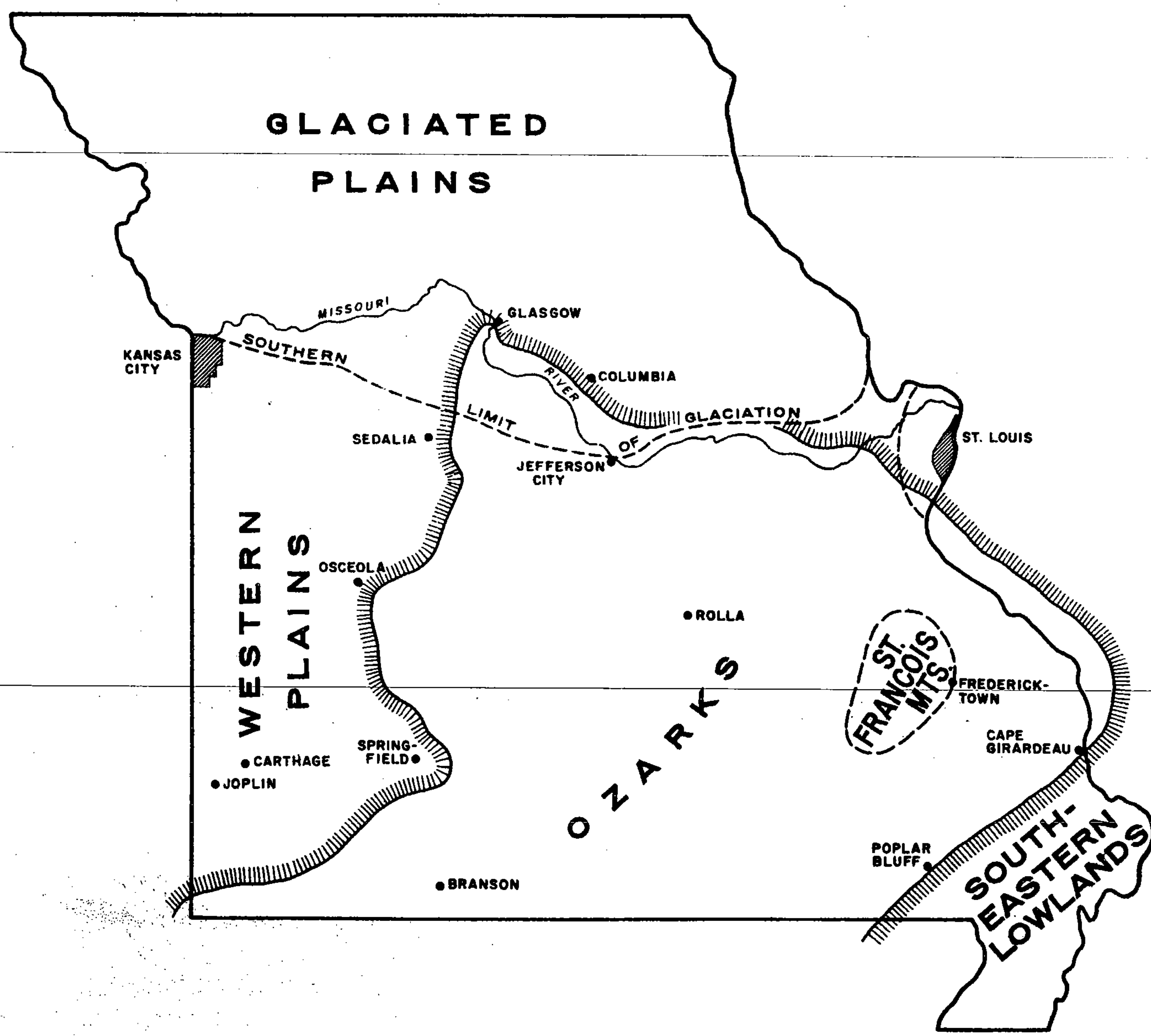


Figure 1.

Lowlands

The Southeast Lowlands, as the name implies, is a flat, low area in southeast Missouri. This is by far the smallest distinct physiographic province in Missouri, and until about 1930, was largely swampland. Today it forms perhaps the largest man-made drainage area in the world. About one third has a heavy, plastic clay mantle from five to twelve feet deep, which is underlain by clean, dense sand. The clay often dries to form a hard surface crust, but a foot or two below the surface it is usually wet, and, indeed, may even by mucky. Along the Mississippi River and in the northwest part of the lowlands area a layer of very fine sandy loam, approximately ten feet deep, is found. This is also underlain by sand. The remaining part of this region is generally dense sand from the surface to a depth of several hundred feet.

Here, as in the Ozark Region, the culvert sites are thought to be safe from any foundation failures, primarily because the level terrain requires only the shallowest of fills. Although some of the worst foundation material in the state is found here, it is usually shallow, and unsuitable soil encountered in culvert excavation can easily be removed and replaced by a more stable material.

Glaciated Plains

The Glaciated Plains region covers almost all of Missouri north of the Missouri River, where the topography ranges from flat to hilly. The loessial and glacial soils, along with modifications of each by the other, occur in thick blankets in most locations. Valleys vary in width from less than a hundred feet to more than a mile. The wider valleys generally contain alluvial soil deposits which may be 80 feet in depth, with 50 feet being a "rough" average for the larger valleys.

In the central and eastern sections of the Glaciated Plains, the terrain is comparatively level, and highly unstable soils may be encountered anywhere throughout the area. However, the depth to shale, rock, or firm Kansan till is usually less than 20 feet. Consequently, foundation problems are considered less serious here than further west.

Most culvert sites in this section warrant careful study, with northwestern Missouri presenting the worst culvert foundation problems in the state. Fill heights here may be 80 feet or more, and the foundation is often made up of soft, compressible material which may be 80 feet deep.

In this region, the procedures described later as "Design Approach" are applied rigorously and conservatively.

Distress of Culverts

While there are many forms of culvert distress, those largely attributable to foundation conditions are:

a. Loss of flow line from settlement.
b. Opening of joints from settlement or lateral spreading, Figure 2d.
c. Cracking of walls, ceiling and floor from settlement or lateral spreading, Figure 2c.
d. Vertical or horizontal displacement at joints and cracks.
e. Infiltration of soil at cracks and opened joints, which clogs culvert and creates voids in fills, Figure 2a.
f. Deterioration of concrete or metal from alkali or acid soils.
g. Damage to structure by construction equipment during backfilling operations. While not strictly a soil problem, it may occur during soil manipulation and be aggravated by settlement, spreading, or other movement. Hence it is listed here.

To put each of these is proper perspective involves a consideration of pedology, geology, soil mechanics, and structural mechanics as well as a realization that these are never totally separable. For instance, settlement and lateral spreading (plastic flow) are companion problems.

Summer Study

In the summer of 1962, the culvert performance survey previously referred to was conducted in 18 counties on primary and interstate routes, in an effort to evaluate the extent and severity of culvert damage and check on the efficiency of corrective measures that had been used.

Since a complete detailed state coverage was considered to be too ambitious a program, the investigation was confined to thirteen counties in north Missouri and five in the southern part of the state. In the northern glaciated area with zones of loess, a comprehensive cross section of moderate to poor foundation soils may be found under high fills. Conversely, in the five counties selected in the Ozark uplift area, foundations were on or near Ordovician formations of Gasconade and Jefferson City dolomite and Roubidoux sandstone. Here incompressible foundations of rock or firm foundations of clay, sand and gravel under high fills were studied.

To emphasize the scope of the investigation it may be worth repeating that the complete study encompassed these structures;

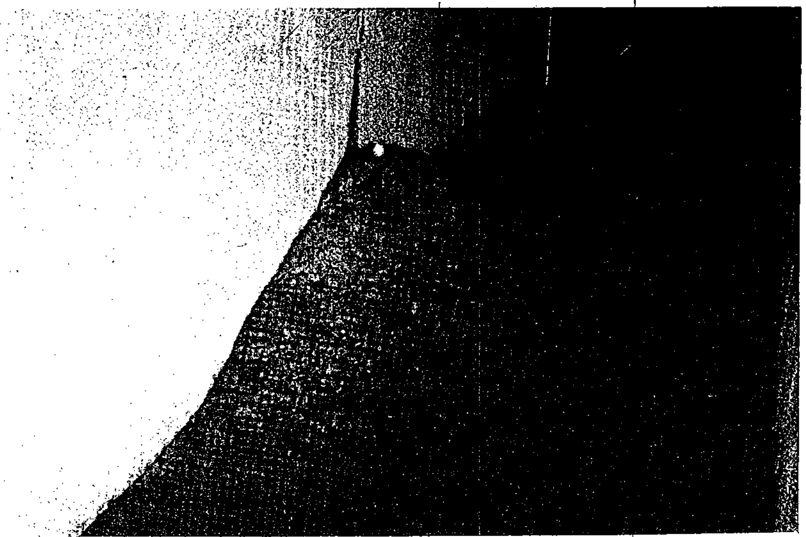


Fig. 2-a, Rt 1-29, Buchanan
Dirt Infiltration

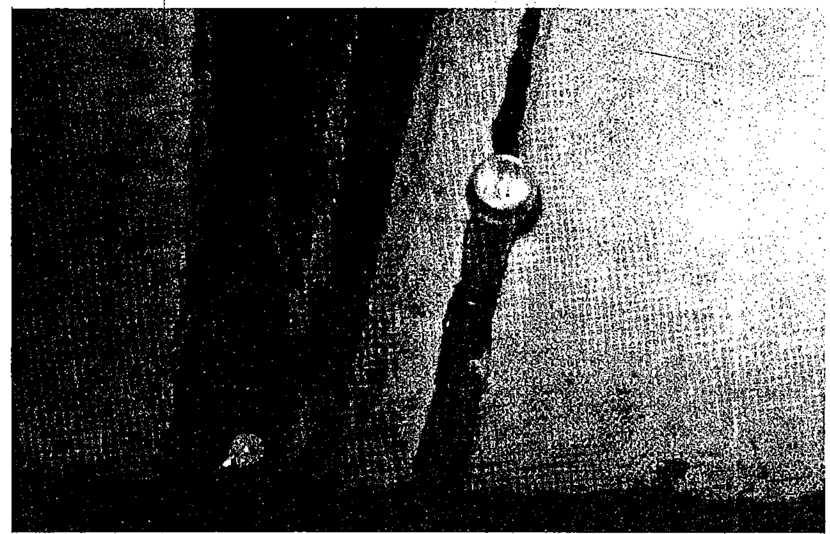


Fig. 2-b, Rt 36, Buchanan
Crack within collar limits

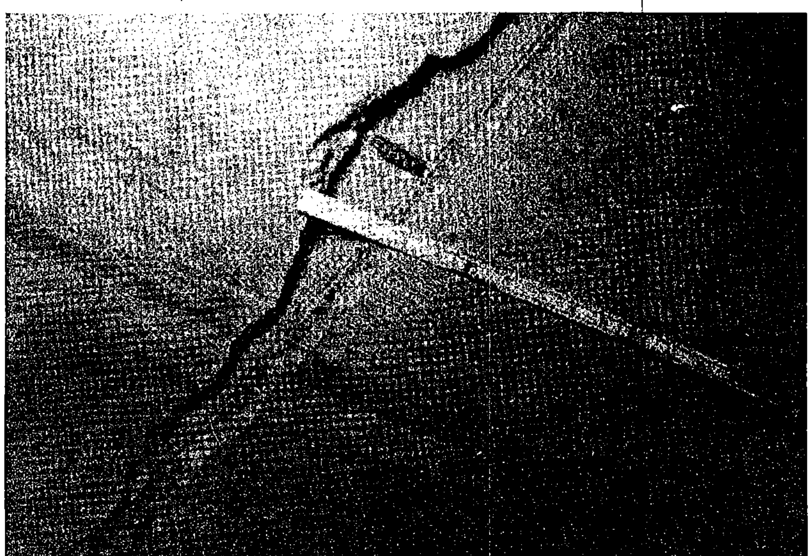


Fig. 2-c, Rt 36, Buchanan
Cracks in Barrel

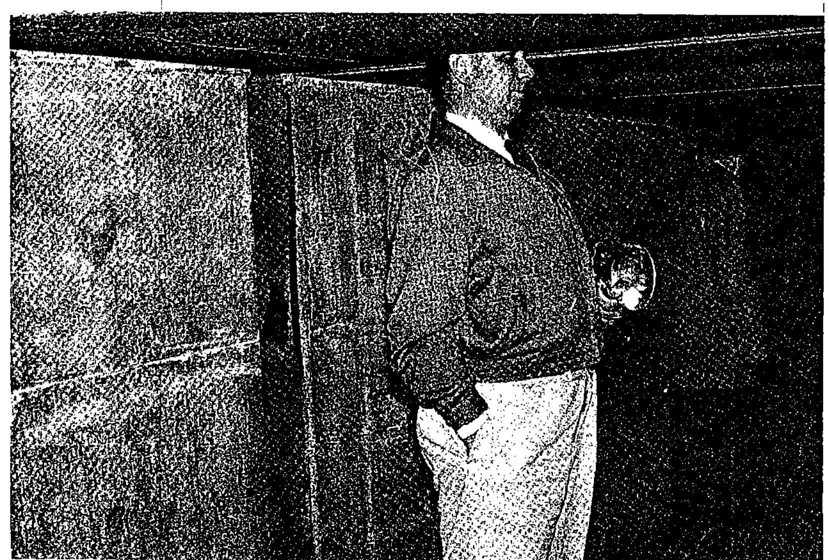


Fig. 2-d, Rt 169, Worth
Open Joints

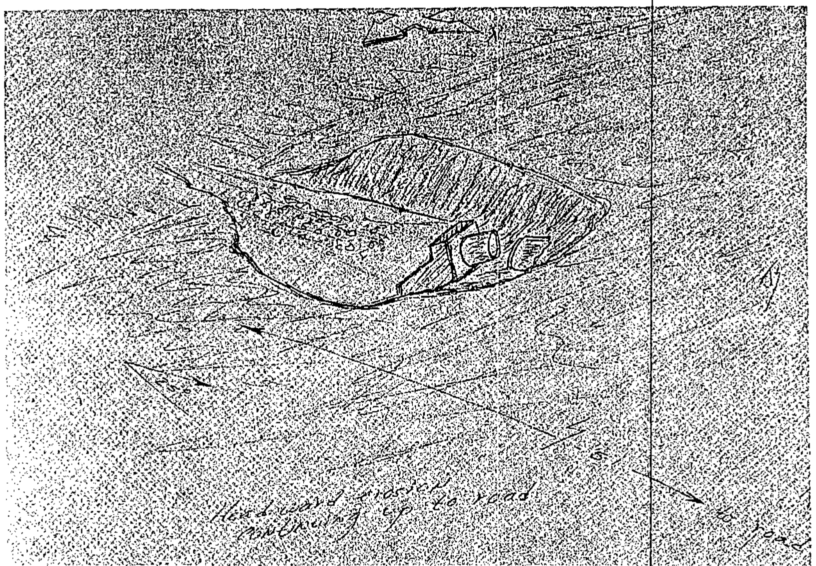


Fig. 3-a, Rt 136, Sullivan
Headwall undercutting

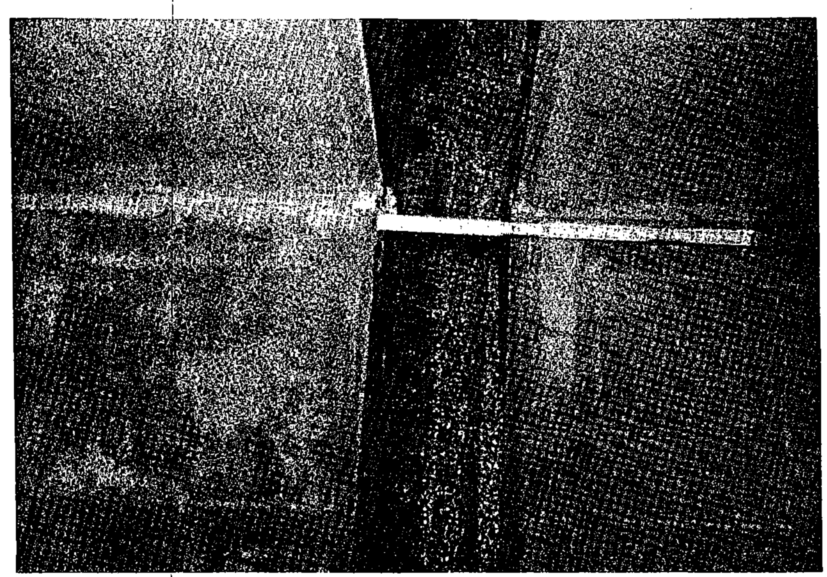


Fig. 3-b, Rt 169, Worth
Collared joint opened

Fig. 3-c, Rt 169, Worth
Flume and Energy Dissapator

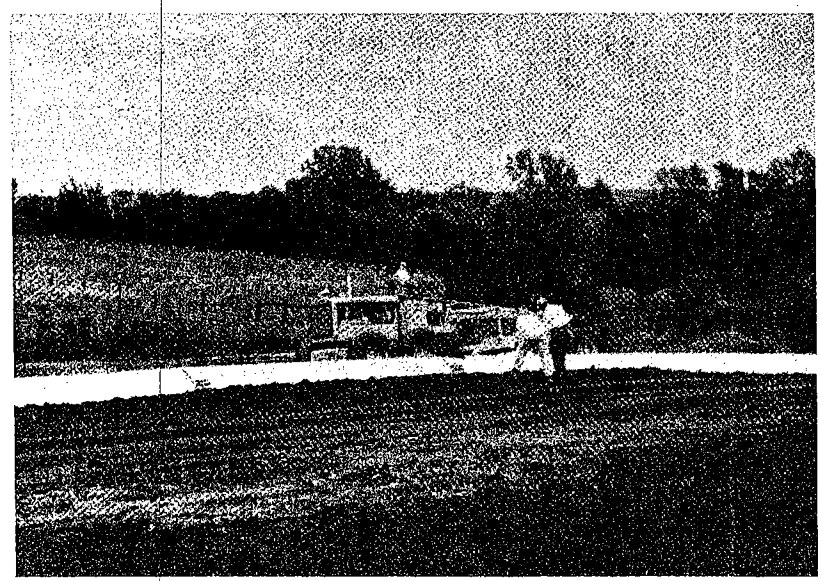


Fig. 3-d, Rt 169, Worth
Pipe Cambered 2 feet

Type	Number
1. Concrete Box	169
2. Concrete Pipe	67
3. Corrugated Metal Pipe	3
Total of	239 culverts investigated

An interesting sidelight shows that the 239 culverts represent a total of 43,255 feet or 8.2 miles of culvert crawled or walked through and examined. As stated earlier, this study encompasses those three projects described later under "Detailed Studies". The three corrugated metal pipes examined were all in excellent condition, but are not further considered in this report because of the few that were available for examination.

The study had definite limitations, with the principal ones being:

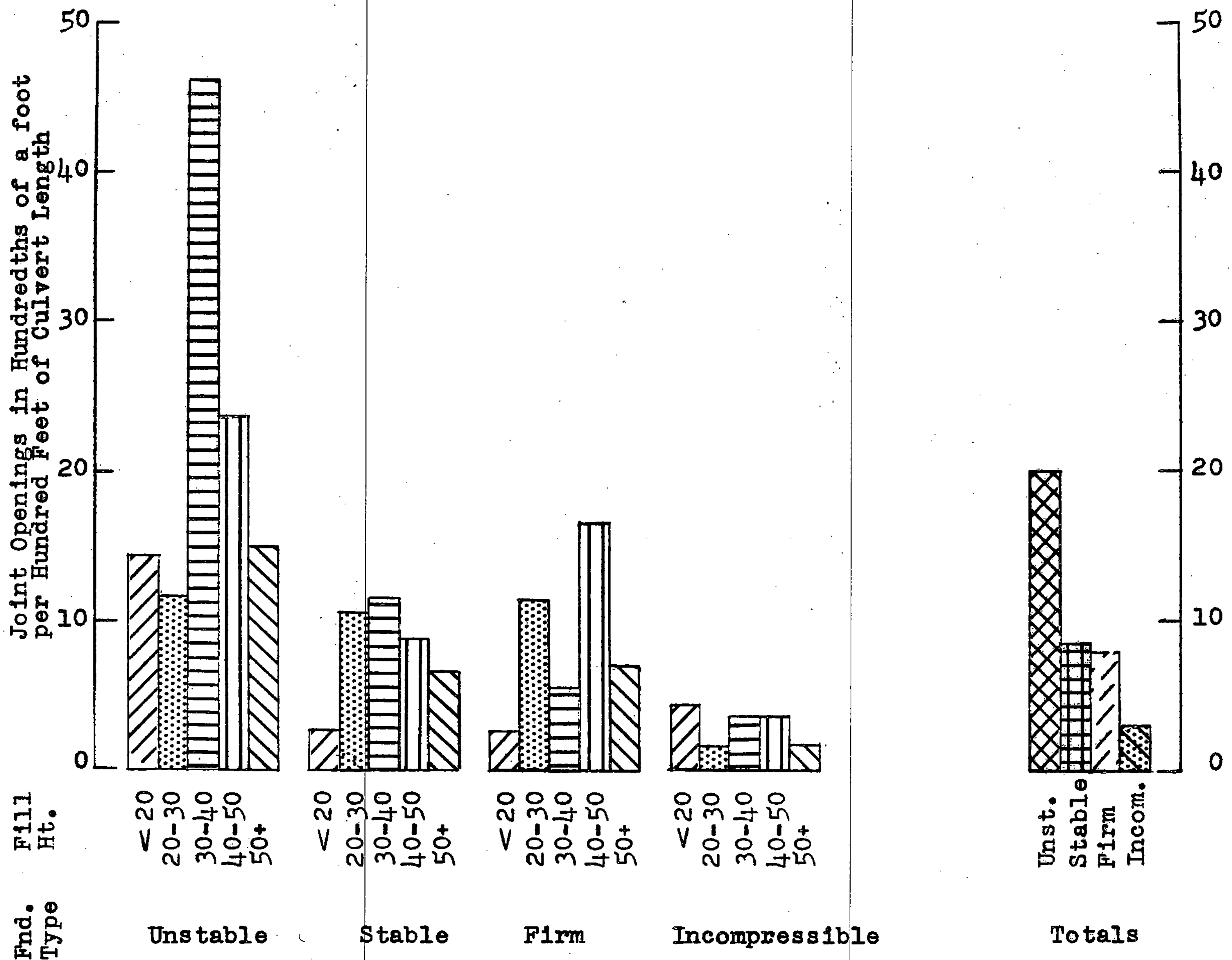
1. Size of structure men could pass through.
2. Silting and ponding prevented inspection.
3. Incomplete soil information. Time was not available for augering each site at the time of study. Soil information was necessarily based on observation, construction diaries, District Geologists' reports (and memories), and any other source considered reliable. This post-mortem is subject to all the fallibilities inherent in such a reconstruction.
4. Loss of camber was largely determined by visual inspection. Time was not taken to run levels.
5. The very few corrugated metal pipe installations on the projects included in the investigation.

Foundations were grouped under four classes:

a. Incompressible, limited to rock or shale.
b. Firm, including glacial tills, compact sands, gravels, etc.
c. Stable, as medium or silty clays, compact silts, loose sands, reworked tills, etc.
d. Unstable, usually low strength, colluvial or alluvial soils, and some soft glacial clays.

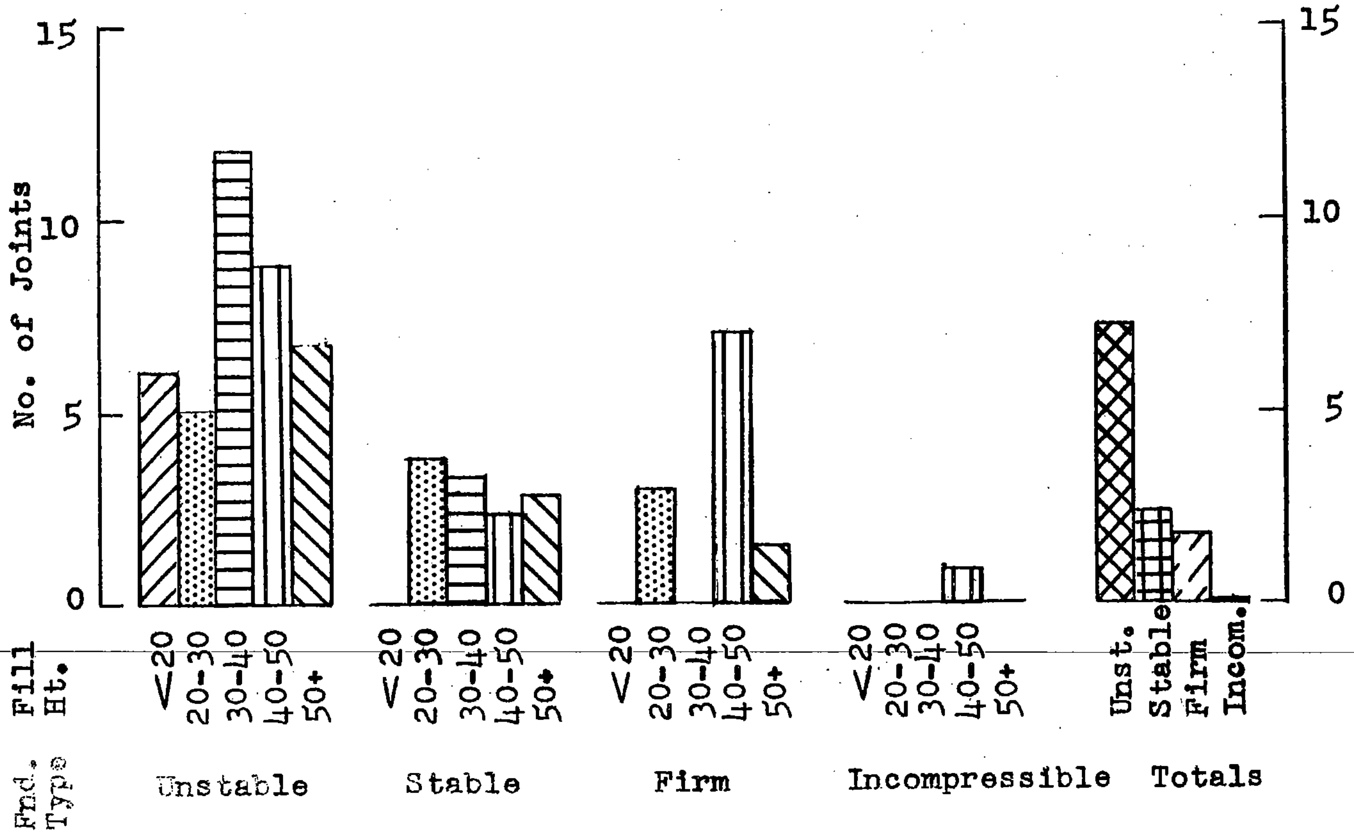
These classes were intended to evaluate as-built conditions. For example, if a shallow depth of low strength soil were removed and replaced with rock the site description would be changed from "Unstable" to "Firm".

Because of the number of variables involved, evaluation of data was difficult. Nevertheless, we feel that the following statements can be made:

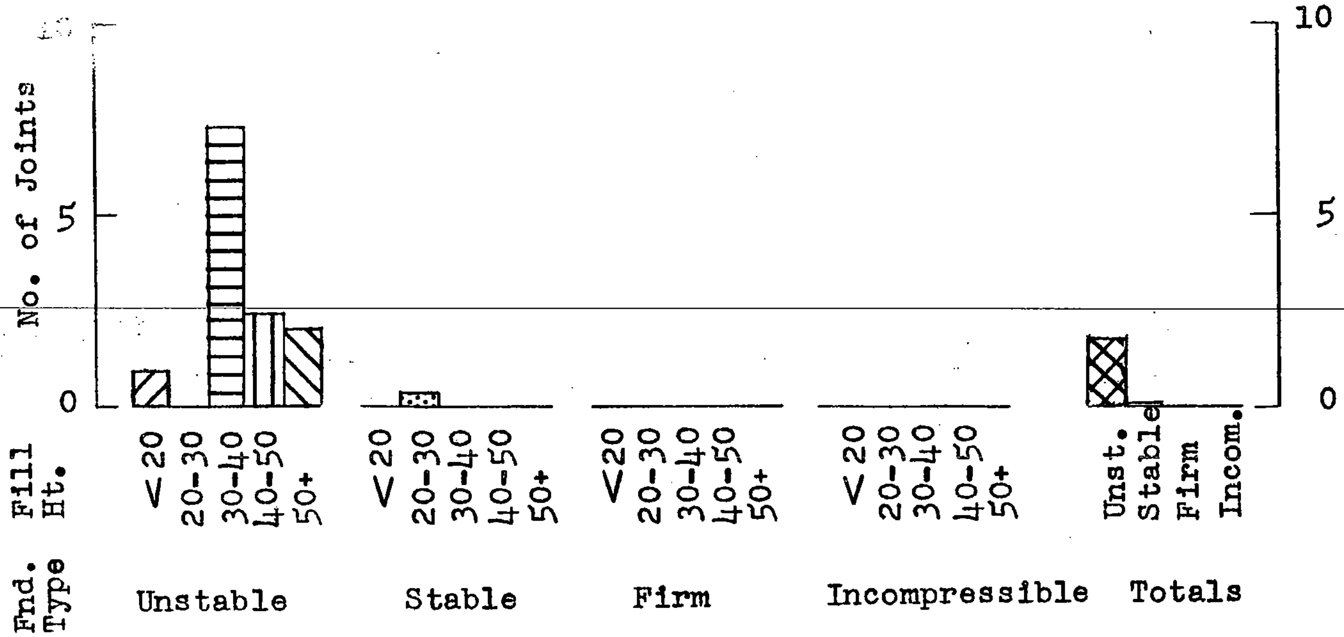


Joint Openings per 100' of Culvert Length.

Figure 4.



Joint Openings > 0.10' per 1000' of Culvert Length



Joint Openings > 0.30' per 1000' of Culvert Length

Figure 5.

1. Concrete Pipe Culverts

None of the reinforced concrete culvert pipes showed openings of more than a few hundredths of a foot. The flow lines of several were sagged, which is a normal expectation unless the culvert is adequately cambered. There were no radial cracks, and but few hairline longitudinal cracks were observed. These were usually seen in pipes under higher fills which were founded on unstable soil.

One type of distress was noted in some of the more erosive soils in the Glaciated Plains, where re-trenching of narrow valleys is occurring. As this progresses headwalls of both box and pipe culverts are undercut. The pipes have little cantilever strength, and the outlet headwalls, along with a joint of pipe, fall away. See Figure 3a. This re-trenching should be noted in soil studies, and erosion control measures used in areas where it is expected to occur. Otherwise culvert distress is inevitable and, worse yet, fill stability is endangered.

2. Shallow Fills

Culverts under fills of less than 10-12 feet showed almost no distress in previous random observations and were eliminated from the study. This indicates that heavy construction equipment has but little effect on the subsequent behaviour of culverts.

3. a. Horizontal Joint Movement

Joint openings were measured at mid-points of walls, ceiling, and floor. Since measurements were from concrete to concrete, .03' was deducted from all openings, representing the thickness of joint material. Joint openings on box culverts are greater on the softer foundations and smaller on the firmer foundations. The range for the average maximum joint opening was 0.20 of a foot of joint opening per 100 lineal feet of box for the unstable foundations as against 0.03 of a foot per 100 feet of a box for the incompressible foundations.

The magnitude of greatest joint opening followed a similar decreasing trend from unstable to incompressible foundations, as demonstrated by Figure 5. In the firm and incompressible foundations, no joint openings exceeding 0.25 feet were found, and in stable foundations only 2 were noted exceeding 0.25 feet. Joint openings exceeding 0.50 feet were noted only for unstable foundations.

The maximum horizontal openings were:

Foundation	No. Joints	Maximum Joint Opening	No. of Joints having opening >0.50'	>0.25'	>.10'
Unstable	136	0.72'	5	15	57
Stable	178	0.34'	0	2	20
Firm	197	0.24'	0	0	13
Incompressible	90	0.15'	0	0	1

No relationship was discovered between fill height and number or size of joint openings. Figures 4 and 5 demonstrate this surprising trend.

In the Ozark region with firm incompressible foundations, the maximum joint opening was 0.04 feet.

b. Vertical Joint Displacement

The pattern of joint movement in the vertical direction is similar to that in the horizontal direction. As shown in Figure 6, this movement ranged from an average of 0.10 of a foot per 100 feet of box for unstable foundation to less than 0.03 of a foot per 100 feet for incompressible foundation.

As foundation conditions improved, there was a sharp decrease in the number of joints with displacement greater than 0.10 of a foot, and an even sharper decrease for those greater than 0.25 foot. The effect of the soil quality on vertical joint movement can be seen in Figure 6 and the following table. Here again, as in the horizontal movements, no relation was found between vertical placement of the joints and height of fills.

Foundation	Maximum Displacement	No. of Joints Opened >0.25'	No. of Joints Opened >0.10'
Unstable	0.38	11	19
Stable	0.15	0	8
Firm	0.15	0	7
Incompressible	0.05	0	0

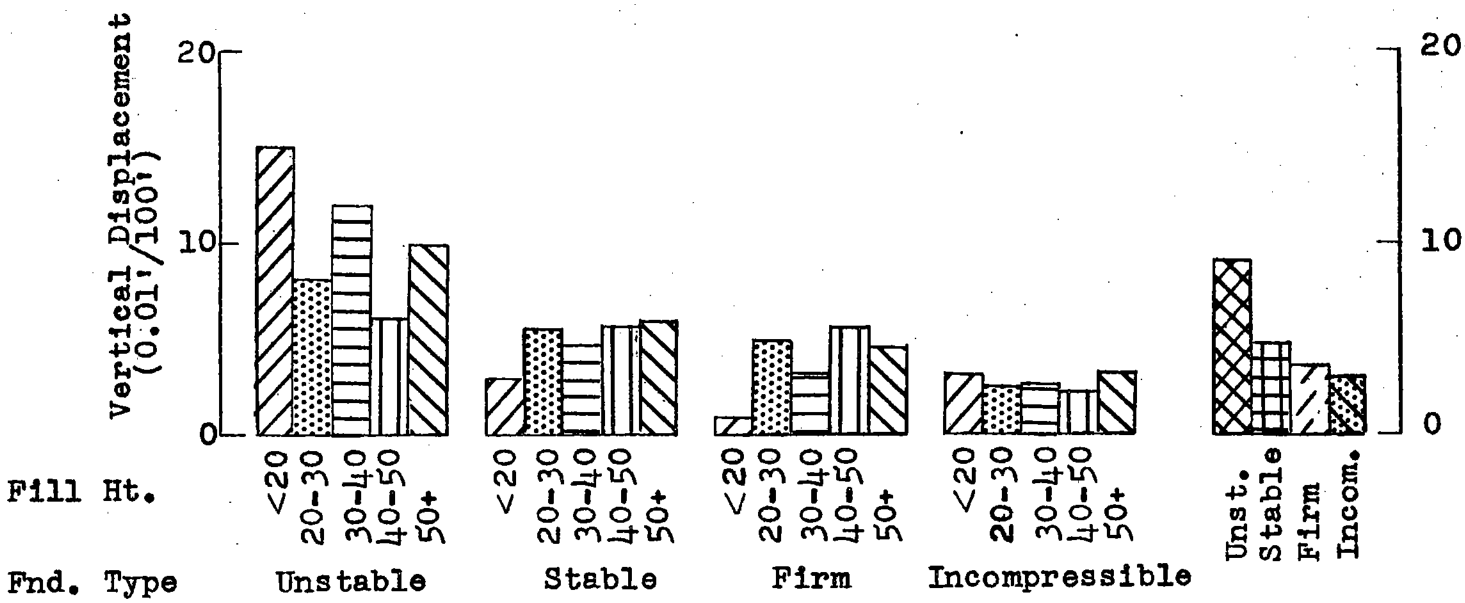


Fig. 6a. Vertical Displacement of Joints per 100' of Culvert Length

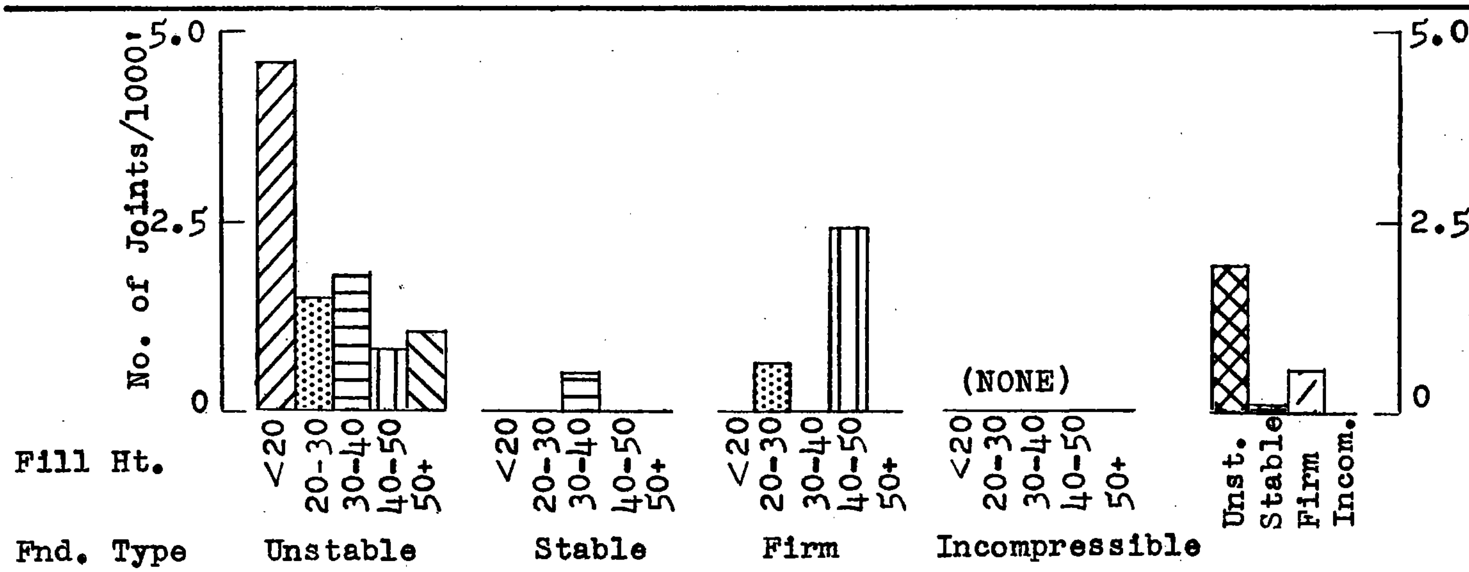


Fig. 6b. No. of Joints per 1000' with Vertical Displacement > 0.10'

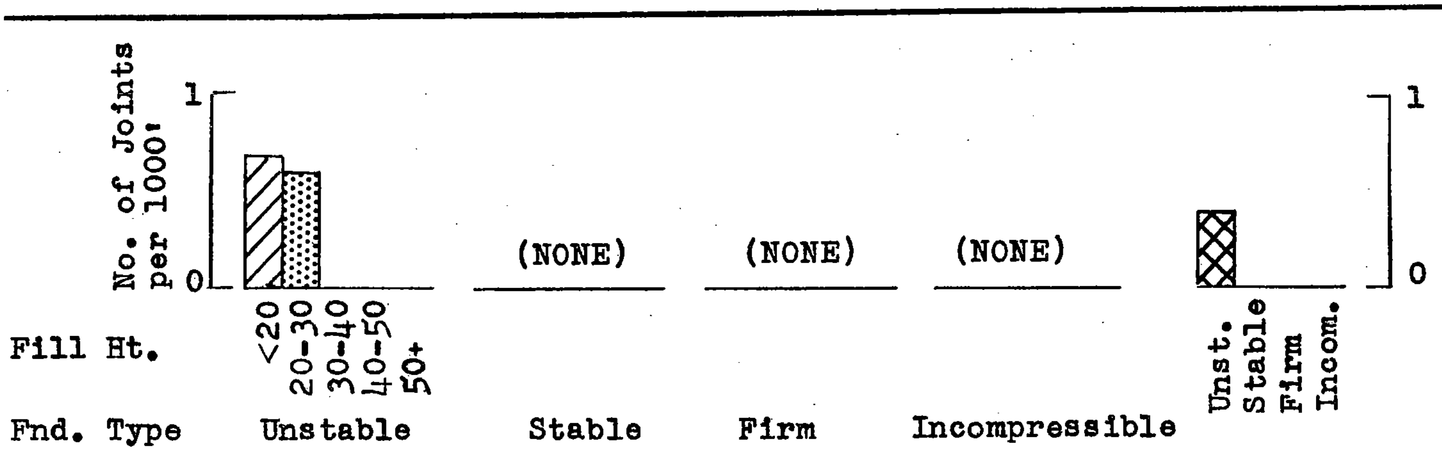


Fig. 6c. No. of Joints per 1000' with Vertical Displacement > 0.30'

Figure 6.

Theoretically, on a uniform foundation both sides of a joint should settle the same and there should be no vertical displacement. The fact that joint displacement exists points up one of the major problems of culvert foundations— unequal support along the barrel coupled with unequal loading from toe of slope to shoulder of roadway.

In the Ozark area, no displacements exceeded 0.25 feet and 3 were measured at 0.10 feet.

4. Cracks

In this survey cracks are tabulated by these magnitudes:

a. All visible cracks, including many of hairline dimensions. Cracks less than 1/8" width are regarded as innocuous to hydraulics and structural life.

b. Cracks exceeding 1/8" width.

c. Cracks exceeding 1/2" width.

The graphs of numbers of panel cracks per 100' of culvert length, Figure 7, shows:

	TYPE OF FOUNDATION			
	Unstable	Stable	Firm	Incompressible
Total Cracks/100'	4.9	2.8	2.3	3.3
Cracks >1/8"/100'	1.8	1.0	0.5	1.3
Cracks >1/2"/100'	0.4	0.3	0.0+	0.0

These data indicate that cracks less than 1/2" decrease with improvement of soil type except for incompressible foundations.

It is the cracks greater than 1/2" that can be classified as structural failures due to foundation conditions. It is in the 1/2" and larger cracks that the reinforcing steel is broken. This can lead to total concrete destruction in time. It is through these larger cracks that dirt infiltrates, creating loss of support to both fill and culvert.

The average length between joints in a culvert section is 40+ feet. For this study a section is made up of four panels, the two walls, floor and ceiling.

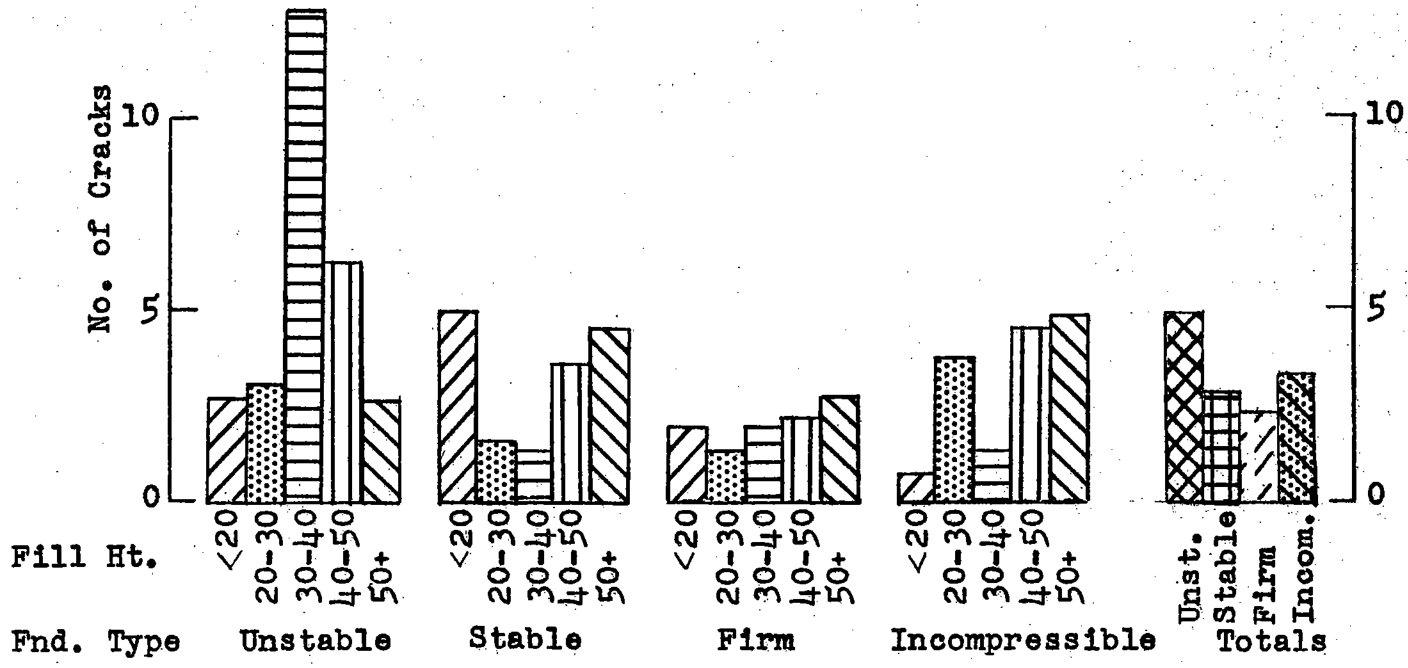


Fig. 7a. Total Cracking per 100' of Culvert Length

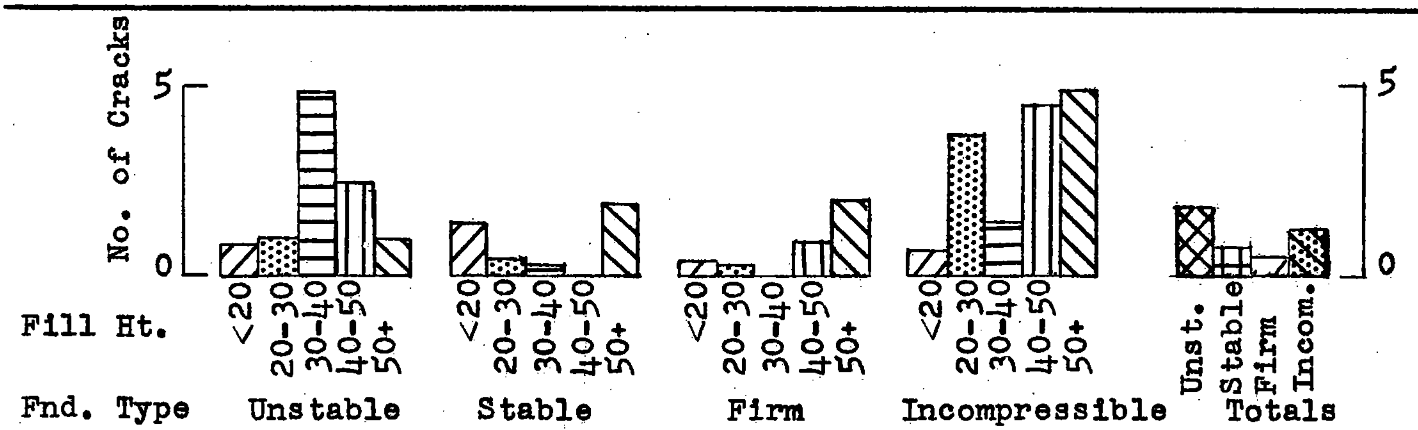


Fig. 7b. Cracks >1/8" per 100' of Culvert Length

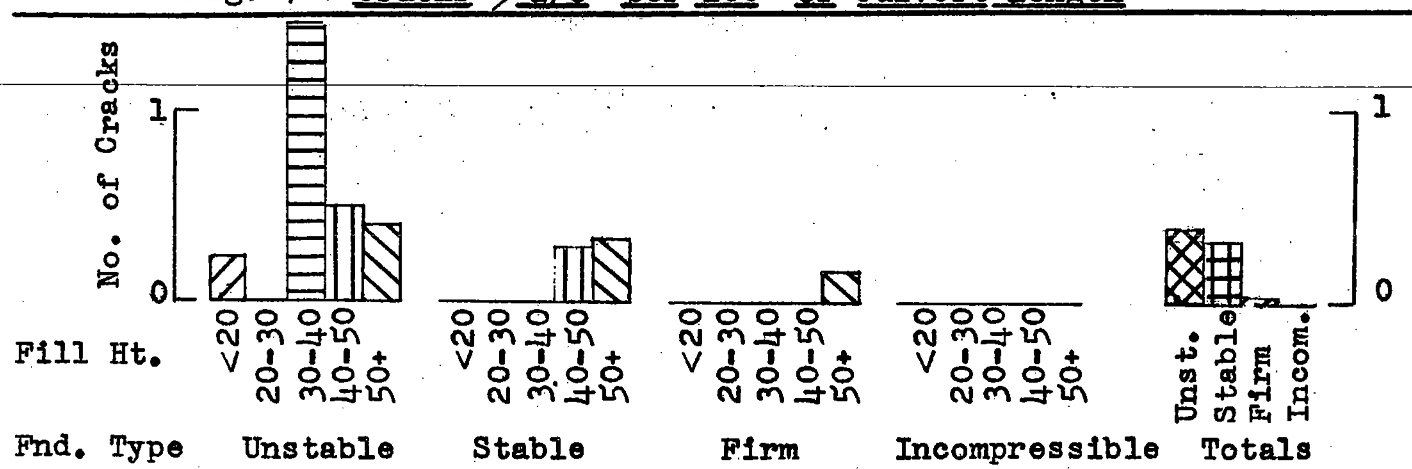


Fig. 7c. Cracks >1/2" per 100' of Culvert Length

Figure 7.

29% of the panels were cracked in culverts on unstable foundations, and 14% of those on firm foundations. Oddly enough, the data for culverts on incompressible foundations show 30% of the panels cracked, a greater percentage than either of the above classes with presumably poorer foundation material. This is thought to be caused by unequal support with points of totally incompressible material. This parallels findings for rigid pavements in rock cuts.

In considering the location of cracks with respect to the joints, the following was found (See Figure 8):

a. Most of the cracks occur more than 15 feet from a joint.

b. Cracks less than 5 feet from a joint are few, regardless of fill height or foundation. In studying the few culverts in which collared joints were used, large cracks in the barrel within the width of the collar were often seen, see Figure 2b. So few collared culverts have been constructed up to this time that no separate statistical data could be tabulated. This barrel cracking within the collar limits suggests that thicker felting between the culvert and the collar should be used.

c. There is no apparent relation between fill height and distance from cracks to the nearest joint.

d. The class of foundation material and the distance from cracks to the nearest joint cannot be directly correlated.

The frequency of small cracks decreases from unstable foundation to firm, with an increase for incompressible material.

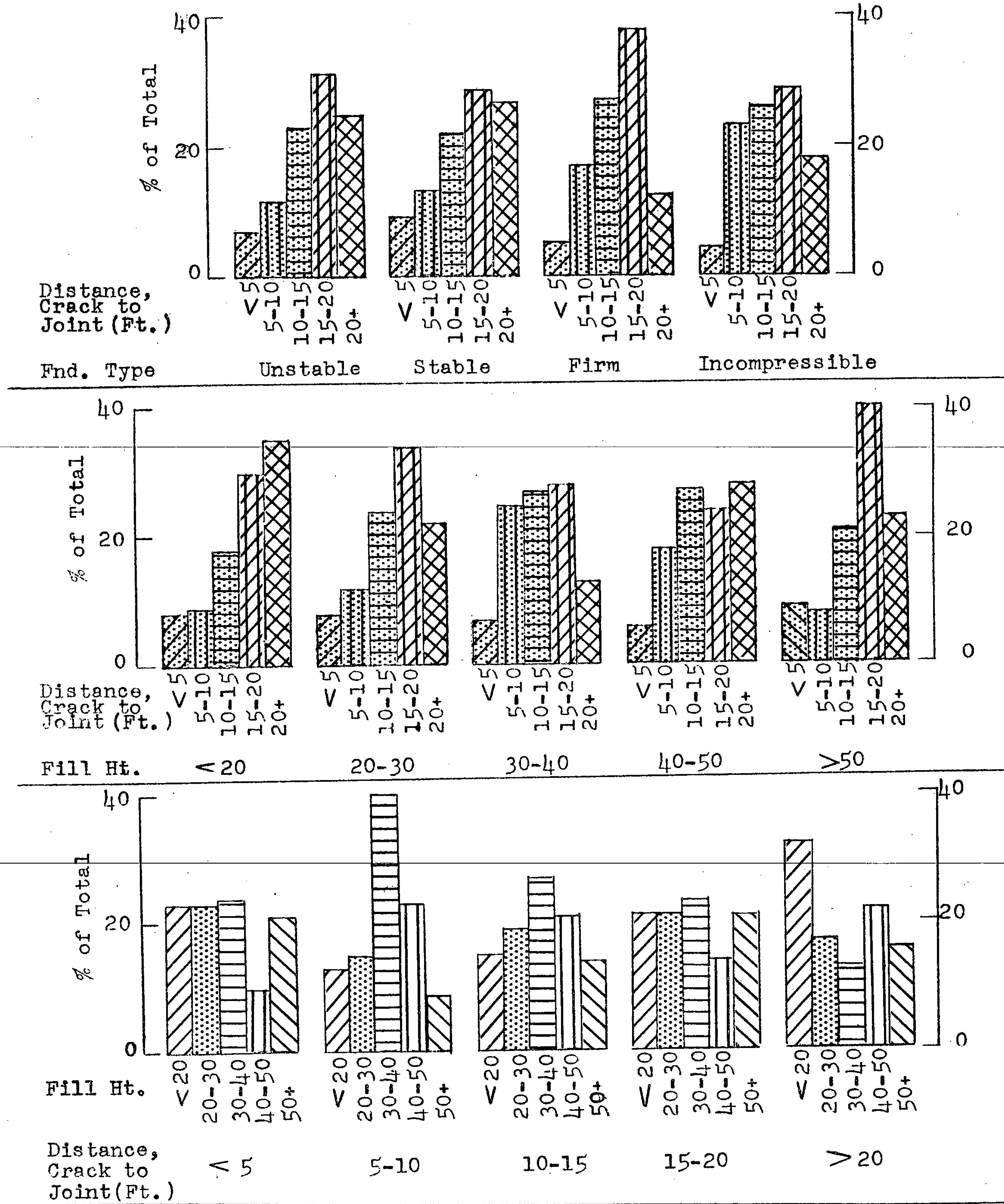
5. <u>Miscellaneous Observations from the Summer Study:</u>

Several things were observed as part of this study:

a. Voids from several tenths to more than 6 feet in depth were measured back of joints or cracks at 15% of the box culverts.

b. It is not known whether voids exist around pipe culverts because no joint openings or cracks were found that were large enough to permit such observations.

c. Dirt infiltration was observed on several box structures. Repeated observations indicate that, at open joints or cracks, this may recur periodically. That is, soil infiltrates, is flushed away, more soil accumulates and is later removed. This process may be repeated numerous times and it is believed that all the structures with large voids have been infiltrated previously.



Distances from Cracks to Nearest Joints

Figure 8.

d. Sag was observed in 30% of the box culverts, ranging from several tenths to more than a foot.

e. Sag was observed in 23% of the pipe culverts. An estimate of sag for both culvert types was determined by observation, since it was impractical to run levels through some 200 structures.

Page 22 missing from original text

After this, the remaining three structures were redesigned with collars and built by change order. A collar is merely a band of reinforced concrete around the joint. It is some 2 feet wide and 9 inches thick, centered on the joint and separated from the box by 55# roofing felt. It gives the effect of sliding bell and spigot.

The 3 collared box structures settled the estimated 2 to 2 1/2 feet, cracked considerably and several joints opened up to 0.3 foot. No dirt has infiltrated, however, and the flow line is good.

Several concrete pipe culverts placed over compressible soils showed sag of flow line but none exhibited opening of joints exceeding 0.02 foot. One pipe of particular interest is 348 feet long, under 45 feet of fill over deep compressible silt. It was cambered 2.6 feet, settled about 2.4 feet, has no radial cracks, 57 longitudinal hairline cracks, and joint openings of .01 to .02 foot under the roadway. Box culverts nearby showed radial cracking up to 1/2 inch and the 0.3 foot joint opening mentioned previously.

Rt 169, Worth County

Again, sequential investigation indicated sites of concern. At all but 5 sites, the structures could be moved to stable soil by using flumes and energy dissipators. See Figure 3c.

Preliminary observations and tests indicated that settlement of 2± feet could be anticipated at four sites. The four "experimental structures" designed for these locations, and their performance are described as follows:

1. A 60-inch corrugated metal pipe, under 23 feet of fill and cambered 2 feet, see Figure 3d, has given excellent performance. This was the second corrugated metal pipe structure built in Missouri in the primary system. Report No. 23 of the National Corrugated Metal Pipe Assn. presents details of this installation. The flow line is excellent.

2. A 7' x 6' box, under 45" fill, cambered 2 feet and built with collars, has opened up to 0.65 foot at several joints with no dirt infiltration. It has not displaced vertically at the joints and has 7 radial cracks, one of which, near the center, is 1 1/4 inches and has broken the longitudinal steel.

3. A 5' x 5' box of standard design under a 36 foot fill and cambered 2 feet has opened 0.5 foot at several joints. It has 3 radial cracks with one 1/2 inch crack near the shoulder (steel broken). The joints under the roadway are displaced vertically about 0.1 foot.

4. A 5' x 5' box, under a 33 foot fill, was cambered 2 feet with continuous reinforcing in the floor. The joints have not opened or displaced vertically. There are numerous small cracks in all panels (about 70 in the floor).

These structures have settled much as predicted and the flow lines are good. While settlement characteristics were quite similar, shearing stresses and rates of loading were different. The collared structure is under heavier fill and was loaded much faster, which we feel accounts for its greater elongation.

Rt I-44, St. Louis

The structures here were all built on relatively incompressible foundations of the Ozark region. No sag has been observed. No undue opening or displacement has been observed. Cracking has been limited to 1 to 15 hairlines per structure except for two structures. One structure has 41 hairline cracks in the side walls.

All concrete pipe culverts were in excellent condition except for slight sag.

Corrugated Metal Pipe Performance

Materials Division personnel from District 7, which is in the southwest part of Missouri, made a study of corrugated metal pipes in that area. An attempt was made to correlate pipe performance and life with geologic conditions at the various locations. Formations from both Mississippian and Pennsylvanian systems were involved.

Of pipes rougly 20-25 years old, 65% of those in Pennsylvanian areas were partially or completely destroyed by rusting. Only 15% of the pipes of the same approximate age in Mississippian areas were badly rusted. Members of the Cherokee Group of the Pennsylvanian were particularly associated with poor performance. Since shale is abundant in the Cherokee Group, which is usually identified in Missouri in areas of coal production, it was thought that acidic surface water might be the cause of the early destruction of the pipes in Pennsylvanian areas. Determination of pH values did not verify this assumption. Pipes subjected to continuous attack by organic acids in the swamp water of southeast Missouri have ordinarily resisted exceptional deterioration and lasted their full expected life.

The apparent correlation between geologic formation and metal culvert life has not been verified in other Missouri areas where coal occurs, but the merits of such classification are recognized.

Conculsions

As a result of this study of culvert performance, we believe that several things have been discovered, a few discounted, many confirmed, and a few strongly indicated.

Seemingly the following conclusions are justified:

1. The extent of distress is much less than anticipated or rumored.

2. Study indicates cracking and displacement is influenced by foundation conditions and fill height is of minor significance. This survey did not include fill heights less than 10 feet, except for cursory examination. It probably would be advisable to make a more careful examination of sites with fills or less than 10 feet.

3. For box culverts built to the structural requirements of Missouri, almost all distress can be associated with foundation inadequacies.

4. On certain types of foundation soil, settlement cannot be prevented. Both its magnitude and the time required can be predicted with reasonable accuracy.

5. Settlement estimates can enable an accurate camber line to be laid. Camber, based on settlement estimates, should be considered individually for each structure.

6. With foundation conditions and fill heights encountered in this study, a proper camber seems to be all that is required for pipe culverts.

7. In areas where large settlements are expected and small pipes or boxes satisfy drainage requirements, overdesign of the size of culvert seems advisable. This allows some tolerance of settlement estimates, particularly on nonuniform foundation soils, and the same time does not impede the flow.

8. Hairline longitudinal cracks were found in some concrete pipe culverts under high fills on unstable foundation. This indicates either higher strength pipe or beneficiation of foundation is desirable at critical locations.

9. The short joint spacing of concrete pipe culverts seemingly distributes joints so that they do not open to permit infiltration.

10. The performance of short sectioned concrete pipe, as compared to that of box culverts, indicates less distress would occur if shorter sections were used in box structures.

11. Collars on box culverts are effective in preventing dirt infiltration. Although they are not designed to restrain vertical joint displacement, no such movement has been observed where collars have been used. No increase in unit price for concrete in collars has yet been noted, but contractors are beginning to associate the use of collars with muddy working conditions, so some increase might be anticipated.

12. Joint displacement, both vertical and horizontal, and cracking correlates closely to foundation conditions. Eliminating consideration of hairline cracks, almost no distress was noted on firm or incompressible foundations.

13. With large cracks occurring in the barrel of culverts within the limits of the collars, Figure 2b, it seems that thicker mastic between the barrels and collars should be used. Plans now call for one layer of 55 pound felt. For estimated settlements exceeding 1 1/2 feet, mastic material one inch in thickness should tend to prevent this cracking.

14. The one culvert studied with heavy continuous reinforcing in the floor is performing satisfactorily except for numerous hairline cracks.

15. The three corrugated metal pipes studied were performing satisfactorily.

16. It is possible to set up a design approach to reduce or eliminate adverse performance. Such a procedure is described in the final section of this report.

Design Approach

Our experience leads us to believe that the steps below offer a sound approach to culvert design.

1. Every route with fills exceeding 10 feet should have culvert sites investigated first by:

   a. Inspection of aerial photos, when feasible.

   b. Application of local geological and pedological knowledge. These first two steps can eliminate most of the later work.

2. Remaining sites should be investigated as thoroughly as required. Normal procedure at all sites should include descriptive logging of material brought up by flight augers, and examination and testing of split tube samples.

The samples should be evaluated for grain size, consistency limits, and natural moisture, as well as to determine the foundation profile.

For saturated cohesive soils the settlement can be estimated from the so-called Skempton equation of

$$\text{Settlement} = H\ [\ (.007 \text{ to } .009)\ (LL - 10)\ ]\ \left(\frac{1}{1+e}\right)\ \left(\log \frac{P_2}{P_1}\right) \quad *$$

where H = thickness of soil layer, e = void ratio, LL = liquid limit, P_1 = initial pressure in soil, and P_2 = pressure after construction. Some judgment is required in estimating the ratio of natural moisture to liquid limit, for, obviously, a soil with low natural moisture will settle less than one with a high water content.

The consistency index, $\frac{LL - Wn}{LL - PL}$ should be checked. If the consistency index is near zero, settlement values as computed above are quite valid, and instability from plastic flow is likely. As the consistency index approaches or exceeds 1, settlements are less and the stability increases. Precise limits cannot be set out, for both settlement and stability are functions of foundation depth, fill height, and side slope design, as well as quality *per se* of soil.

3. If the results of the flight auger and split spoon investigation indicate settlements expected to exceed 0.75, and probable lateral instability, undisturbed samples should be obtained and tested. Interpretation of these soil mechanics tests should provide a reasonable estimate of settlement, time, and stability, using established techniques.

4. Stability analyses as made by most conventional methods are not conservative enough to prevent lateral spreading. These are total stability methods and do not guard against local overstress. Shearing strength vs. local shearing stress should be checked.

5. If settlements exceed 0.75 foot and zones of localized overstress are noted, special measures should be considered as:

a. Use of collars for box culverts.

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* All notations designated in ASTM, *Procedures for Testing Soils.*

b. Moving structure if possible. This may be either horizontally to suitable foundation or vertically to reduce load. Both of these usually involve multiple complications. Hydraulics is the only reason for the culvert, and movement of location should be such that it does not disrupt flow. Vertical movement usually involves ponding water upstream, and construction of flumes with energy dissipators downstream—the embankment thus is changed from a roadway fill to an earth dam, with all accompanying problems of dam design.

c. Mucking out and backfilling is most effective, if excessive excavation and waste can be avoided. Removal of even a part of the unstable soil, and replacement with firm backfill reduces settlement, as well as providing a more suitable working platform for culvert construction.

d. Stage construction or vertical sand drains may be suggested, to provide build-up of foundation shear strength.

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ENGINEERING GEOLOGY OPERATIONS IN THE TEXAS HIGHWAY DEPARTMENT

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Highway Design Division
Texas Highway Department

We of the geology sections of the highway design division and bridge division in the Austin office and the representatives from the district offices throughout the Texas Highway Department welcome the opportunity to visit with our distinguished guests assembled here today and discuss a subject near and dear to us--namely highway geology. I thought perhaps you might like to know the general status of the geology operations in the Texas Highway Department--how we got started in it, what we are doing in it today, and what some of our plans are for the future. There may not be much new here from a scientific point of view, but we can and should compare notes on our methods of operations and applications. Developments in this area are of considerable importance to the relatively new and rapidly expanding field of highway geology.

Engineers of the highway design division of the Texas Highway Department became interested in the application of geology to highway engineering problems following the close of World War II. At that time the Highway Department was faced with the task of modernizing and rehabilitating the existing system of roadways, which had suffered heavily during the war years, and expanding that system to take care of the ever increasing demands being placed upon it. Highway design and research engineers were making bold new approaches to many of the old design problems and practices.

Research studies on freeze and thaw damage in the late 1940's gave good indications that the problem in certain parts of the state had a rather consistent relationship to the geologic unit associated with the highway, however, it was also learned that all the suspected damage could not be blamed entirely on frost penetration. As a result, the design division employed a geologist to work on pavement behavior studies and other design problems and projects related to geology. Many facts concerning the indicated relationship were quickly established, but now, fifteen years later, work is still being done on some of its many ramifications.

The geologist started the program, which is basically the same today, by preparing generalized geologic engineering reports and maps, by making elementary geology lectures at the various highway engineering meetings, and by assisting in the location of base material sources. The program was broadened into the investigation of ground water problems, the correlation of surface geology along the proposed highway routes and the correlation of subsurface geology where foundation studies were being made for roadways and structures.

Geologists are presently employed by approximately one half of the twenty-five districts, which make up the Texas Highway Department, in addition to geologists employed in two of the headquarters divisions. The geologists employed by the headquarters divisions are available as staff personnel to assist those districts which do not employ a geologist and to act as consultants in districts which do have a geologist in their immediate organization.

The district engineer determines the requirements for a geologist in his district. If he feels a full time geologist is justified, he has the authority to employ one. If it is his judgement the work can be adequately handled by the staff geologists available to him from the Austin office, then it would not be incumbent on him to employ one full time. The problem in making this determination is similar to that in many other areas where the work load varies considerably with the availability of project funds.

The staff geologists in the Austin office spend a major portion of their time in the development of two types of reports and associated maps for use by field geologists and engineers. One is a broad report for general reference and the other a specific problem study such as material site investigations. Usually the broad area report will provide the basic geology for a complete district, but may in some cases be restricted to one or two counties. Normally, in starting a broad area report, work maps are first prepared. This is accomplished by collecting such existing geologic maps and reports as may be available and transferring the data to highway planning survey maps which have a scale of one inch equals two miles. The resulting maps have certain accepted accuracy limitations, but meet our need of a general type work map. We now have maps of this type completed on one hundred and forty-six of the two hundred and fifty-four counties in the state. Because of the limited accuracy of the maps, we have been reluctant to release them for use outside the Department.

In the development of these maps from existing data, we are fortunate to have available in Austin the files and libraries of the Bureau of Economic Geology, the Geology Department at the University of Texas, the State Board of Water Engineers and agencies of the Federal Government. We also have an excellent working arrangement with the Department of Geology and Engineering Library here at A. & M. College.

After map preparation is completed, report preparation follows somewhat in the same manner. The object of these reports is to provide the engineer with geologic background material covering his area which is readable and he can understand without a complete review of his college courses in geology. To do this, we eliminate as many pure geologic terms as possible and practically all paleontology. The introduction of the report gives a broad explanation of the area geology and regional physiography for the identified district or specific counties of the district. A brief geologic history is of academic interest to the engineer and it helps him to understand the lithology and physical development of geologic units he is working with.

Information of this type often sparks an interest which grows into a more comprehensive study of geology by the engineer and hence another working tool. The main body of the report contains detailed descriptions of the physical characteristics of each geologic unit, the physiography of the area with particular emphasis on the surface expressions to be expected from specific units, and the soil and vegetation usually associated with each unit. Possible sources of highway construction materials are described, if such exists, giving thickness, dip, strike and areas of expected outcrop. The report is completed with a glossary and bibliography.

The second type of report and associated maps is the end product of a specific problem study. These studies first require a determination of the exact nature of the problem. It is our experience that this can be accomplished best through a conference between the project engineers and the geologist. The problem of acceptable base material sources within an economic distance of a proposed roadway project is the most common. For these investigations, we establish the project type, project length, maximum haul distances and geographic limitations during the conference. The project type and length will tell us which materials will be acceptable and the quantities needed. Maximum haul distances and geographic limitations of the investigation are usually a matter of economics. With the control area established, the geologist can put to use the geology work map developed on the highway planning survey base maps and the general area report. The geologic units which are probable material producers are once again studied, the aerial photography is examined by stereoscope and, if feasible, additional studies are made on the Kelsh plotter. Investigations made previously in the immediate area are studied to determine how much is already known about the area. Maps are then produced which have a scale of one inch to one thousand or one inch to two thousand feet showing the geology and planimetric features. At this point the geologist will go to the field, walk the outcrops and make corrections to the maps as required. The geologist is then ready to start with the subsurface exploration using core drilling equipment.

Wet and dry barrel sampling can be carried out up to an excess of one hundred foot depths. Once an area appears to be a probable material source, large eighteen-inch diameter cores are taken and carefully laid out to insure accuracy in logging. The depth of test holes of the large cores are limited by the economics of the operation. The value of having eighteen-inch cores has proved itself many times. When drilling with a six- or eight-inch bit the material is often so badly broken and pulverized it is often very difficult to determine exactly what the original material was, especially in borderline cases. If circulating water is used, all sand and clay seams are washed out, which gives an erroneous picture of the strata by the log. If an eighteen-inch hole is drilled without circulating water, the cores can be measured as they are logged, and the hole also measured. This measurement is made to the inch and an exact description for that portion of the hole entered on the log. A core of this size provides a sufficient quantity of material in its more natural state to be examined and described. Representative samples for laboratory testing can be determined and the desired quantity obtained. If the cores do not give a clear picture of the strata, it is possible for a man to go down in an eighteen-inch diameter hole and examine the beds that are too deep to be seen from the surface. Usually two or three large holes strategically located about an area will provide enough information so the correlation of small cores taken will accurately establish the limits and quantity of material available.

The recovered samples, after field examination, are moved to the district laboratory where they are prepared and subjected to such tests as may be necessary to determine the quality of the material. When it has been determined that the particular material tested will meet specifications, the geologist returns to the test area and proceeds to determine the quality of base material by a systematic drilling program. After the necessary volume of acceptable material has been located, the final report and maps are prepared.

This geologic-engineering report for the investigation will include the geologic age of the materials to be produced, the site location and haul distance to the project, estimated quantity of stripping, description of possible quarrying methods and the expected effects of excavation on the nearby drainage systems. A large scale map or maps showing outcrop patterns of the source formation or member, and the site location and description of present land use will be included with orthographic projections which show the estimated water supply system which fell beneath the embankment.

The report covering the investigation was completed in as much detail as possible for future reference in the event something should happen to the water flow. It was established that the spring opening was very near the level of the water table and should this water table be lowered by a small amount, the spring would not flow. The report is now part of the permanent records of the Highway Department.

As previously stated, a good many of our districts employ their own staff geologists. Where a district employs a geologist, the geologist is associated with the design section either directly or through the soils laboratory. Generally, he is directly under the district laboratory engineer where he works with the materials brought into the laboratory for identification or testing. If the geologist understands the tests, he better understands what the characteristics of the material will need to be to meet the test. This is valuable information when a material search is being made or a highway location is being determined.

When the economics of a location and design are being studied, the geologist works closely with the design engineers. Alternate solutions which may be found in different routes, availability of different materials, or different design details which are considered. Careful evaluation of all the factors and combination of factors includes the pertinent information the geologist can provide.

Although the district geologist is usually associated with design engineering problems, he may have many tasks to perform throughout the district in association with resident engineers or maintenance engineers. The resident engineers will call for assistance during construction problems such as unexpected groundwater and the maintenance engineers will call for help when slides become a factor in roadway cuts.

Subsurface exploration with core drills for structure foundations has been the accepted practice of the Texas Highway Department since the first shop made drill was built in 1938. With the refinement of coring equipment and the need for the most accurate information possible to support the complicated designs for roadways and structures, the problem of accurate drill logs and core interpretations has culminated in a manual which will soon be in the printing stage. This manual has been prepared by the bridge division and will be used as a standard for soil and bed rock classification in connection with core drill operations. It presents a step by step procedure for field classification and logging techniques for better analysis and correlation.

The Texas Highway Department operates twelve units of core drilling equipment which are composed of a drill truck, water-supply truck, and tool truck. Each unit carries approximately 120 feet of drill stem, 30 feet of casing, drilling mud, bits for all types of drilling, and for soft materials, dry barrel or push barrel samplers are used, for hard materials rock bits or Dennison barrel samplers are used with diamond bits available when needed. Each unit is so equipped that it may operate independently with the exception of fuel and periodic maintenance. Normally each core drill unit has an operating crew assigned and is composed of three men. Prior to 1950 only soil classification and identification was obtained from core drilling. At that time the Texas Highway Department developed its own cone penetrometer, this penetrometer varies from others in that it uses a 170-pound weight with a 24-inch drop. It has been used, tested and correlated with soils, bed rock and other testing devices. More recently the Texas Highway Department has started using the in-place vane shear tester and later the miniature vane shear tester. It is our feeling that the vane testing gives good results in areas where neither the triaxial or cone penetrometer test may be satisfactorily applied. The Texas Highway Department recently developed its own belling tool, this was designed for the specific purpose of testing a bell-ability of a material at the same time other drilling exploration was in progress.

Research is in progress to correlate core drill information with the terra-scout refraction seismograph which we hope will expedite such surface exploration. Radiological density and moisture probe, the air driven penetrometer and other new sampling tools are also being studied.

In closing, I would like to comment on the word we have used on our display in the lobby--"Photogrammetric Geology." In the spring of 1958, the Texas Highway Department purchased a sterio-plotter from the Kelsh Instrument Company for the primary purpose of checking photogrammetric maps purchased under contracts. Needless to say we now have five plotters and still contract a large part of our engineering mapping. However, with the development of the use of photogrammetry in various areas of highway engineering with which the geologists are associated, and with the availability of the equipment for experiment, several applications for geology have been developed. By using the Kelsh plotter, horizontal measurements can be made to 1/40 of an inch of the projection scale and vertical measurements to 1/6000 of the flight height of the photography. Much of the photography we are buying enables us to map areas with a vertical accuracy of .2 foot and horizontal accuracy of .5 foot or six inches maximum error. With these accuracies, many things which previously required trips to the field hundreds of miles away can now be done in the office. When topographic maps are made for route studies, generally 1" = 100', we can put the geology directly on the proposed locations for consideration in the selection of the route. Accurate topographic maps of a proposed material source provide better material quantity estimates when correlated with the subsurface information. Dip and strike are readily measurable, even small irregularities in a trend may be detected. Fractures may be separated from faults, particularly where an outcrop may be exposed. It is not always possible to determine which block moved in a certain direction, but the up side can be determined from the down side. Outcrops can be traced in much the same way they are in the field, with a better overall understanding of the exposed facts.

Vertical to horizontal scale exaggeration does not exist as it usually does in the average stereoscope. Slopes have a more realistic appearance. A stereoplotter operator can measure a cross section with as much if not more accuracy in a fraction of the time it would take two men in the field to measure it.

Identification of the beds is limited to the usual rules of photo interpretation. We do not assume that work in the field is not necessary, it is still as important as it ever was. We are only trying to eliminate some of the detail in field work.

The geology section and the photogrammetry section of the highway design division in the Austin office are organized as a joint operation as each complements the other in their service to highway engineering. This is one of the fastest growing operations in the Department and we extend to each of you an invitation to visit us at our location at Camp Hubbard anytime you are in the vicinity of Austin.

GEOLOGY - A VITAL PART OF SUBSURFACE

ENGINEERING IN ILLINOIS

By GORDON R. BENSON
Chairman, State Soils Committee
Illinois Division of Highways

There is a tendency on the part of many practicing engineers to evaluate a structural or an earthwork situation solely on the basis of the borings, the test values and the procedures of soil mechanics. While in many cases this practice has resulted in notable successes, the engineering periodicals often describe dramatic failures in which the engineers failed to employ either a knowledge of precedents, a working knowledge of the geology of the area or both in properly evaluating the meaning of the test values.

The knowledge of precedents is fundamental to the fullest development of the art of foundation engineering. History records many outstanding engineering feats during bygone centuries when the designers labored under the tremendous handicap of little or no specific knowledge of the character of deep underlying materials. They had only the experience of their predecessors supplemented by the sum of their own observations.

During the last century, a new branch of science called soil mechanics has developed at an astonishing rate inspired by the need for procedures to evaluate subsurface materials relative to their stress-strain-time characteristics. The rational procedures of soil mechanics have presented an excellent opportunity and logical basis for preserving essential features of experience. In short, soil mechanics has made it practicable to utilize the considerable fund of precedent already accumulated and still to be obtained. The worth of soil mechanics in this connection cannot be overemphasized.

Geology is as basic to subsurface engineering as is soil mechanics. Possibly its most significant role is to make us aware of the departure from reality in our simplifying assumptions. Whereas the theories and computational procedures of soil mechanics would prove impractical without simplifying assumptions regarding the properties of the subsurface materials, nature is not simple. The geology of a site must be understood before any reasonable assessment can be made of the errors involved in our calculations or predictions. Indeed, in some instances the geologic structure or the result of geologic processes may completely override all considerations of soil mechanics. The nature and orientation of the relict joints in a residual soil may govern the stability of the sides of an excavation for a

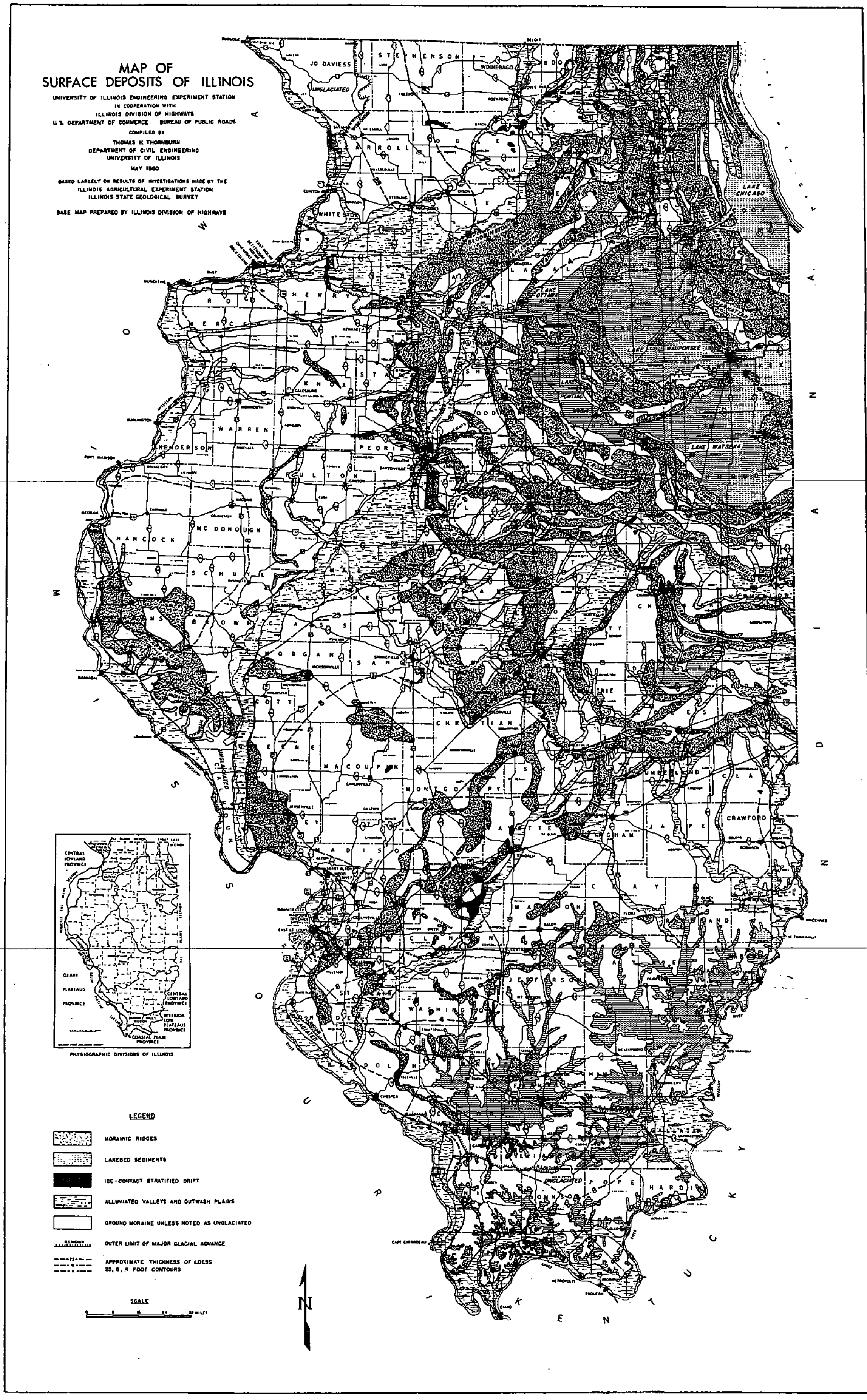
MAP OF
SURFACE DEPOSITS OF ILLINOIS
UNIVERSITY OF ILLINOIS ENGINEERING EXPERIMENT STATION
IN COOPERATION WITH
ILLINOIS DIVISION OF HIGHWAYS
U.S. DEPARTMENT OF COMMERCE BUREAU OF PUBLIC ROADS
COMPILED BY
THOMAS H. THORNBURN
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF ILLINOIS
MAY 1960
BASED LARGELY ON RESULTS OF INVESTIGATIONS MADE BY THE
ILLINOIS AGRICULTURAL EXPERIMENT STATION
ILLINOIS STATE GEOLOGICAL SURVEY
BASE MAP PREPARED BY ILLINOIS DIVISION OF HIGHWAYS
PHYSIOGRAPHIC DIVISIONS OF ILLINOIS
LEGEND
MORAINIC RIDGES
LAKEBED SEDIMENTS
ICE-CONTACT STRATIFIED DRIFT
ALLUVIATED VALLEYS AND OUTWASH PLAINS
GROUND MORAINE UNLESS NOTED AS UNGLACIATED
OUTER LIMIT OF MAJOR GLACIAL ADVANCE
APPROXIMATE THICKNESS OF LOESS
25, 6, 4 FOOT CONTOURS
SCALE
N

Figure 1.

foundation, quite irrespective of the properties of the soil between the joints and quite at variance with the predictions of theory based upon the assumptions of homogeneity.

Geology, also, like soil mechanics, provides a means for correlating our experience, but on a regional or physiographic basis. Regional studies of foundation conditions have proved very useful to the practicing engineer. They pertain to areas in which experiences should be similar; hence, the conclusions are valid only if the physiographic units have been established on a sound basis of geologic similarity.

Finally, whether we realize it or not, every interpretation of results of a test boring and every interpolation between two borings is an exercise in Geology. If carried out without regard to geologic principles, the result may be erroneous or even ridiculous. Conversely, if done with a keen perception of local geologic conditions, the results are likely to be much more reliable. It is hardly necessary to labor the point that intelligent subsurface exploration is impossible without a working knowledge of Geology.

In order to facilitate a comprehension of the subsurface conditions in Illinois, a few remarks relative to the Pliestocene Geology seem to be warranted. The history of subsurface deposits in Illinois is for all practical intents a story of continental glaciation and weathering. As the glaciers expanded from the areas of the North across Northern Illinois and almost to the extreme Southern tip of the State, they brought with them rock and soil material over which they had passed. Many of these materials were deposited directly from the ice, but as the ice melted back, the major rivers such as the Mississippi, Illinois, and Wabash carried vast amounts in suspension and deposited them at varying distances from the ice front. These latter deposits not only filled the main valleys but also affected the tributary valleys. Strong winds coming primarily from the Northwest picked up some of the material exposed in the alluviated valleys and carried it across the adjacent upland in the form of loess. It would be difficult to find a square foot of material in the State in which the surface has not been affected either directly or indirectly by the action of the continental ice sheets.

Although most of the deposits covering the surface of the state have the same general origins, they vary widely in their character depending upon the various factors which influenced their method of deposition. Thus one can find materials varying from the fine-textured clays of the Big Muddy River Valley in the Southern part of the State to the very coarse textured deposits of sand and gravel found in the Fox River Valley in the Northeastern part of the State. Furthermore, subsequent to the time of their deposition, the various deposits have been exposed to climatic agents which have weathered and eroded the surface. This too has contributed to differences in the nature of the soils.

With the expenditure of up to a million dollars per mile of the Interstate Highway System as well as for many other City and Primary Route improvements, over-all

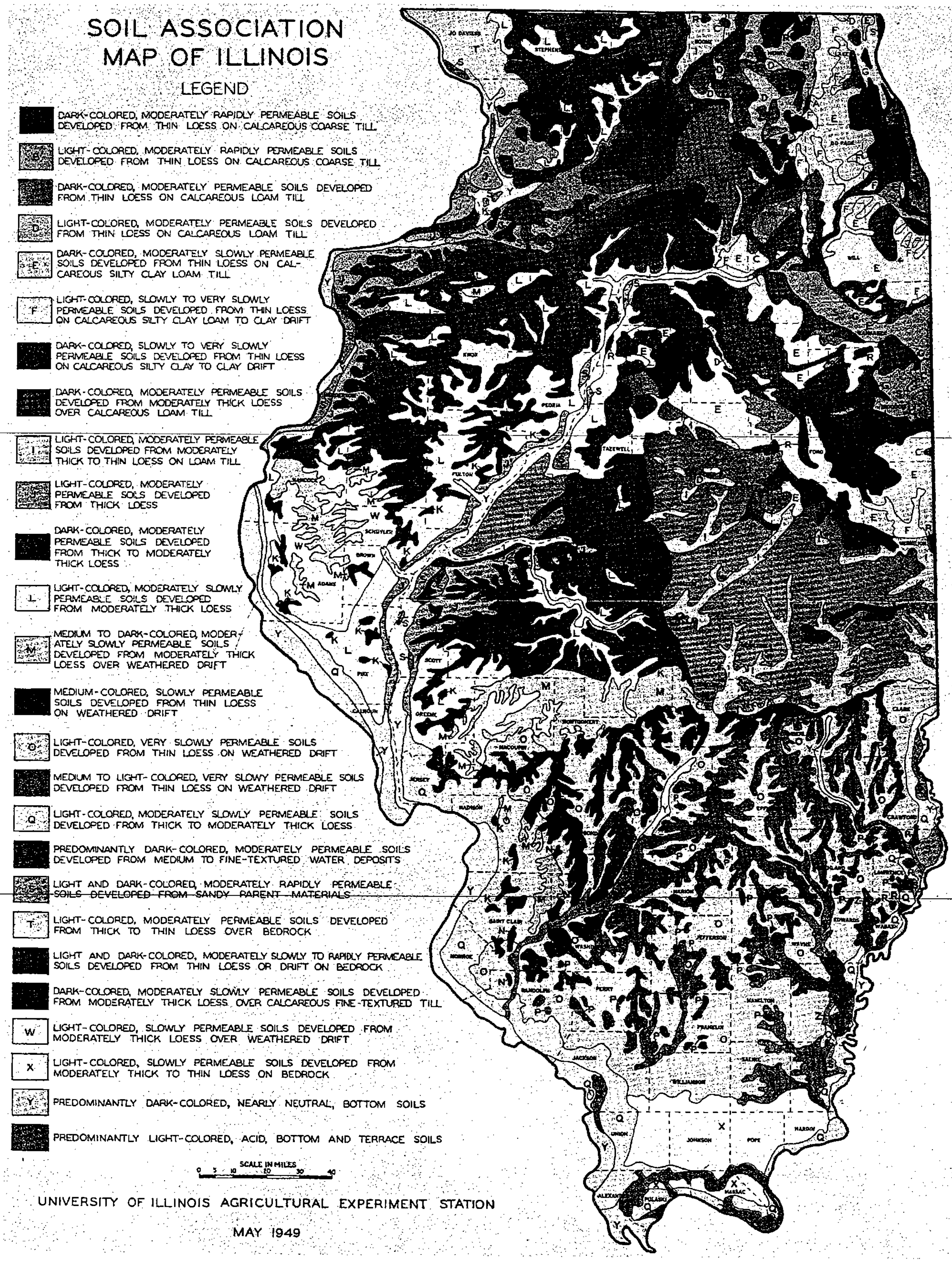
SOIL ASSOCIATION MAP OF ILLINOIS
LEGEND
DARK-COLORED, MODERATELY RAPIDLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS COARSE TILL
LIGHT-COLORED, MODERATELY RAPIDLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS COARSE TILL
DARK-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS LOAM TILL
LIGHT-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS LOAM TILL
DARK-COLORED, MODERATELY SLOWLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS SILTY CLAY LOAM TILL
LIGHT-COLORED, SLOWLY TO VERY SLOWLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS SILTY CLAY LOAM TO CLAY DRIFT
DARK-COLORED, SLOWLY TO VERY SLOWLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON CALCAREOUS SILTY CLAY TO CLAY DRIFT
DARK-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK LOESS OVER CALCAREOUS LOAM TILL
LIGHT-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK TO THIN LOESS ON LOAM TILL
LIGHT-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM THICK LOESS
DARK-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM THICK TO MODERATELY THICK LOESS
LIGHT-COLORED, MODERATELY SLOWLY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK LOESS
MEDIUM TO DARK-COLORED, MODERATELY SLOWLY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK LOESS OVER WEATHERED DRIFT
MEDIUM-COLORED, SLOWLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON WEATHERED DRIFT
LIGHT-COLORED, VERY SLOWLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON WEATHERED DRIFT
MEDIUM TO LIGHT-COLORED, VERY SLOWY PERMEABLE SOILS DEVELOPED FROM THIN LOESS ON WEATHERED DRIFT
LIGHT-COLORED, MODERATELY SLOWLY PERMEABLE SOILS DEVELOPED FROM THICK TO MODERATELY THICK LOESS
PREDOMINANTLY DARK-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM MEDIUM TO FINE-TEXTURED WATER DEPOSITS
LIGHT AND DARK-COLORED, MODERATELY RAPIDLY PERMEABLE SOILS DEVELOPED FROM SANDY PARENT MATERIALS
LIGHT-COLORED, MODERATELY PERMEABLE SOILS DEVELOPED FROM THICK TO THIN LOESS OVER BEDROCK
LIGHT AND DARK-COLORED, MODERATELY SLOWLY TO RAPIDLY PERMEABLE SOILS DEVELOPED FROM THIN LOESS OR DRIFT ON BEDROCK
DARK-COLORED, MODERATELY SLOWLY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK LOESS OVER CALCAREOUS FINE-TEXTURED TILL
LIGHT-COLORED, SLOWLY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK LOESS OVER WEATHERED DRIFT
LIGHT-COLORED, SLOWLY PERMEABLE SOILS DEVELOPED FROM MODERATELY THICK TO THIN LOESS ON BEDROCK
PREDOMINANTLY DARK-COLORED, NEARLY NEUTRAL, BOTTOM SOILS
PREDOMINANTLY LIGHT-COLORED, ACID, BOTTOM AND TERRACE SOILS
SCALE IN MILES
0 5 10 20 30 40
UNIVERSITY OF ILLINOIS AGRICULTURAL EXPERIMENT STATION
MAY 1949

Figure 2.

excellence in highway engineering is now more to be desired than ever. In the vast network of our highway transportation system, the complex pattern of high-speed interchanges and grade separations has necessitated the excavation of cuts, the raising of fill, and erection of structures of an order that was little more than a dream a few decades ago.

New alignments now bypass urban areas deviating from the traveled paths and roadways established in the preceding years which were designed to link population centers directly rather than to afford ease of long distance travel. Not infrequently the new alignments must traverse difficult terrain from the standpoint of highway construction that had been heretofore avoided.

To the highway designer increased understanding of the stress-strain-time characteristics of subsurface materials is imperative. Shear and consolidation properties of both shallow and deep deposits due to the heavy surcharges coincidental to the raising of high embankments is a frequent problem. While these problems have faced the highway engineer for years, their incidence and magnitude is now greatly increased.

The State of Illinois has been quite successful in coping with these engineering problems through the staff of the Illinois Division of Highways and occasional consultant assistance by the University of Illinois and the State Geological Survey. There have been instances, however, when errors have been made in the evaluation of existing conditions. Some of these errors could have been avoided by a better understanding of the Geology of the area.

The record of the design and construction of the Federal-aid Interstate dual bridges over the Vermilion River at Danville is an excellent example of the importance of geology in subsurface engineering.

Due to the weight of the interstate program, a consulting firm had been employed to develop the design of these dual structures. As the designer, the consultant was also responsible for obtaining or arranging for the foundation borings.

When the soils report for the section was reviewed in 1959, the State Soils Committee recognized a potential stability problem on the east side of the river where the approach embankment would rest upon the flood plan of the partially alluviated valley.

Shelby tube samples were requested in the area of the embankment. The samples were tested by the Illinois Division of Highways; an analysis made; and a modified design of the embankment determined. The new design consisted of a variable width berm placed at mid height of the slope and varying from 46 to 56 feet in width with the up slope and down slope at 2 horizontal and 1 vertical.

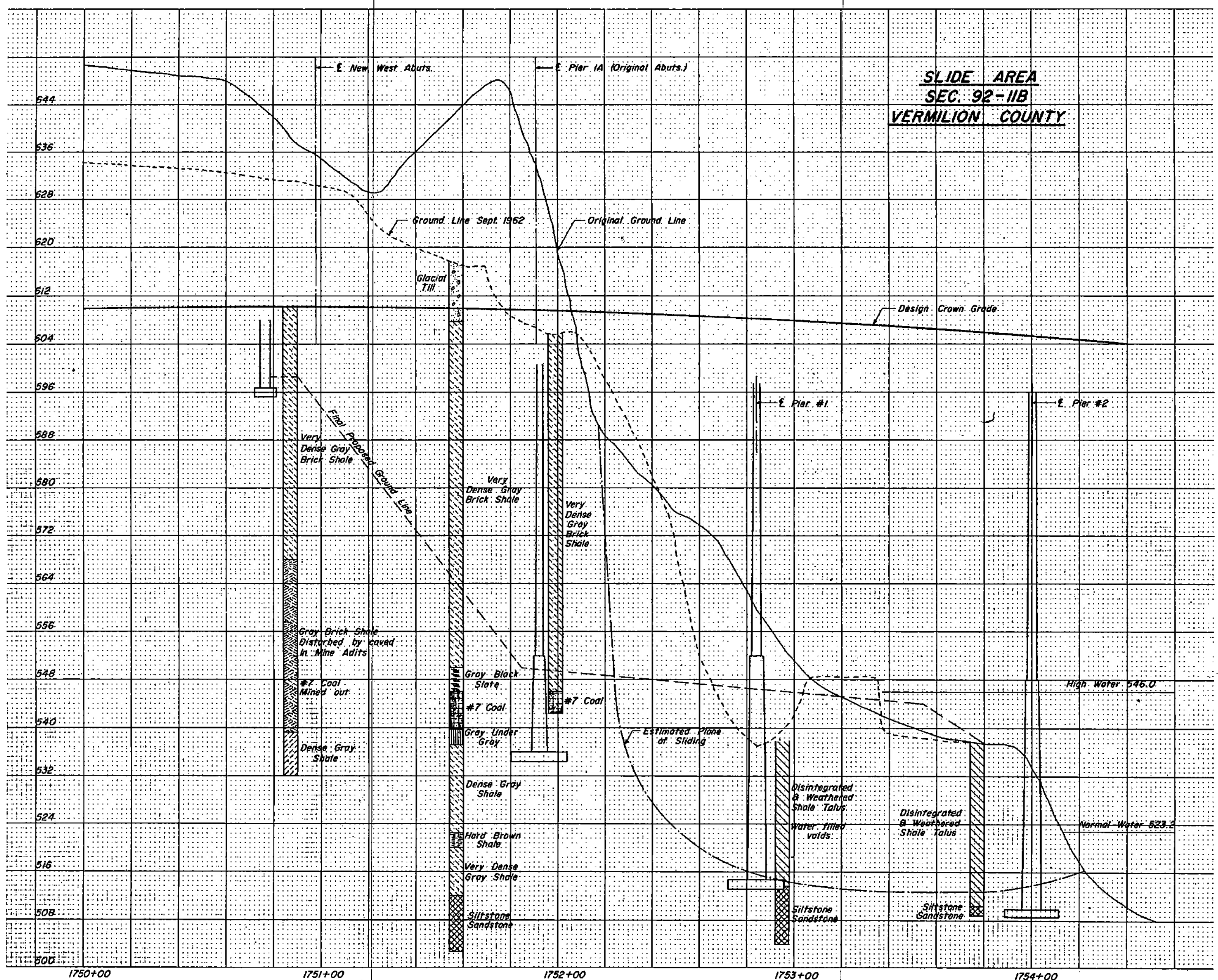
SLIDE AREA
SEC. 92-11B
VERMILION COUNTY
℄ New West Abuts.
℄ Pier 1A (Original Abuts.)
℄ Pier #1
℄ Pier #2
Ground Line Sept. 1962
Original Ground Line
Design Crown Grade
Final Proposed Ground Line
Estimated Plane of Sliding
High Water 546.0
Normal Water 523.2
Glacial Till
Very Dense Gray Brick Shale
Gray Brick Shale Disturbed by caved in Mine Adits
#7 Coal Mined out
Dense Gray Shale
Gray Black Slate
#7 Coal
Gray Under Gray
Hard Brown Shale
Very Dense Gray Shale
Siltstone Sandstone
Disintegrated & Weathered Shale Talus
Water filled voids
644
636
628
620
612
604
596
588
580
572
564
556
548
540
532
524
516
508
500
1750+00
1751+00
1752+00
1753+00
1754+00

Figure 3.

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of the Number 7 coal having been removed from under the bluff by "gopher" mining operations during the early 1930's. Estimates ran as high as 90% of the coal having been recovered.

Inspection of the materials exposed in the excavations for Pier Number 2 of both structures yielded a further insight to the existing subsurface conditions. Rotated masses exposed consisted of repeated overlaps of a downward succession of nodular limestone, thin bony coal, underclay, and river gravel clearly showing the evidence of earlier movements, and the fact that the river had at one time flowed closer to the bluff. It also showed that the flowline was then as now a few feet above the calcareous siltstone-sandstone formation underlying the area. The underlying bedrock down to the siltstone-sandstone formation had moved in a series of rotational masses outward and upward into the river so that the weak bedrock formations above the siltstone-sandstone formation were shoved up, out, and over the river gravel on the original shore.

In order to accomplish a stabilization of the unbalanced sliding condition, it was realized that the upper wallward part of the talus would have to be removed to reduce the surcharge or driving moment of the rotational movement. The top of the talus was to be evened off with gentle slopes toward the river to aid in surface runoff.

It was realized that the recommended reduction of the talus slope would expose the bedrock shale above the Number 7 coal in a steep nearly vertical wall, and that this shale was already comparatively unstable as a result of fracturing and weathering. It was also realized that it would become subject to accelerated deteriorations when thus exposed, with probable additions to the talus, so the bedrock face was to be cut back to a more stable slope. The final decision called for a slope face of 1 1/2 to 1, the addition of another span of 93 feet to each structure, and the design of new abutments in back of the top of the proposed slope. The originally proposed abutments were to be redesigned as piers.

The prior coal removal extended for some distance back under the bluff. Following removal of the coal, the shale above the mine adits collapsed to fill them, as was evident in some portions of the valley wall and some of the test borings. The collapse doubtless occurred shortly, certainly not more than a few years, after the mining was concluded. It may be reasonably assumed that in the subsequent time, the overlying material has been substantially restabilized and that past experience indicates that additional subsidence or consolidation of consequence will not be experienced. The realization that further movement of the slide area exists as a remote possibility, however, prompted placement of the footings for Piers Number 1 and 2 of both structures as well as the new piers at the location of the originally proposed abutments upon the siltstone-sandstone formation under the Number 7 coal. The reasoning was that while any movement would be structurally serious, piers on spread footings would be translated less easily than piers on piles, and that a tilted pier on spread footings would pose less of a problem in any future rehabilitation than piers both tilted in attitude and translated in position as would result if piling were used.

The proposed new abutments would rest upon spread footings founded upon the shale. The borings indicated in excess of 25 feet of intact shale from the footing elevation to the elevation of the material affected by the caving of shale into the abandoned mine adits.

It was further decided that protection of the rock face under the structures was a necessity because of the apparent lack of weatherability of the shale. This protection was accomplished by the employment of a 6-inch paved slope wall.

During the same construction season, another dual structure was being placed over the Vermilion River a few miles to the East. The same consulting firm and boring agency were involved, and again unforeseen difficulties were encountered by the contractor.

Pier Number 1 of both structures according to the contract plans were to be founded at Elevation 210 with spread footings resting upon "fine sand and silt, gray and very dense."

When the contractor began excavating for the footings, rock was encountered at Elevation 519. Inasmuch as the contract established no bid item for rock excavation and force account work was indicated, the State Highway District Office boring unit was pressed into service to determine the extent of the rock in a vertical direction. It was then discovered that an extensive system of alternately bedded sandstone, limestone, and shale was present. With the concurrence of the Bureau of Public Roads, it was decided to raise the footings from Elevation 510 to Elevation 514.

Another instance worthy of mention occurred late in the 1962 construction season. A dual multispan structure over the Des Plaines River at Joliet had been placed under contract, and the excavation for certain of the piers on the east side of the river had started.

The contractor excavated to the contract plan elevation for the footings which were to be founded upon limestone, and encountered only bouldery gravel. Inasmuch as the design footing pressure was 5 tons per square foot, and there was no rapid, accurate, and inexpensive way to determine the allowable load for this material, it was recognized that the footing elevations would have to be lowered.

The layer of bouldery gravel was about 5 feet thick, and was underlain by approximately 5 feet of completely disintegrated limestone. This badly weathered material was also considered to be unsuitable because of its friability since it could easily be reduced to fine sand or silt size particles. A thick bed of hard limestone underlies this weathered material, and it was determined that the pier footings should be placed thereon. By lowering the footings an average of about 11 feet, the pier shafts were sufficiently extended to require redesign of both shafts and footings in some cases.

The Illinois State Geological Survey has over the years prepared and presented a great number of publications relating to the geology of various quadrangles. These bulletins contain considerable specific information, and are of great value in many instances to the practicing engineer. Further, a bulletin has been available for several years relating to the bedrock topography of Illinois, and the information has generally proven quite accurate. It is readily apparent that the individuals responsible for the logging of the borings for these examples just cited had not availed themselves of these helpful publications.

It is also quite obvious that applying an engineering classification premised upon a visual inspection of the destruction products of drilling without cognizance or comprehension of the geological origin can be grossly misleading. Certainly if a knowledge of geology is important to the engineer interpretating boring data it is equally important to the technical man responsible for the compilation of data in the field on which design will ultimately be based.

As a final case history, the construction record of a dual 168-inch corrugated metal pipe culvert beneath a 50-foot embankment on Federal-aid Interstate 80 seems pertinent.

During the 1961 construction season, the contractor had placed the dual 168-inch pipes and the initial 35 feet of embankment. In the summer of 1962, the grading was completed in preparation for paving operations.

The day after the embankment had reached design height, members of the contractor's staff noticed that the north edge of the embankment appeared to be low. Upon further investigation, it was discovered that a movement of the embankment was beginning to show up on the north side of the embankment slope. A check of the pipes revealed that the tops were buckling downward about 80 feet in from the downstream end. Within the period of the next 5 days both pipes completely collapsed for about 20 feet of their length. The Soils Committe was called upon to study the situation and to make recommendations for remedial measures. In their review of the design borings, it was noted that the soil cover in the valley was generally quite shallow, and vertically downward typically consisted of about 4 1/2 feet of stiff dark mottled clay, and a 1-foot layer of water-bearing gravel over a tight shale.

Page 43 missing from original text

engineering will suffer. Let us hope that this is a passing phase in a transition from pure empiricism to the highest professional artistry in which soil mechanics and geology may play their proper, necessary, and worthy roles.

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MATERIALS GEOLOGY, CO-ORDINATION OF THE AGGREGATE INVENTORY, QUALITY CONTROL AND RESEARCH

By THEODORE L. WELP
Senior Geologist
Iowa State Highway Commission

The major items of importance in the use and evaluation of aggregates in construction are: (1) the aggregate inventory, to locate and classify the material and maintain permanent records of each source; (2) quality control, to insure that uniform quality materials meeting specifications are produced and incorporated in construction; and, (3) research, to study, compile and analyse the geologic and engineering properties of the material.

This involves a close co-operation with all the departments in the highway organization. It is a very large task, I think you can see it is not done by the geologists alone. It is taking from the work of many people the information on materials from the various phases of highway construction, centralizing, compiling, analysing and applying it to an evaluation of the aggregate materials used in relation to their geologic properties.

With the use of IBM data processing and computers it is now possible to data card program much of this information. Once programmed the number of analyses and correlations, that become possible are almost endless. Lists of sources by quality, location, any number of different test limits, uses, or types of construction can be made. Various types of maps can be prepared from this data listing.

By taking parts of other department programs such as maintenance maps and records, traffic analysis studies made with movie cameras, over-all performance surveys, and research reports, and applying these to materials geology the scope of the analysis possible is further expanded.

By co-ordinating these many phases, various test methods can be evaluated, reasonable and realistic specification limits can be set, specifications can be evaluated in terms of performance, construction type determinations aided by available aggregate summaries and problems isolated for special study.

This paper is a presentation of an operational method of the intensive application of geology to the general problems connected with aggregates.

The Aggregate Inventory Program

The first and basic phase is the aggregate inventory program. This is the starting point for the data to work on problems connected with aggregates. It should be a central record containing all data on available materials, their use and performance in service.

In Iowa aggregate source files have been kept for over 40 years. This is a centralized record for each source of material that has been used for construction or prospected by Commission personnel. Each file contains all the available data on the geologic and engineering properties of the material. The records for each source are filed by location within a county and a separate file kept for each county. These records are being systemized for microfilming and eventually IBM data processing.

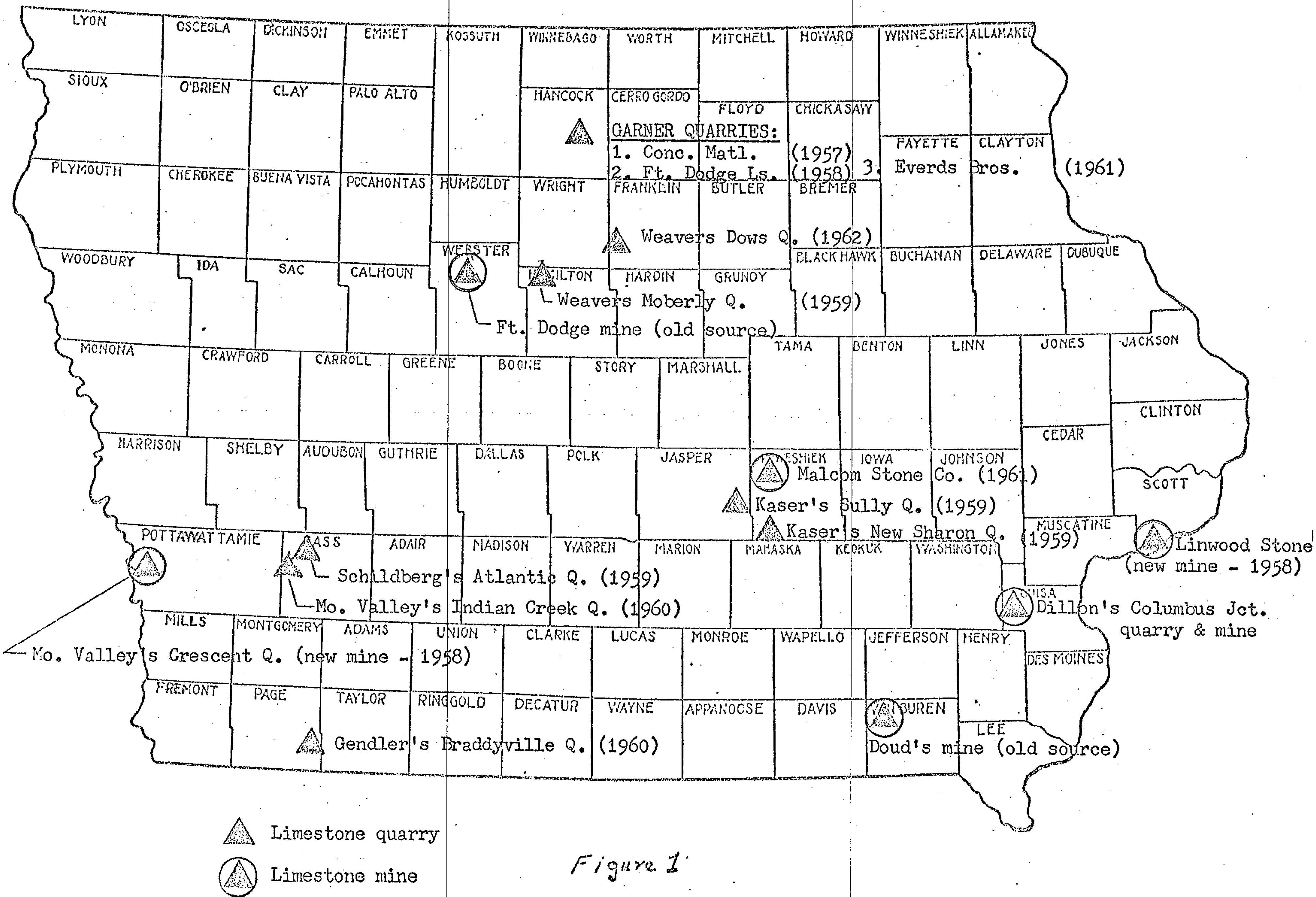
At the present time these files include records of over 7,000 sources of aggregates from: (1) unconsolidated granular deposits, sand and gravel and (2) consolidated sedimentary limestones and dolomites. These are no igneous rocks and only one quartizite source in Iowa. Our files show about 3,000 rock deposits and somewhat over 4,000 gravel deposits.

Although there are more gravel deposits listed nearly twice as much aggregate is produced by crushing rock.

We have two main methods of obtaining information on the location and quality of aggregate deposits.

The first method is investigation by Highway Commission personnel. As a matter of routine the materials department personnel make a record of, or obtain samples from potential deposits of aggregate material. Prospecting is done upon request, in certain aggregate poor areas. Permanent records are kept of all prospecting. All drill records of the soils section are sorted and catalogued for possible aggregate sources. Negative areas are delineated. Some or all of the modern methods of investigation are used, such as, soil maps, geologic maps, resistivity, and borings. Records made include description of pits, geologic logs, maps or sketches, estimates of available material, description of production facilities and production capacity.

Today most of the investigations by the Commission personnel are in the evaluation of deposits that have been located by the aggregate producers. In these cases we work with them to determine the extent of coring or sampling necessary for an evaluation. We generally examine and make records of, from 100 to 200 possible deposits per year. Many of these locations are prospected in detail by the producers.



LYON
OSCEOLA
DICKINSON
EMMET
KOSSUTH
WINNEBAGO
WORTH
MITCHELL
HOWARD
WINNESHIEK
ALLAMAKEE
SIOUX
O'BRIEN
CLAY
PALO ALTO
HANCOCK
CERRO GORDO
FLOYD
CHICKASAW
GARNER QUARRIES:
1. Conc. Matl. (1957)
2. Ft. Dodge Ls. (1958)
3. Everds Bros. (1961)
FAYETTE
CLAYTON
PLYMOUTH
CHEROKEE
BUENA VISTA
POCAHONTAS
HUMBOLDT
WRIGHT
FRANKLIN
BUTLER
BREMER
Weavers Dows Q. (1962)
WOODBURY
IDA
SAC
CALHOUN
WEBSTER
BLACK HAWK
BUCHANAN
DELAWARE
DUBUQUE
HARDIN
GRUNDY
Weavers Moberly Q. (1959)
Ft. Dodge mine (old source)
MONONA
CRAWFORD
CARROLL
GREENE
BOONE
STORY
MARSHALL
TAMA
BENTON
LINN
JONES
JACKSON
CLINTON
CEDAR
HARRISON
SHELBY
AUDUBON
GUTHRIE
DALLAS
POLK
JASPER
IOWA
JOHNSON
Malcom Stone Co. (1961)
Kaser's Sully Q. (1959)
Kaser's New Sharon Q. (1959)
SCOTT
MUSCATINE
Linwood Stone (new mine - 1958)
POTTAWATTAMIE
ADAIR
MADISON
WARREN
MARION
MAHASKA
KEOKUK
WASHINGTON
Schildberg's Atlantic Q. (1959)
Mo. Valley's Indian Creek Q. (1960)
Dillon's Columbus Jct. quarry & mine
MILLS
MONTGOMERY
ADAMS
UNION
CLARKE
LUCAS
MONROE
WAPELLO
JEFFERSON
HENRY
DES MOINES
Mo. Valley's Crescent Q. (new mine - 1958)
FREMONT
PAGE
TAYLOR
RINGGOLD
DECATUR
WAYNE
APPANOOSE
DAVIS
LEE
Gendler's Braddyville Q. (1960)
Doud's mine (old source)
Limestone quarry
Limestone mine

Figure 1

In Iowa our aggregate producers have very active prospecting programs. Most of the larger companies employ one or more geologists. The producers work in close co-operation with the Highway geologists in locating and prospecting potential deposits of material as close as possible to the job site. The most favorable deposits are sampled by borings.

The second method is information obtained from projects contracted to outside agencies. The information that is used in the inventory is generally a portion of a research project that has been contracted with wome other agencies, such as, the Geological Survey or one of the universities. This type of project is generally a regional study of an area on a problem selected by the Highway Commission.

The organization with the contract obtains, compiles and prepares for publication the information necessary to isolate areas for prospecting.

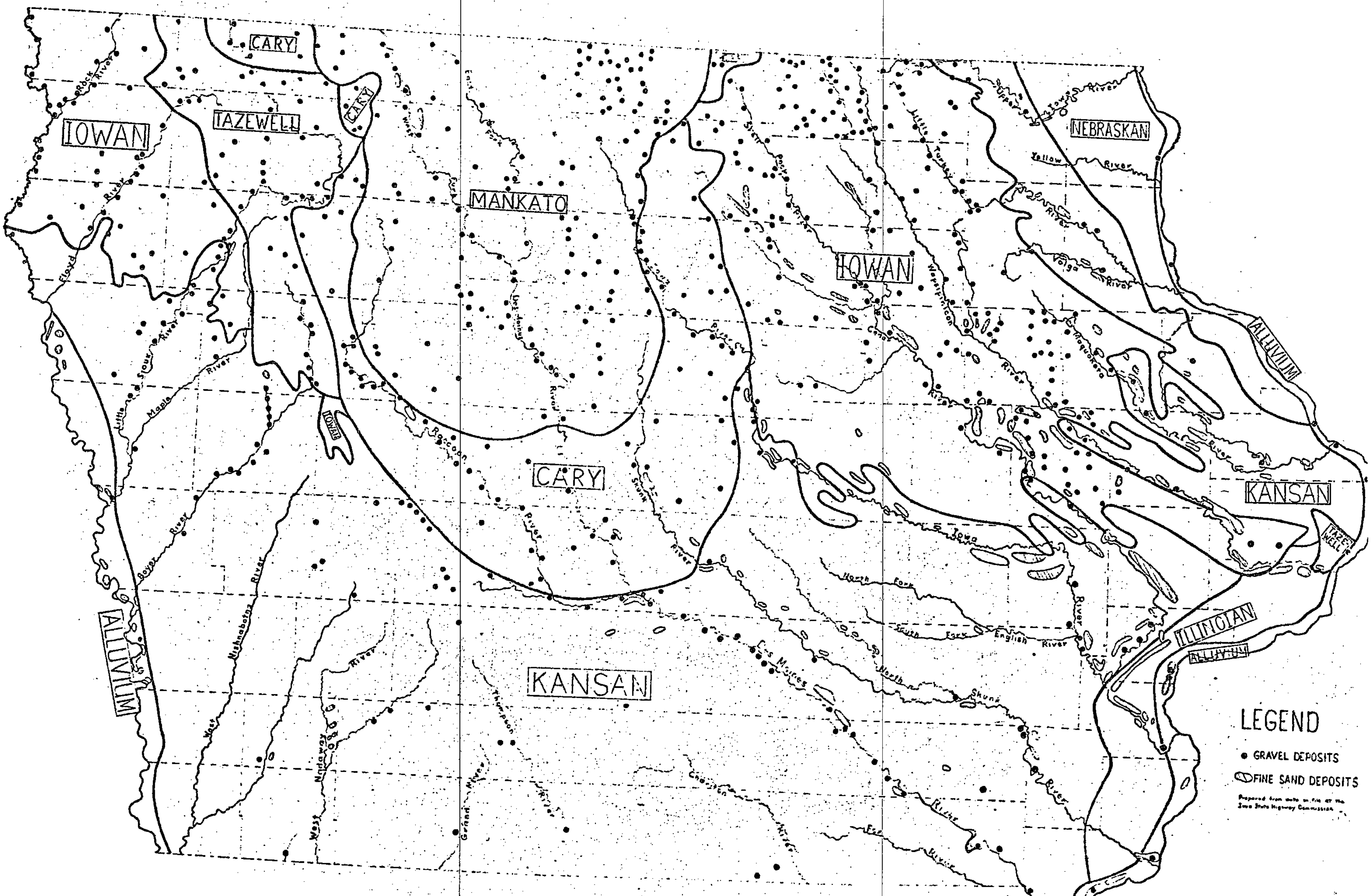
For example, studies of the geologic and engineering properties of the Pleistocene materials in Iowa produced an inventory of the fine sands. There are also reports on the geology of southwest Iowa, and studies on the Allubial materials of the Missouri river.

As the data are gathered they are analysed and fitted into the program to give us the maximum amount of information on the availability, distribution, geologic and engineering properties of these materials from the surface to depths up to 400 feet.

In local areas where materials are scarce aggregate producers are going farther beneath the surface in their search for materials. In the last few years several new mines have been opened and two or three others are being considered.

Methods of Presenting a Summary of Data

The inventory has been published in book form and county map form. Methods of presenting the data are much the same as used by other states. The book lists the sources by location with such pertinent data as owner, average test results, and geologic correlation. The maps are by county with code for test results showing the location of the source. In addition each county has a geologic column showing test result ranges for the various geologic units in the county. Office copies are constantly kept up to date and periodic revisions are published. All records are in files open to the public.



GLACIAL MAP OF IOWA
SHOWING
DISTRIBUTION OF GRAVEL AND FINE SAND

Figure 3

Performance Surveys

As part of the aggregate inventory program surveys being made to determine how well aggregates are performing in service in various types of construction. This is to establish the correlation between geologic and engineering properties, specification limits and performance.

Although this is done for nearly all types and qualities of material the most work has been done for portland cement concrete pavement. Pavement history record cards have been kept for every concrete pavement in Iowa. These contain design, construction and materials data. There are cards for several thousand projects, using almost 200 different sources of coarse aggregate, over 200 sands and several different brands of cement.

The information on the cards is now being prepared for IBM programming. From this it will be possible to sort rapidly all pavement of a certain design by coarse or fine aggregate, or any other of about 40 different criteria of design and construction.

From the pavement history cards maps of each county have been prepared showing each project by code number, and listing the source of coarse aggregate for each project. The maps are made with an overlay system and can be easily added to without redoing all previous work. The first printing covers all concrete pavement for the State of Iowa up to 1958.

Service records are being established for all aggregates used in concrete and being made to develop a program for classifying aggregates according to known service life.

For each geologically different source, records of the pavement condition are being compiled. These records include as much data as can be obtained. Photographs are taken showing representative stretches of pavements. Observations of both geologists and engineers are recorded. Whenever possible data on crack surveys and maintenance records are compiled. In some cases cores are taken for study from both good and bad pavement.

Representative stretches of pavement are chosen for detailed study. These are gage plugged for growth measurements and periodically inspected and photographed.

Uses of the Inventory Program

The Inventory program has a number of practical applications and in addition supplies data for statistical analysis for research. For each road project the geology

SOUTHEAST IOWA - District 5 Approx.
Plus Jasper and Poweshiek Counties

Source	Average P. I. 0 - 4	4 - 6	6 - 8	Remarks
Durham, St. Louis	x	x	Some	
Oskaloosa	x	x		Variable
Sulley, Spergen	x	x		
Keokuk	x	x	x	
Malcom	x			
Sigourney	x	x		
Ollie, Burlington	NP x			
Ottumwa	x	x		Only a few tests
Selma, St. Louis	x	rare		
Douds, Spergen	NP x			CBR 5-12-17-20-23
Farmington	x			
Argyle	x	x		Top lift up to 7
West Point		x		
Lee Co.	x	x x		CBR 19-21-21-20-20
L & W Stone	x - - - -	- - - - - -	- - - - - x	Variable - 3,2,2,11 4,7, 4,7,3,3,6,4,4. CBR 6,15,23,27,31.
Centerville Area	x	x	occa-sional	CBR 3,5,9,12,13
Mt. Pleasant		x	occa-sional	

Figure 5.

Tabulation of average plasticity index for quarries in District 5.

METHODS OF SAMPLING CORES FOR QUALITY

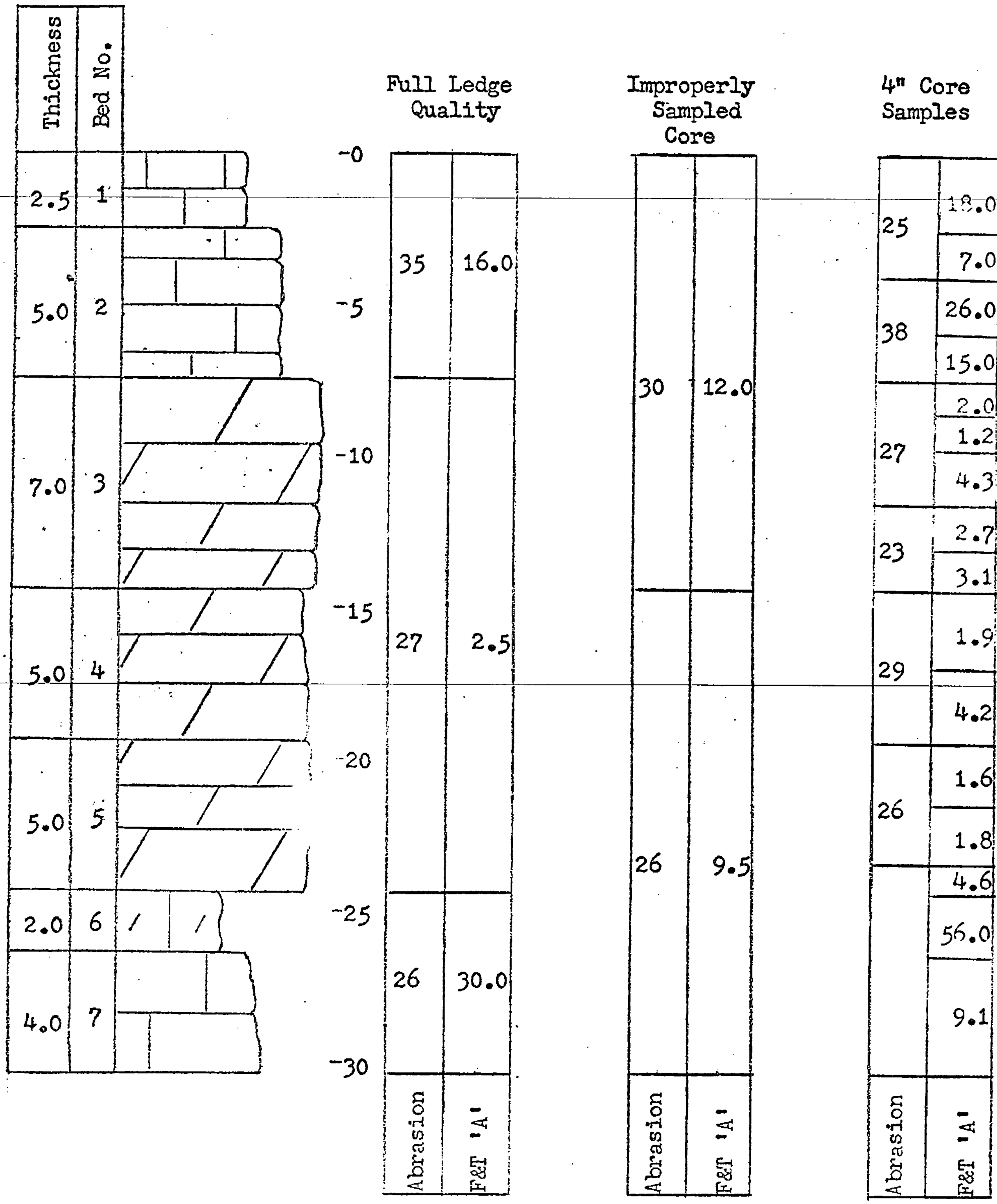


Figure 6.

Research

The third phase is the application of geology to aggregate research.

We try to develop research on aggregates that will have practical and valuable application to highway construction. This, again, like the aggregate inventory and quality control phases, is a co-operative venture between geolgoists, engineers and other specialized scientists.

For many years the materials department has been very active in the field of research on aggregates and their behavior in various types of construction. For example, studies have been made on the distribution, the basic geologic and engineering properties and various methods of testing aggregates. Long range studies are being made on the properties of concrete containing various coarse aggregates and on the test methods that can be used to evaluate these properties.

Some of the research work is done in the materials department by Highway Commission research personnel, while other portions that require special equipment, techniques and highly skilled personnel are let on contract to other state or federal agencies. The research generally let on contract is a problem or phase of problems designed to supplement the research being done by the Commission.

The materials geologist is an integral part of this program. The aggregate inventory shows the kind of material available, where it is, about how much there is of it and it's geologic properties. He can evaluate and relate various tests to the geologic characteristics of the parent material used for aggregates. He can advise on the selection of aggregates for various experiments and he can add the geological and engineer findings to the date analysis.

The best way to illustrate the application of this type of program is by the following two examples.

In the early 1950's the aggregate inventory showed that the Southwestern half of Iowa was lacking in sources of aggregate suitable for construction and the most likely materials could be found in consolidated rocks of Pennsylvania age. A research project was initiated with the Iowa Geological Survey to prepare a regional geologic report on this area. At the same time the Commission undertook studies of the engineering properties of the material and the study of possible upgrading and, or, possible special types of construction.

The second example is in the field of concrete.

One of our main research programs has been in the field of concrete aggregates, particularly the aggregates made from the carbonate rocks and some gravels.

Service record studies indicated that there was a correlation between concrete deterioration and the source of the coarse aggregate. Deterioration of the concrete began at pavement ages ranging from 5 to 20 years, depending on the source of coarse aggregate used. Since some of the pavements were made with aggregates meeting current specifications the problem is extremely important.

A compilation of all available data on a wide range of aggregates and service records indicated the following: (1) that in general, there was a good correlation between current specifications and service life; (2) that a large range of poor performance aggregates were eliminated by current specifications. These were the high clay content rocks; (2) that a small number of sources could produce material meeting specifications and have a service life of only 8 - 15 years (these appeared to be high carbonate content rocks with generally low insoluable residue content); and (4) that for these sources there were no readily apparent diagnostic characteristics or test results on aggregate or concrete.

The last conclusion was based on a compilation of several hundred tests results that included, physical properties, chemical analysis, and mineral content. Also compiled were compression tests on concrete cylinders and growth figures on beams in outside storage for about 20 years.

In addition a review was made of published works on the problem by other states and some of their data incorporated in our analysis.

The indications were that there existed a complex relationship among all the components of the concrete system. That concrete with poor performance contained aggregates that were similar, according to the current tests to those with good performance records.

As a result of this review a concentrated program has been developed to study this problem from several possible approaches. Aggregates from 11 different sources were selected for study. These were chosen to represent a wide range of carbonate rock types and service life. Research was instigated at the Commission to study the aggregates and the concrete made with these aggregates, both in test specimens and pavement.

Aggregates were tested by normal routine Commission ...thods which include, freezing and thawing, abrasion, chemical analysis, and special tests, such as, x-ray analysis, etc. Concrete specimens were made for compression and freezing and thawing tests.

In addition contracts were made with Iowa State University to develop new methods to study the basic properties of carbonate rocks and to study the concrete made with these aggregates. Highly specialized studies are being made to try to determine the mineral structures of the calcites and dolomites present, mode of dolomite occurrence, the distribution of magnesium content, porosity, mineral variation and clay content. This study is to be made on each layer making up the rock face used in concrete.

Dr. Lemish, the next speaker, and the man who has contributed greatly to our knowledge of carbonate aggregates, undertook to study reactions that may be taking place with time in concrete made with these same 11 aggregates. Pavements selected for studies, from each of the several sources will range in age from 1 year old to about 40 years old. Pavement conditions will range from excellent to badly deteriorated.

One phase, recently completed and reported on by Mr. DeYoung and Mr. Faul, at the Highway Research Board meeting in Washington D.C., was on results obtained on freezing and thawing concrete beams in an automatic freezer. Tests were run according to ASTM C291-61T. Our experience here indicated that this particular test was extremely sensitive to unsound particles. Whenever the coarse aggregate contained small percentages of chert, shale, soft or shaly rock, erratic results were obtained.

The over-all research project is well underway. Indications are that in a few years we will have extensive data on test procedures, aggregate properties, and concrete.

Summary

The materials geologist can bring together many diversified data on aggregates and efficiently aid in the application of this knowledge to highway construction.

The aggregate inventory contains the basic data. The materials geologist can apply the data and his knowledge to the quality control and research programs.

The program must be more than just paper work. It must be workable and useable by many people with differing educational backgrounds. This means it must be reasonable and still provide data that will have practical application.

Once a program is established the geologist must check on its operation and periodically revise it to meet changing needs and conditions.

CARBONATE AGGREGATE RESEARCH

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Iowa State University
Ames, Iowa

Introduction

At the present time a considerable amount of research on carbonate aggregates is being conducted in the United States and Canada. This research is concerned with the behavior of carbonate rocks when used as coarse aggregate in concrete. In view of the widespread use of carbonate aggregates and the current large-scale highway construction program, research in this area is of considerable economic importance. It is also an area in which geologists have an opportunity to contribute.

The purpose of this paper is to review carbonate aggregate research in terms of (1) a brief account of its history, (2) the problem, (3) a resume of current work, and (4) some opinion on areas and direction of future endeavor.

History

The history of carbonate aggregate research can be divided into two periods: an early period prior to 1955 in which a petrographic approach was chiefly employed, and the current phase begun after 1955 with the recognition and investigation of carbonate aggregate reactivity. In this latter phase petrographic, chemical and physical methods have been employed.

One of the first papers of the early period was that of Laughlin in 1928[1] demonstrating the value of petrography in selecting limestones for aggregate use. Some other papers concerning carbonate aggregate are those by Sweet (1948),[2] Mather (1953),[3] and Roy, et al (1955).[4]

After the recognition of the reactivity of certain carbonate aggregates, a new phase began shortly after 1955 by two groups—Lemish and his associates in Iowa under sponsorship of the Iowa State Highway Commission and Swenson and his group in Canada under the sponsorship of their National Research Council. Since then others who have commenced carbonate aggregate research are Hadley at the PCA Laboratories, Newlon and Sherwood in Virginia, Dunn in New York State, Mather of the Corps of Engineers, as well as Chaiken and Halstead of the Bureau of Public Roads.

The Problem: Its Recognition and Aspects

The problem, briefly stated, is the deleterious behavior of certain carbonate rocks when used as coarse aggregate in concrete. On the basis of service records the relationship between carbonate aggregates from specific sources and deterioration in concrete has been well established in the United States and Canada. Recognition of the problem depends on observations regarding the character of deterioration of concrete made from deleterious aggregate. The recognition and various aspects of the problem will be considered below.

Field evidence and experience in Canada as reported by Swenson and Gillot[5] are similar to that described in Virginia by Newlon and Sherwood.[6] The characteristic features are the excessive expansion and associated cracking in concrete. The cracks are sharp and little if any spalling is evidenced at the edges of the cracks after the cracks have persisted for 20 years. The cracks appear two to three years after placing the concrete. In extreme cases Swenson reports cracking occurring two to three months after placing. The phenomenon in Canada occurs when a high alkali cement is used and the resulting concrete is exposed to a moist environment. In Virginia the cracking has been observed in concrete made of "low alkali" cement and the degree and extent of cracking appears to compare favorably to Canadian occurrences when the reactivity of the rock and alkali contents of the cements are taken into account. The cracks are 2 to 4 inches apart and can penetrate up to two-thirds of the concrete slab. No reaction products are observed in the cracks and the internal areas surrounded by cracks are sound.

In Iowa the nature of deterioration in highway concrete is characterized by a progressive cracking and spalling away from joints or transverse breaks in the concrete. In affected concrete deterioration first appears after 5 to 20 years of service and becomes progressively worse with time and severity of such conditions as traffic and exposure. Concretes made with two argillaceous carbonate aggregates from the Glory and Earlham quarries (these are no longer used and do not pass present acceptance tests), showed the incipient first signs of deterioration after 5 years of service. Other concretes made from aggregates which pass current acceptance tests showed their first manifestations of distress over a period of 8 to 15 years. Some concretes show their initial deterioration after 20 years of service. In all cases the general progressive pattern of deterioration is characterized by spalling away from joints. In cores of highway concrete the cracks cut across the aggregate and concrete matrix. Cores of concrete made from Glory aggregate and in an advanced state of deterioration are characterized by reaction rims on the coarse aggregates and a carbonated matrix. Such concrete is generally in a weakened condition throughout and breaks easily under a hammer. An important aspect observed in Iowa is the lack of definite field and laboratory evidence for excessive expansion.

In comparing the Canadian and Virginian observations with those in Iowa it is evident that differences exist. The deterioration in Canada and Virginia described and studied to date can be characterized by excessive expansion with associated cracking occurring within a relatively short time. In Iowa spalling and a gradual but progressive deterioration away from joints without excess expansion characterizes the nature of deterioration beginning after 5 to 20 years of service.

The observations and over-all experience with the problem as it occurs in different areas has had a strong influence on the approach and direction taken by those involved in carbonate aggregate research. The investigation of aggregates related to excessive concrete expansion within a relatively short time was initiated by Swenson and has been continued by Hadley and Newlon. Study of the expansion properties of the aggregate and the dedolomitization "reaction" has characterized this research. In Iowa the research directed by Lemish has approached the problem by studying the physical properties of the rock and a study of the alkali-induced reactions such as silicification and dedolomitization in carbonate rocks both in concrete and other environments.

Other aspects of the problem exist which should be considered in reviewing current research or predicting its future trends. One aspect is the question of what is the expected service life of a concrete? How long do we expect it to last? Another aspect directly related to the recognition of the problem is the extent and thoroughness with which service records of concrete are kept. This strongly influences the criteria used to determine satisfactory or unsatisfactory service. What is a problem to one group may not be considered a problem by another.

Although the problem is basically the deleterious behavior of certain carbonate aggregates in concrete it is readily apparent many aspects of it still have not been completely defined. The current status of the problem is very much like the description of the elephant by a group of blind men. An integrated review of the many aspects of the problem is needed to guide the direction of future carbonate aggregate research.

Current Research

For the purpose of this paper the current research which has been published can be grouped into two areas:

1. The work by Swenson and Gillot, Hadley, and Newlon and Sherwood on reactive argillaceous dolomites related to excessive concrete expansion as described in Canada and Virginia.

2. Investigations in Iowa by Lemish and his associates on alkali-induced reactions in concrete and other environments.

Research on Expansive Aggregates

The first research in this area was reported by Swenson in 1957[7] and in a later paper in 1960 by Swenson and Gillot.[5] Their studies are concerned with the Ordovician carbonate aggregate from Kingston, Ontario, which passed all their physical and chemical tests but was related to expansion and cracking of concrete using this alkali reactive aggregate. In their investigation they found that the Kingston carbonate reaction was not detectable by standard ASTM tests. It could be detected by measuring expansion of concrete prisms exposed to high humidity or by measuring expansions of rock in alkaline solution. They tentatively concluded that rock composed of near equal proportions by weight of dolomite and calcite are suspect and stated that a possible connection exists between the expansive reactivity and the dedolomitization reaction.

Hadley[8] continued research in this area and emphasized the expansion characteristics and the dedolomitization reaction. The expansion is considered to be related to a dedolomitization reaction between the alkalies and the mineral dolomite and postulates the reaction as follows:

$$CaMg(CO_3)_2 + 2MOH \rightleftharpoons M_2CO_3 + Mg(OH)_2 + CaCO_3$$

where M stands for K, Na, or L.

The alkali carbonate could react with hydration products of cement to regenerate the alkali; i.e., $Na_2CO_3 + Ca(OH)_2 \rightleftharpoons 2NaOH + CaCO_3$. Hadley also developed a simple and rapid way of testing the expansion of the rocks by modifying the approach used by Gillot and measured tapered prisms in a special micrometer gauge periodically while soaking them in alkaline solutions. After further studies of expansive rocks Hadley postulated that the potentially reactive carbonate rock had the following characteristics:

a. Carbonate mineralogy --
Dolomite mineral comprises 40-60% of total carbonate fraction.

b. Clay mineralogy --
Rocks containing between 10 to 20% clay.

c. Texture --
Partially dolomitized calcilutites with isolated dolomite rhombs in microcrystalline matrix of calcite and clay.

The investigation of Newlon amd Sherwood[6] continued along similar lines and stressed the laboratory evaluation of the expansive behavior of various rock types. The Hadley expansion prism method was employed and they found that the behavior of prisms could be divided into three groups of high, medium, and low expanding rocks, and a fourth group of slightly contracting rocks. In a state-wide survey of rock behavior in alkaline environments, they find that most rocks contract slightly. 52 out of 231 samples expanded to some degree after eight weeks of soaking in alkaline solutions. Very few rocks can be called excessively expansive. In concrete studies they show a relationship of expansive aggregate in concrete beams as related to the alkali content of the cement and degree of expansivity of the aggregate. They find the potentially reactive aggregate have essentially the same characteristics as postulated by Hadley.

The research to date in this general area has been significant and provided a description of the type of rock which is associated with expansion. Hadley[8] believes the dedolomitization reaction is related to expansion. To date however no satisfactory mechanism to account for the expansion by means of the dedolomitization reaction or any other process has been demonstrated.

One comment seems appropriate at this time on the use of the work "reaction" by the researchers in this area. "Reaction" is restricted by implication to describe a dedolomitization process believed to be related to expansion. The term is also further restricted to implying a deleterious effect when applied to concrete. Geologically speaking, all rock will react to adjust to their environment and it would be better to avoid restriction of the word to a specific reaction and/or deleterious effect.

Research in Iowa

The research in Iowa on carbonate aggregates has been conducted in a broad way utilizing several approaches which can be summarized as studies involving:

1. Physical properties of aggregates.

2. Chemical behavior of aggregates.

3. Aggregate behavior in concrete.

Although a considerable amount of research has been conducted in the physical properties of aggregates a review of the alkali-induced reactions in aqueous solutions and concrete will be presented at this time. A general review of this work has been published recently in the Transactions of the AIME.[9] It should be stated that a considerable amount of the work is related to Devonian rocks including deleterious aggregates from the Glory Quarry characterized as an argillaceous dolomitic type with high insoluble residues. In studying deteriorated concretes made from such impure dolomite, evidence of their chemical reactivity was characterized by reaction rims.[10] Such rocks were selectively affected whereas relatively "pure" carbonate aggregates (high carbonate mineral content and low insoluble residues) were not. As a result of these investigations a large part of our initial research effort consisted of a concentrated effort to investigate the nature of the reactions causing the rims.

The initial investigation of the rimmed aggregates taken from deteriorated concrete indicated that rims were silicified and contained a higher silica content.[11] Reaction shells were grown experimentally in impure dolomitic aggregates in silica-rock solutions at pH 12. Further work on rim growth demonstrated that reaction rims on the impure dolomitic aggregates could be experimentally grown in concrete bars by soaking them in water at 125°F or by alternate wetting and drying.[12] It was also possible to show in a quantitative experiment that rim growth on chips was due in part to introduction of silica from the mortar. It was established in this experiment that sodium and potassium were not introduced in the shell.[13]

The effects on the various constituents of the aggregate (carbonate minerals, quartz, clay) as well as the cement was undertaken in a study of the compositional variations associated with the reaction by measuring the chemical and mineralogical changes across the aggregate cement interface.[14] In the cement environment it was found that all components of the rock are affected to form a silicified dedolomitized shell. The shells were found to represent the presence of amorphous silica much of which can be locally derived from the fine grained quartz in the rock. Silica can migrate in or out of the rock and is accompanied by the breakdown of dolomite and increase in calcite. Several reactions occur simultaneously and appear to represent the attempt of the rock to come to equilibrium with the hydroxyl-rich environment.

One of the questions asked frequently is, where does the silica come from? In comparative studies on impure dolomitic aggregates in mortar and cement paste environments it was found that the aggregates increase their silica content when reacted in a mortar environment and decrease their silica content when reacted in a cement paste environment. It was concluded that cement paste is probably not a source of silica but it does plan an important role in providing a high pH environment.[15]

Comparative tests on a series of concrete bars made from aggregates with good and poor service records respectively show that concrete in which the reaction has occurred do not gain in strength whereas concrete bars unaffected by the reaction had a 50% increase in strength. In either case no excessive expansion occurred.[13]

In some recent work as yet unpublished it has been established that the high temperature and aqueous environment utilized in early experiments to induce the reaction in the laboratory effects the rate but not the nature of the reactions.

Up to this point the reaction associated with a deleterious argillaceous dolomitic aggregate of the Glory type has been emphasized. Many other intermediate types were also studied. Study of another deleterious aggregate of Mississippian age from the LeGrand Quarry indicates the complexity of the problem in understanding the cause of distress in concrete. In contrast to the dolomitic high insoluble residue rocks, the LeGrand is a dolomitic type with low (1-3%) insoluble residue. The LeGrand rock does not grow rims, it does not expand in alkaline solutions or when used in concrete. In contrast to the Glory type of aggregate the LeGrand rock does pass present acceptance tests. Such experience clearly indicates that a considerable amount of work is required to understand the behavior of carbonate aggregates in concrete.

As of yet the mechanism of how the alkali-induced reaction contributes to failure in concrete cannot be answered. The nature of some potentially reactive rocks has been ascertained. Service records indicate a casual relationship of aggregate to distressed concrete and in some manner the alkali-induced reactions described above, directly or indirectly contribute to distress.

Summary

In reviewing the current research on carbonate aggregates it is readily apparent that a considerable amount of research knowledge has been acquired. Research on argillaceous dolomites with expansive behavior has partially defined the over-all character of such rock and a relatively simple expansion test can be effective in detecting potentially deleterious aggregates. Fortunately such rocks are not abundant or widely used.

The work in Iowa on their local problem has been successful in defining the nature of the reactions rocks undergo in alkaline environments and it does indicate that high insoluble residue dolomitic rocks should be considered suspect. Current testing procedures eliminate this type of rock. However, it is equally evident that all the potentially deleterious rocks cannot be described by such criteria.

No one mechanism or variety of mechanisms for the manner in which carbonate rocks contribute to distress in concrete can be postulated at this time. The problem though in part solved still has many aspects requiring more detailed study.

Future Work

Without doubt the direction of future research will require better techniques and more intensive detailed work in a variety of areas such as the pore structure of the aggregate, better means of characterizing rock textures, and a more intensive effort to understand the mechanisms by which aggregates can cause distress. Ways of inhibiting distress caused by aggregates is also a fertile field of endeavor. Although it has been necessary to date to emphasize coarse aggregates, the other components of concrete, the fine aggregate and cement paste, should not be neglected. A better understanding of the system known as concrete and the changes it undergoes with time is required before the true role of carbonate aggregate reactions can be properly evaluated. Sound concrete requires that all the materials used, the coarse aggregate, cement and fine aggregate from a compatible system and more knowledge than presently available is required to understand the changes taking place in concrete undergoing deterioration. It is in this area where many of the answers to the problem will be found.

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THE EFFECT OF FRACTURED GROUND ON HIGHWAY STRUCTURE DESIGN AS DEMONSTRATED BY TWO CASE HISTORIES

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Introduction

Strictly speaking, a fracture is defined in the American Geological Institute's Glossary of Geologic and Related Sciences as: ". . . . a break in rock due to intense folding or faulting." However, in the study of ground conditions, preliminary to the designing and building of highway structures, any rupture or break present in the rock material to be traversed or upon which the structure is to be founded is of potential importance. Therefore, in the following discussion, liberties are taken with the strict definition and it will be understood that the term fracture implies any break in rock, whether tectonicly induced or not. On this basis, jointing resulting from the cooling of surface volcanic flows, rock cleavage such as that commonly found in crystalline metamorphic rocks, and true fracturing such as that associated with major faulting and folding will all be considered as forms of fracturing.

During the past several years, personnel of the Arizona Bureau of Mines have been involved in the geologic examination of sites for two Arizona Highway Department projects and in both of these the determination of the nature and degree of fracturing has been of considerable importance. These projects, the Mule Pass tunnel and the Burro Creek bridge, are of rather respectable size by Arizona Highway Department standards and, it is my hope that a description of the geologic conditions encountered and their effect on both the design phase and the construction phase in the case of the tunnel will be of interest.

Before proceeding with a description of the projects, however, I wish to stress that Bureau of Mines personnel had no direct part in the actual design of these two structures and can claim no credit for them. I believe, though, that there is little question that the geologic conditions were of considerable consequence in the establishment of the final design criteria.

Mule Pass Tunnel

In 1956 the Mule Pass tunnel project was started with the awarding of a construction contract, which included the main tunnel and 0.8 miles of roadway, at a cost of \$2,129,300 (Figure 1). The tunnel, which is located on the western outskirts of Bisbee, Arizona, on U.S. Highway 80, is 1400 feet long and has a finished section 42 feet wide at the springline and a height of 23 feet at centerline.

Prior to the awarding of the contract, however, considerable exploration was undertaken by the Highway Department including, at a cost of $70,000, the driving of a 6-foot by 7-foot pilot tunnel, 1800 feet long, on centerline, and at what was proposed to be the back of the main tunnel section. It was at this point in the exploration program that a geologic examination of the site was made by geologists of the Arizona Bureau of Mines.

The prevailing rock at the tunnel site is a fine to medium-grained granite, of Nevadan age, which intrudes Precambrian schist. This, the Juniper Flat granite, occurs as an elongate intrusive mass which trends in a general northwesterly direction, essentially parallel to the bearing of the tunnel. At the southeastern end of this granitic mass, where the tunnel is located, the intrusive has an outcrop width of approximately one-half mile.

A review of the structural trends in the general region of the tunnel site indicated that four major directions are discernible. These include north-south, characterized by normal faults; N30°E, with a probable component of lift-lateral strike slip; an east-west trend which is defined by normal faults and by dikes intrusive into both the schist and the Juniper Flat granite; and the fourth direction, N45°W, which is represented by overthrust faults wherein the upper plate has moved relatively northeast, axes of major folding displayed in the Paleozoic sedimentary rocks of the area, dikes genetically related to the Juniper Flat granite, and by the Juniper Flat granite, itself.

Detailed mapping of the pilot tunnel proved that these trends are equally prominent within the Juniper Flat granite. In the 1800 feet of tunnel mapped, 211 faults and fractures were actually recorded by dip and strike, and these include only those traceable from one wall to the other. Of these, 167 contained breccia and/or gouge ranging from a few tenths of an inch up to as much as three feet in thickness.

By count, fractures of the east-west system were the most numerous, accounting for over one-third of those recorded. These were followed closely in number by fractures of the N45°W trend. Concerning the N45°W set, however, there is some question concerning the validity of the count inasmuch as the bearing of the tunnel coincided very nearly with this direction and it is quite likely that the tunnel actually was driven within a zone of fracturing of this trend. In fact,considering the dip of the N45°W fractures, which averages 30° to the southwest, there is a strong likelihood that the tunnel was driven along a zone associated with the overthrust faulting pervalent in the area.

Dips of fractures of the north-south, N30°E, and east-west fractures are usually steep, ranging from values of 60° to vertical.

Although no record was made of the attitudes of what may be called lesser fractures, some indication of the condition of the ground was obtained by noting the average size of the blocks defined by these features. These blocks ranged from about 6 inches (excluding breccia zones) up to several feet in size and averaged probably about two feet in cross section.

On average, considering the degree of fracturing and prevalence of gouge zones, support was designed for the classification of "very blocky and seamy ground" as outlined for example in Proctor and White (Rock Tunneling with Steel Supports).

Temporary support, during construction was provided by steel rib and post sets designed for a loading of 12 feet of rock at 160 pounds/cu. ft. and these were placed on three-foot centers. In the west 200 feet, however, rather severe fracturing was encountered and it was necessary to reduce the spacing between ribs.

Although this stretch of ground had been indicated on the geologic map of the pilot tunnel as a weak zone, it was not entirely obvious from the pilot tunnel exposures that caving would be as severe as it ultimately developed. As in other stretches of the tunnel, a prominent N45°W fault, dipping between 15° and 30° southwest defined the back of the pilot tunnel and a somewhat higher than average density of high-angle east-west faults were recorded. The trouble developed from the fact that, unlike the condition found in the driving of the rest of the tunnel, a number of closely spaced, low-angle breaks, subparallel to the faults which defined the back of the pilot tunnel, were concealed in the back and therefore considerable slabbing of the back and therefore considerable slabbing of the back resulted. Stand-up time in this area was reduced almost to zero and before conditions could be controlled a cave, some 15 to 20 feet high, had developed and a system of square-set support had to be installed to catch up the caving ground.

An interesting side light on this particular tunnel concerns a minor water problem that occurred. During the mapping of the pilot tunnel, several zones were noted where small quantities of water were making along fractures but, considering the arid nature of the region and the location of the tunnel near the crest of a drainage divide, the geologist did not consider this of consequence. The Highway Department did, however, fortunately design for drainage behind the lining and the tunnel has made an average of 5 gallons per minute since its completion and a minor weep in the lining is still giving some trouble.

Burro Creek Bridge

The second project I wish to review is the Burro Creek bridge upon which construction is scheduled to start approximately April 1, 1963. As originally proposed, the structure was to be a steel arch spanning 670 feet between pin points and with a rise

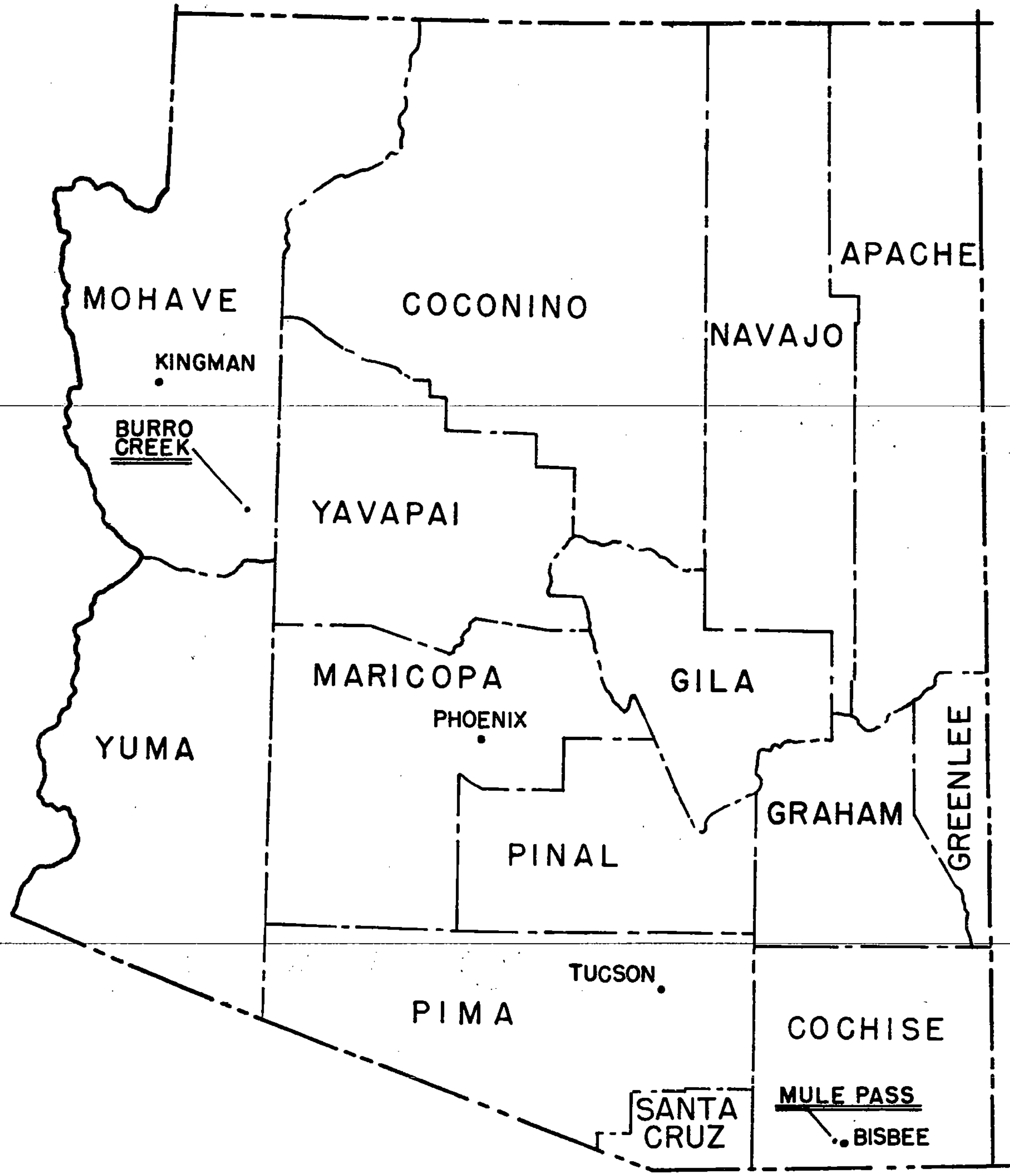

Figure 1. Index Map of Arizona showing approximate location of Mule Pass Tunnel and Burro Creek Bridge site.

of 125 feet of the bottom chord. Located approximately 70 miles south of Kingman, Arizona, the State highway 93 (see Figure 1), this bridge will be of considerable importance because it will eliminate some 4 miles of twisting roadway containing reverse curves and grades of up to 8 1/2 percent. Highway 93 carries considerable heavy truck traffic and this stretch has proven most hazardous.

At the bridge site Burro Creek flows from north to south and has cut a canyon nearly 400 feet deep through a sequence of rocks divisible into three separate units. These consist of an upper unit between 200 and 250 feet in thickness comprised of alternating basalt flows and cinder beds, a middle unit of flat-bedded sedimentary material ranging from 50 to 100 feet thick, and a basal unit of massive rhyolite tuff with an exposed thickness of 130 feet.

Basal Unit

The basal unit has been tentatively classified as a welded rhyolite tuff although no petrographic study has been made of the rock to verify this. The unit is exposed in the bottom of the canyon and forms a steep cliff in the west wall at the construction centerline. During the preliminary examination this exposure was not examined in detail because of falling rock from the work in progress above, however, the unit is well exposed in the east canyon wall approximately 200 feet north of the centerline. At this exposure the rock is grey-white in color and consists of fine volvanic ash partially fused to form a relatively hard, massive unit. Throughout the unit, as exposed, a prominent planar structure, interpreted as flow banding, is present. This structure strikes N45°E and its dip varies from 80° southeasterly through vertical to 80° northwesterly. Numerous small, discontinuous veinlets of light grey volcanic glass are intruded into the unit more or less parallel to the planar structure. During a later examination, when the outcrop on the centerline was accessible, the continuity of the unit was confirmed.

Middle Unit

The middle unit consists of a light tan to white water-lain tuff which rests unconformably on the underlying rhyolite tuff. The fragmented material of which this unit is composed is very similar in character to the material in the basal unit, and, undoubtedly was derived from the erosion of that unit. At the construction centerline the unit strikes N80°E and dips 12° in a southerly direction. Downstream, to the south, the dip progressively decreases and is only 7° at a point approximately 750 feet south of the centerline where the unit disappears below the floor of the canyon.

Upper Unit

At least six individual flows, ranging from 10 feet to 70 feet in thickness, are present in the west abutment and five flows, ranging from 20 feet to 70 feet in thickness, are present in the east abutment. These are locally separated by lenticular beds of cinder and weakly sintered pyroclastic material ranging up to 15 feet in thickness.

During the preliminary examination it was concluded that the lower two units would present no special problems, inasmuch as both units are more or less homogeneous within themselves and far enough below the pin elevations that the loads contemplated would be distributed over a large enough area that the unit loading would be quite small in these units. The preliminary plans called for the skewbacks to be placed in the upper unit, however, and that unit was therefore of particular importance.

Detailed examination of the basalt flows of the upper unit along the centerline in both abutments revealed that the flows are highly jointed and that two sets of fractures predominate. All flows display a vertical set of joints typical of the columnar structure frequently found in basalt flows. The second set, essentially horizontal, subparallel to the flow surfaces, and in general more closely spaced near the upper surfaces of the flows, was found to be most highly developed in the thinner flows. On weathered surfaces and in shallow cuts, many of the vertical joints were found to be open by as much as two inches. Others were partially filled with either fine, weakly cemented calcium carbonate or loose cinders which were apparently washed into the fractures. These two sets are believed to be cooling fractures.

In addition to the two principal joint sets, numerous randomly oriented fractures were apparent in the thinner flows and near the tops of the thicker flows. These are believed to have resulted, at least in part, from the breaking up of the solidifying crust of the flows during initial emplacement.

On the basis of the detailed examination of the upper unit, it was concluded that two serious problems existed, if the pin elevation was maintained as the preliminary design specified. First, lenses of weak pyroclastic material would occur in the foundation material and because of the extreme variability of thickness and competency of this material, differential settlement between the left and right skewbacks could be expected. Second, the skewbacks would be founded on one of the thinner, more fractured flows which would further aggravate the situation.

Because of the nature of the joints and fractures observed in the shallow cuts and weathered surfaces exposed during the preliminary examination, and because of the origin presumed for these features, it was concluded that they would persist at depth and special measures would be required for stabilizing the skewback foundations. It was obvious at this point that, in any event, the skewbacks would have to be placed farther back into the abutments and excavation for exploratory purposes was continued.

A second examination was made after the exploratory cuts had been deepened some 20 feet beyond the surfaces exposed at the time of the original examination. As originally concluded, fracturing and jointing were as intense as in the surface exposures but, contrary to the original conclusions, the greater majority of the fractures were tight, with little or no cinder or lime filling.

It was found that by increasing the span to 680 feet and lowering the pin points by 20 feet, the skewbacks would be founded on the lowermost flow and that no pyroclastic beds would be involved in the foundation. Further, this flow is one of the thickest of the sequence and not as badly fractured as the thinner flows.

In view of the extra time required for exploratory work, and in order not to delay the establishment of the final design specification unduly, the Highway Department utilized computers to solve the design problems and were therefore able to complete the design in a minimum of time after the site exploration had been completed.

SOME GEOLOGIC FACTORS IN HIGHWAY SLOPE FAILURES IN NORTH CAROLINA

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Introduction

The problem of slope failure in highway construction is much more acute now than in the past because the present superhighways require deeper cuts and higher fills than was necessary for the highways built to less rigid specifications. In addition, correction of slides and subsidences on previously constructed highways is a costly operation and constitutes a problem in itself. For example, in North Carolina the maintenance and correction of slope failures in highway cuts and fills costs the state several hundred thousand dollars each year. Because this work is done when and as needed, the total magnitude of the problem is not generally appreciated. Accordingly, a project was initiated in June 1962, as part of the state's highway research program sponsored jointly by the State Highway Commission and the U.S. Bureau of Public Roads, to determine not only the extend of the slope instability problem in North Carolina, but also to determine the factors responsible for failures of soil and rock slopes. One of the primary objectives of this study is to provide the State Highway Commission with information about slope failures in a form which will be usable and applicable to the problems of maintenance and slope design. The present report presents some of the preliminary results of this study. The opinions and interpretations discussed herein are the responsibility solely of the authors, and are not to be considered official statements of the sponsors.

Method of Investigation

During the first field season nearly 400 slides in 39 North Carolina mountain and piedmont counties were identified, described, and located on geologic and highway maps. At each slide various data were recorded, including notations regarding physical dimensions, rock type, degree of weathering, groundwater conditions, and type of movement. Most of the failures observed had occurred in cut slopes, but in many areas indications of imminent failure of fill slopes were observed. The lower limit for size of failures to be included in the study was arbitrarily set at 30 cubic yards, as estimated by the field observers. The largest observed slide, located at Balsam Gap on U.S. highways 19A and 23, was estimated to contain in excess of 1/2 million cubic yards of material, but the average size for all slides studied was calculated to be approximately 6,000 cubic yards.

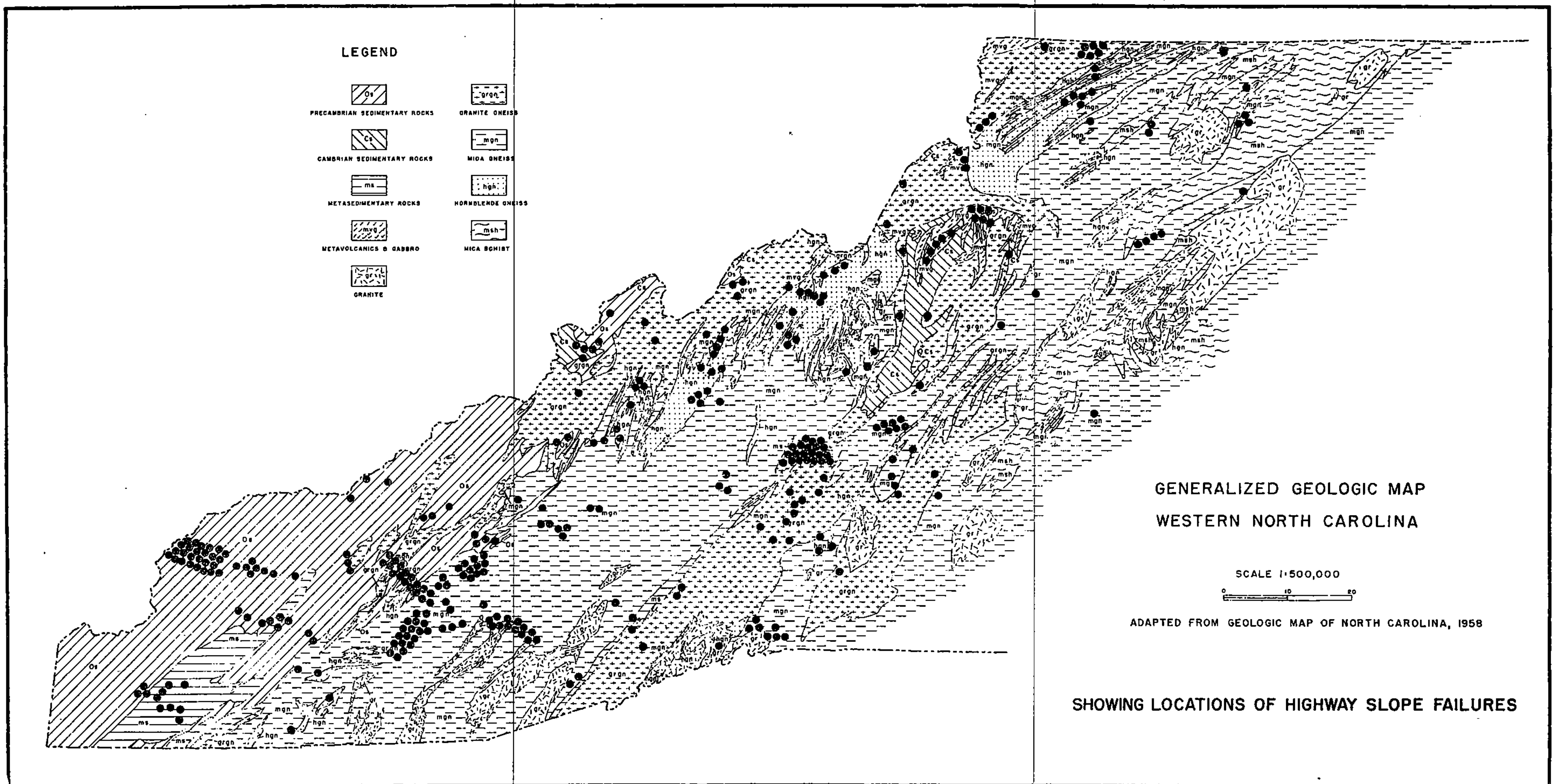
The Highway Research Board classification of slope instability[1] is based on type of movement and type of material, but little has been done to apply principles and theories of rock mechanics and weathering to the problem of landslides and subsidences. No significant advances can be made in the application of mechanics theories to slope design, or in the application of geological and agricultural soils information to route location, until it is known which of the soil or rock formations are slide susceptible, why they are susceptible, and what physical behavior may be expected of these formations under various types of environmental conditions. The large number of slope failures observed in the current study occurs in a variety of rock and soil types and under several different climatic conditions. Because of this large number it is not feasible to study each individual failure in detail. Furthermore, the variety of conditions under which the failures have occurred rules against detailed study of a single failure for the purpose of applying the conditions and principles learned to all of the various types of failures. Thus, the method of approach which has been adopted is based on the study of all of the slides as a unit, in an effort to recognize a factor or factors which may be common to many or all of the failures. Once these factors are recognized, they may be evaluated by applying them to detailed studies of a few individual slides. This quasi-statistical effect-to-cause approach, using data derived from many failures, apparently is unique in the study of landslides. Most reports appearing in the literature concentrate on the detailed analysis of a single failure and may or may not interpret the results in terms of their application to other cases.

The distribution of slope failures in the mountain and piedmont counties of North Carolina is shown on Figure 1, which is a generalized geologic map of the western part of the state. The slides are not distributed uniformly throughout this area, but occur in groups and clusters. Several factors are responsible for this seemingly erratic distribution, one being the location of highways and related construction. Other less obvious factors also influence this distribution, and the identification of these factors is a major objective of the current investigation.

Failures in Soil and Weathered Rock Material

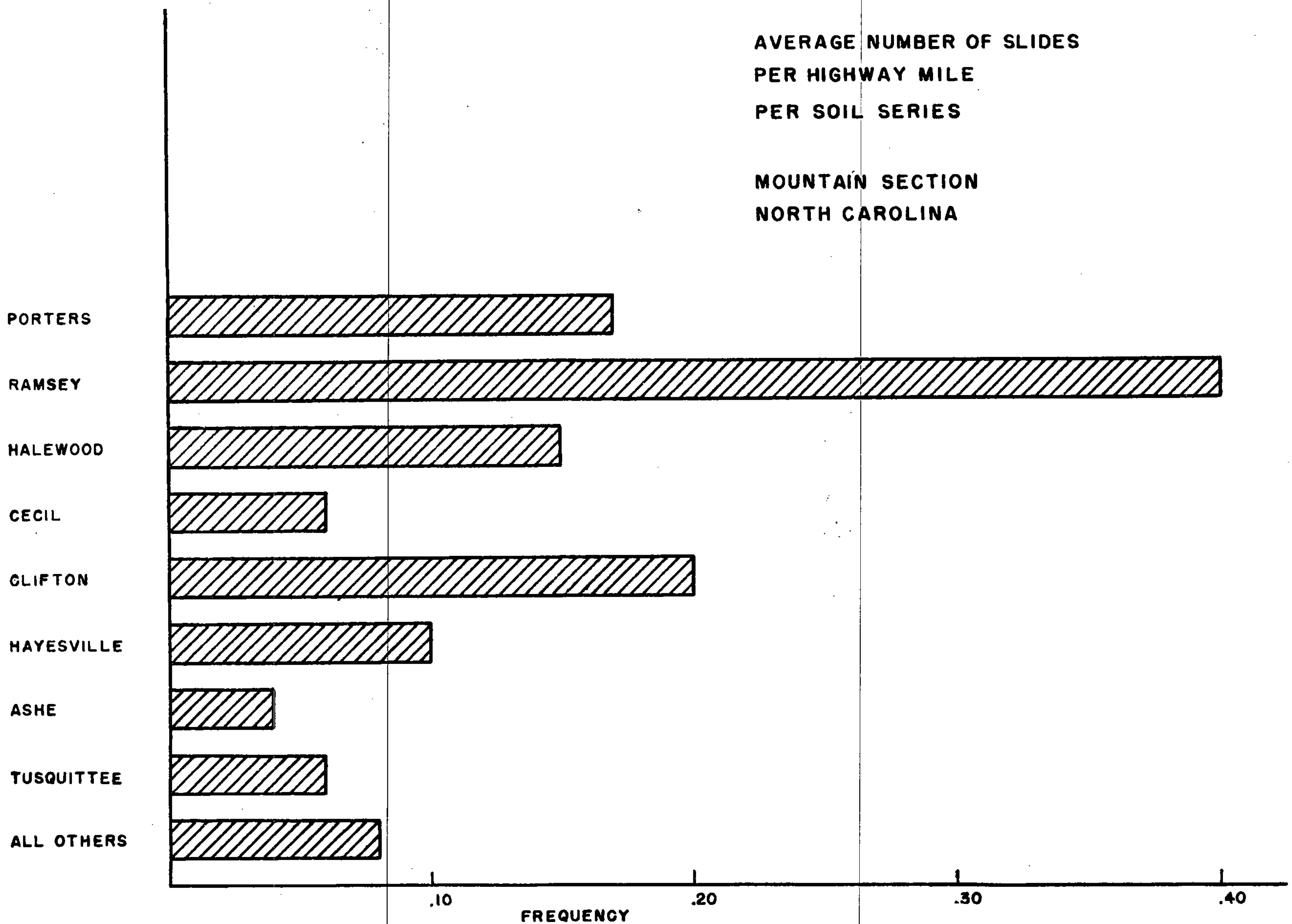
Influence of Soil and Rock Type

More than two-thirds of the observed slope failures occur in deeply weathered saprolite and soil material; the remainder occur in rock. The frequency of slope failures per mile of highway in the mountain counties has been computed in terms of the agricultural soil series in which the failures occur (Figure 2). Since the soil series depend in part on the parent rocks from which they were derived, the slide frequency - soil series relationship should indicate indirectly which of the parent rock types are most subject to failure. In the valleys, more miles of highways and fewer road cuts are found than on the ridges and topographic divides. However, the classification of soil series takes into consideration the topographic position in which the soil occurs, so this topographic factor does not invalidate the frequency rate. For the various soil



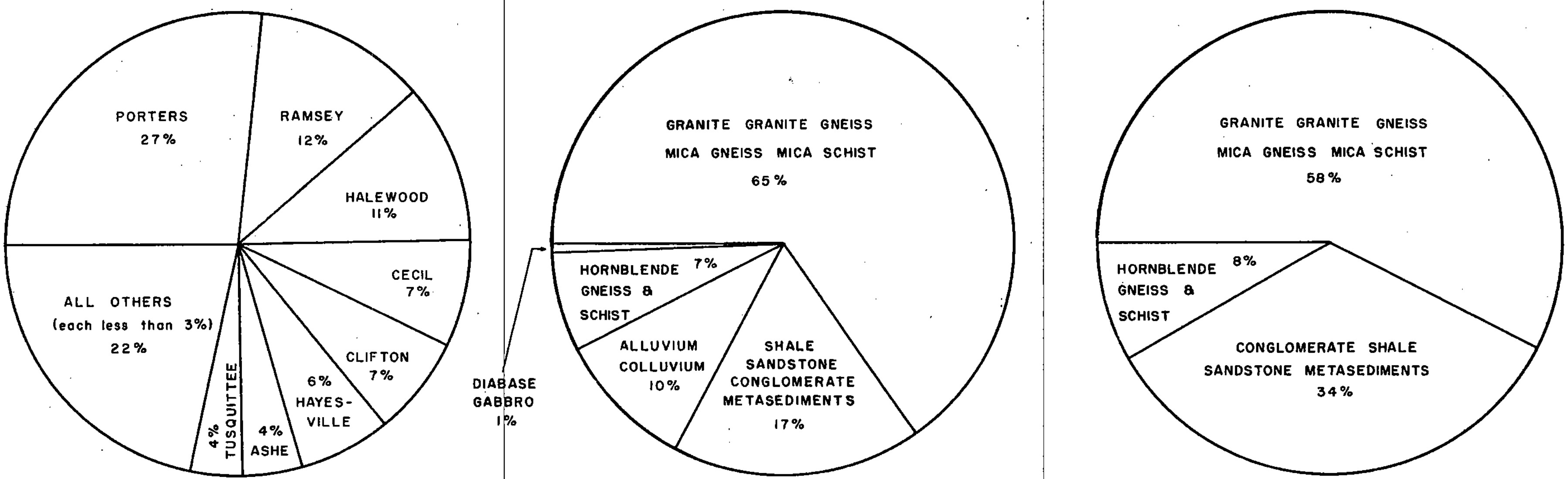
LEGEND
PRECAMBRIAN SEDIMENTARY ROCKS
CAMBRIAN SEDIMENTARY ROCKS
METASEDIMENTARY ROCKS
METAVOLCANICS & GABBRO
GRANITE
GRANITE GNEISS
MICA GNEISS
HORNBLENDE GNEISS
MICA SCHIST
GENERALIZED GEOLOGIC MAP
WESTERN NORTH CAROLINA
SCALE 1:500,000
0
10
20
ADAPTED FROM GEOLOGIC MAP OF NORTH CAROLINA, 1958
SHOWING LOCATIONS OF HIGHWAY SLOPE FAILURES

Figure 1.



AVERAGE NUMBER OF SLIDES
PER HIGHWAY MILE
PER SOIL SERIES
MOUNTAIN SECTION
NORTH CAROLINA
PORTERS
RAMSEY
HALEWOOD
CECIL
CLIFTON
HAYESVILLE
ASHE
TUSQUITTEE
ALL OTHERS
.10
.20
.30
.40
FREQUENCY

Figure 2.



SOIL SLOPE FAILURES BY SOIL SERIES

SOIL SLOPE FAILURES BY PARENT ROCK TYPE

ROCK SLOPE FAILURES BY ROCK TYPE

SOIL SERIES	PORTERS HALEWOOD CECIL HAYESVILLE ASHE FLETCHER MADISON CHANDLER TALLADEGA CHEWACLA FANNIN	RAMSEY HABERSHAM	ALLUVIAL TUSQUITTEE TATE CONGAREE TOXAWAY	CLIFTON	RABUN
PARENT ROCK TYPE	GRANITE GRANITE GNEISS MICA GNEISS MICA SCHIST	SHALE SANDSTONE METASEDIMENTS CONGLOMERATE	ALLUVIUM COLLUVIUM	HORNBLENDE GNEISS & SCHIST	DIABASE GABBRO

DISTRIBUTION OF FAILURES

Figure 3.

series, the range of slope failure frequency per mile of highway is from 0.04 to 0.40, the average for all soil series being 0.12.

Porters, Ramsey, and Clifton soils have the highest failure rates, and the lowest rate is in Ashe soil. According to the U. S. Department of Agriculture Soil Conservation Service,[2] Ramsey soil generally occurs on hillsides, mountainsides, and sharp mountain ridges and peaks. It is derived mainly from weathered products of conglomerate, sandstone, quartzite, shale, and other noncalcareous or only slightly calcareous rocks of sedimentary origin. This soil has a profile that is generally shallower than most of the other soils investigated. Porters soil also occupies hillsides and mountain slopes, and is characterized by a very friable surface soil and subsoil. It forms from the residuum of weathered low-micaceous igneous and metamorphic rocks, principally granite and felsic gneiss and schist, but to some extent also from hornblende gneiss, hornblende schist, diorite, and similar rock types. Clifton soils, except for a redder subsoil, are similar in topographic position and origin to Porters soils. Ashe soil, with the lowest frequency rate, is a light-colored mountain upland soil derived mainly from a residuum from weathered granite, felsic gneiss, and schist, all low in mica content.

On the basis of slope failures occurring only in soil material, the percentage of the total occurring in each soil series is shown as a pie diagram on Figure 3. Ramsey and Porters soils lead with the highest percentages, 12 percent for Ramsey and 27 percent for Porters. Figure 3 also includes a table showing the parent rock material from which each of these soil series presumably was derived, and a second pie diagram on which are shown the percentages of slope failures occurring in each of the respective parent rock types. The third pie diagram on Figure 3 represents the 111 slides which occurred in rock, rather than soil, material, showing the percentage that occurred in each rock type. Comparison of the second and third pie diagrams indicates that granite, granite gneiss, mica gneiss, and mica schist are the rock types which are most susceptible to failure, either as unaltered rock or as weathered soil material derived from the rock.

<u>Influence of Climate and Weathering</u>

Weathering is an important consideration in slope stability, and climate is one of the principal factors controlling weathering. In order to evaluate these climatic effects in the North Carolina mountain counties, temperature and precipitation records were obtained from the U.S. Weather Bureau, and 30-year average figures were plotted on maps at the locations of each of the weather observation stations. Isotherms were drawn connecting points of equal average annual temperature, and isohyets were drawn in the same way connecting points of equal average annual precipitation. Although the numerical values assigned to these contours may not have been constant during the period of soil formation, the contour patterns should represent the relative climatic conditions.

Within the area included on the isothermal map (Figure 4) the range of average annual temperature is from 49°F to 60°F, with the isotherms drawn at 1°F intervals. The frequency

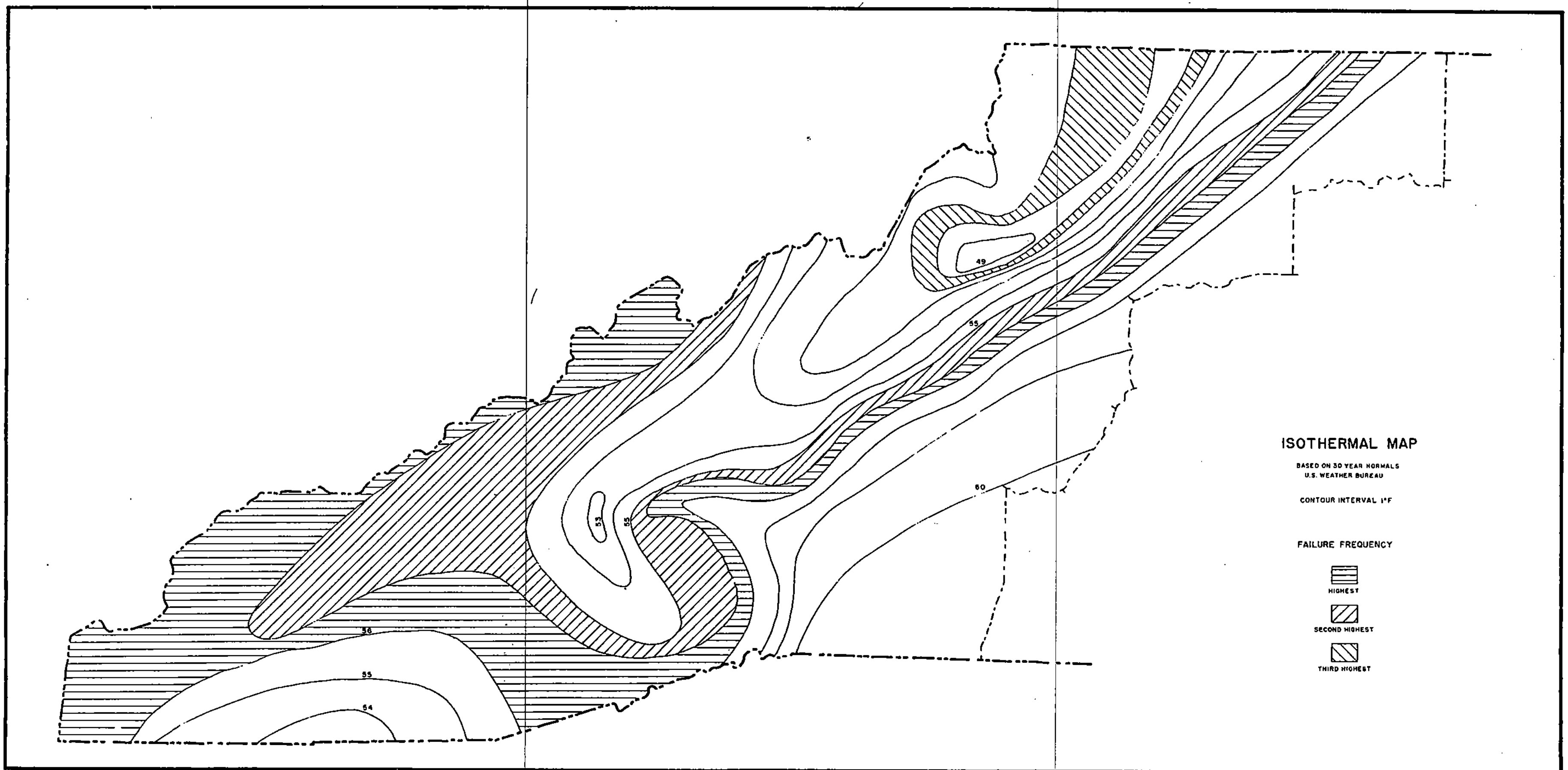

ISOTHERMAL MAP
BASED ON 30 YEAR NORMALS
U.S. WEATHER BUREAU
CONTOUR INTERVAL 1°F
FAILURE FREQUENCY
HIGHEST
SECOND HIGHEST
THIRD HIGHEST
49
55
53
55
60
56
55
54

Figure 4.

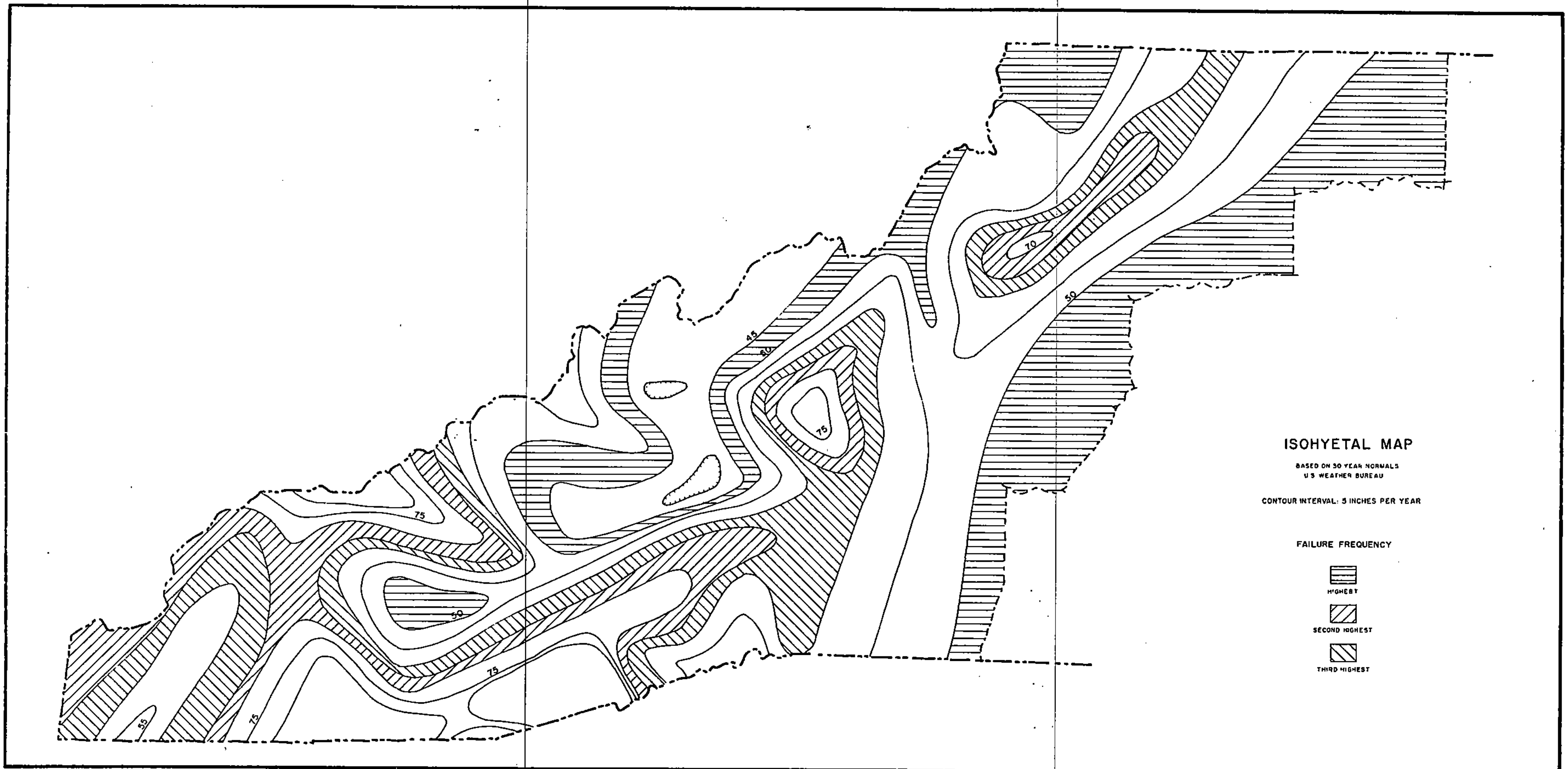

ISOHYETAL MAP
BASED ON 30 YEAR NORMALS
U S WEATHER BUREAU
CONTOUR INTERVAL: 5 INCHES PER YEAR
FAILURE FREQUENCY
HIGHEST
SECOND HIGHEST
THIRD HIGHEST
70
50
75
45
50
75
50
75
75
55

Figure 5.

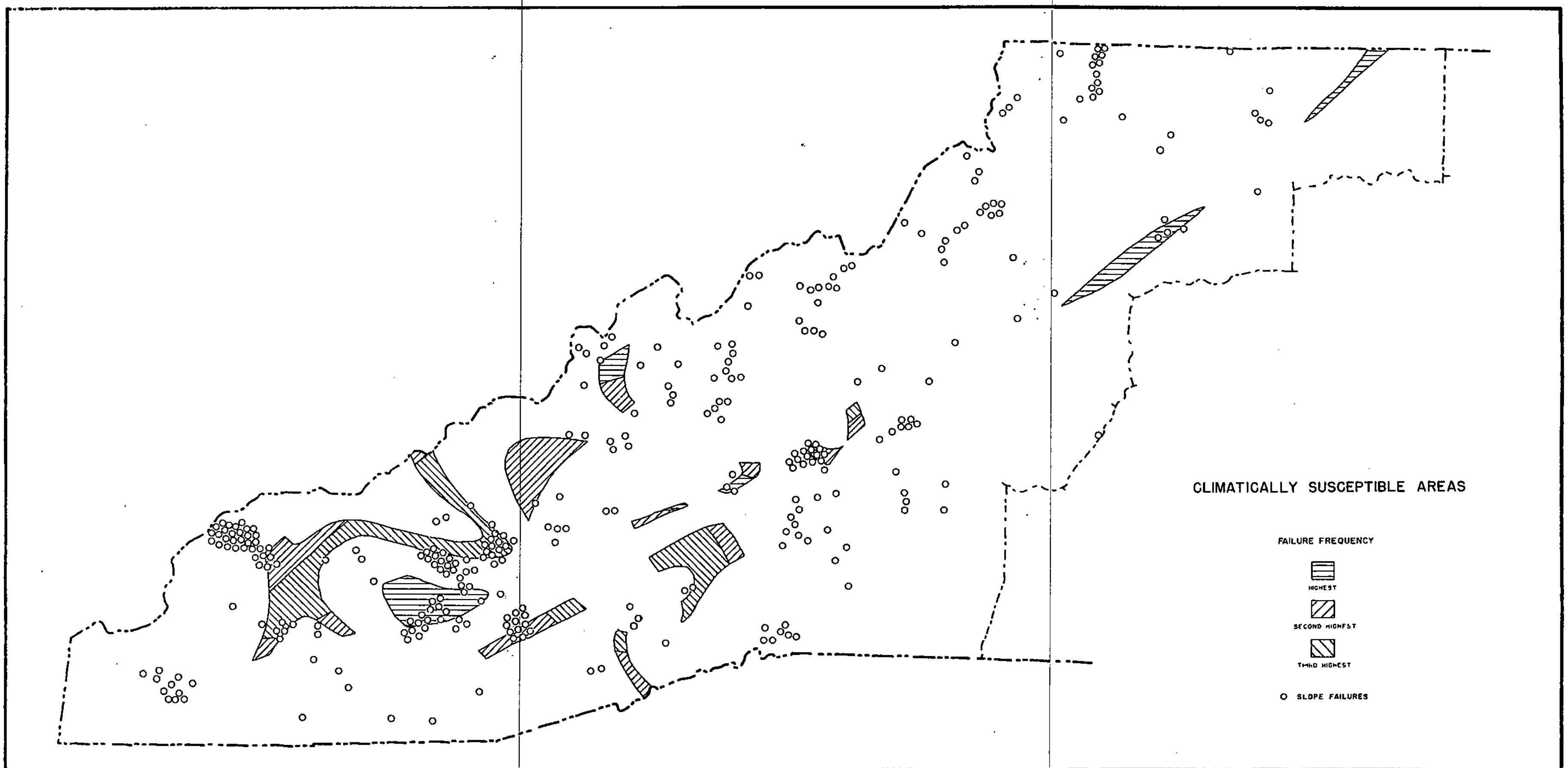

CLIMATICALLY SUSCEPTIBLE AREAS
FAILURE FREQUENCY
HIGHEST
SECOND HIGHEST
THIRD HIGHEST
SLOPE FAILURES

Figure 6.

of slope failures per square mile has been computed for each $1^{o}F$ isothermal band, and the three bands in which the frequency rates are highest are indicated on the mpa. The maximum frequency occurs in the 56-57° band, with the 55-56° band next in importance. Being contiguous, these two bands have the same geographic distribution, and are found along the Blue Ridge Front, the northeast-southwest trending topographic break between the mountain and piedmont areas. Near the southwestern corner of the state the isothermal trends swing westward to the Smoky Mountains, and then return to a northeast-southwest orientation in response to the Smoky Mountains alignment. The third highest slope failure frequency rate occurs in the 50-51° isothermal band, which is located in the northwestern part of the state in area of high rolling plateau topography.

On the isohyetal map (Figure 5) the range of average annual precipitation is from 40 inches per year to 80 inches per year, and the isohyets are drawn at 5 inch intervals. A few anomolous local areas in which the annual rainfall exceeds 80 inches are present within the mountain counties, but they have been disregarded in drawing the isohyetal map. As was done with the isothermal map, the frequency of slope failures per square mile for each of the isohyetal bands was computed and the three having the highest frequency rates are indicated on the map. The highest rate is in the 45 to 50 inch band, which lies in part along the Blue Ridge Front and in part along the eastern margin of the Cumberland Plateau, at the western edge of the mountains. Topography exerts considerable influence on this western zone, especially in the southwestern part of the state. The bands of second and third highest slope failure frequencies occur between the 65 and 70 inch isohyets and the 60 and 65 inch isohyets, respectively. From the Smoky Mountains area in the southwestern corner of the state these bands extend northeasterly along the trend of the Appalachian Mountains, and are found also in the plateau area near the northwest corner of the state.

In the areas of greatest density of slope failures, the maximum frequency band of the isothermal map, by construction, coincides in part with the maximum frequency band of the isohyetal map. However, if these maps are superimposed other areas of coincidence are seen, including some in which maxima of one map coincide with maxima of the other map, and some in which lesser orders of failure frequency are coincident. On Plate 6 all of the areas of coincidence giving highest combined frequency rates are outlined, and the relationship of existing slope failures to these areas is evident from the landslide locations, which are also plotted on this map. Perhaps of even greater interest, because of their possible failure potential, are the outlined areas in which few if any slides occur. In some of these areas there are no highways, and therefore no constructed slopes, but in a few of these areas roads are present and topography is such that cuts should be required. The fact that slope failures do not occur here suggests that factors other than climate exert a strong counter-influence negating the susceptibility induced by climatic conditions, or that climatic factors by themselves are not sufficient to produce failures, and must be reinforced by other, as yet undetermined, factors.

Failure in Rock Slopes

Failures in rock slopes, although not as common as those in soil slopes, occur throughout the northern part of the area studied. Bedded sedimentary and metasedimentary rocks, and igneous and metamorphic rocks exhibiting well developed layering or schistosity apparently are most susceptible to failure, presumably because the bedding and foliation planes constitute potential failure surfaces. Slides occur most frequently in road cuts which expose these types of rocks in an orientation such that the structural surfaces dip toward the road. An example of this type of failure is shown on Figure 7, which is a photograph of only one of ten similar slope failures in a nine-mile stretch of divided highway, on U.S. 70 near Asheville. The cut on the opposite side of the roadway, in which the bedding surfaces dip away from the road, has not failed. In this same stretch of highway, cuts in which the structural features of the rocks dip at some angle other than toward the highway also remain stable. Joints and fractures in the rocks do not provide sliding surfaces for slope failures, but they do exert an important control in determining the lateral and up-slope extent of the failure area. They also provide a mechanism by which blocks or rock material are loosened and separated from the surrounding rock. Movement along the failure surface is promoted by the presence at this surface of clay, shale, or other material having lower shear strength than the adjacent rocks.

Conclusions

Rock Slopes

Rock slope failures appear to be controlled largely by the presence of structural planes of weakness, if these planes are oriented in a way to provide easy relief of gravity induced stresses.

Soil Slopes

Interpretation of the geologic and climatic data indicates that if two different rock types, mica gneiss and meta-sandstone, for example, are weathered together under various climatic conditions, a relatively constant difference in degree of weathering should be observed. Furthermore, the soil series ultimately derived from these rock types should exhibit similar constant relative differences in physical characteristics regardless of the particular climatic conditions in which they are found. The two rock types and their derived soil series thus should maintain their positions relative to each other, in terms of slope failure frequency, if rock type by itself is a dominating cause of failure. However, in some cases there is greater variation in frequency of slides from area to area within a given soil series than there is from one soil series to another. This seems to suggest that within reasonable limitations the original

Figure 7. Rock slide in road cut on U. S. 70 near
Asheville, North Carolina.

mineralogy and texture, common to a given rock type, exert less influence on the engineering properties of the derived soil material than do other factors that may be common to several different rock types, but are not necessarily characteristic of any single species. Field investigation suggests that among the possible common factors which could have local variations sufficient to produce slope instability are (1) physical characteristics and mineralogy of the weathered products, (2) orientation of relict structures, and (3) the groundwater conditions affecting the material before, during, and after construction of the slope. The influence and significance of these factors is currently being investigated.

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THE DEVELOPMENT AND UTILIZATION OF ENGINEERING GEOLOGY

IN THE CALIFORNIA DIVISION OF HIGHWAYS

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Introduction

It had been the original intent to present a certain phase of engineering geology as practiced in California, but after poring over the proceedings of the past symposiums, I found I could add little to what had already been presented. As the title implies, and for the benefit and information of future symposiums, this paper will be broad in its scope.

In Vol. 1, No. 1, of the California Highway Bulletin 1912, now published as California Highways and Public Works, it states "There would be an expenditure of $18,000,000 for construction of 2500 miles of public roads." On the payroll at that time, 1912, there were 306 employees. Among these was a geologist, Clarence B. Osborne, who served as materials engineer for the period 1912 - 1918 and held the title of chief geologist. Therefore, California was among the first, if not the first state, to employ a permanent staff geologist in its highway organization. During this embryonic period the tasks of the geologist were many and varied, and it is doubtful that applied engineering geology was used to the extent that it is today.

From 1920 to 1937 there is no record that engineering geologists were employed, although there were several mining engineers under engineering titles.

It was not until 1937 that a mineral technologist was hired, primarily for research into the reactive aggregate problem, and also for the study of aggregates, with minor attention to cut slope design or foundation investigations. During this time some experimentation was undertaken using the seismic and resistivity methods of exploration.

In 1944 engineering geologist classes were established in the State Civil Service and personnel were hired in these classes. Programs of training and research were then initiated in the fields of foundation exploration and soil mechanics for the purpose of training geologists in these areas.

On the staff of the Division of Highways today are approximately 25 engineering geologists, three mining engineers and two highway technicians with degrees in geology. Six are employed by the Materials and Research Department and the remainder

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are assigned in three of the district offices. The Bridge Department, which functions as a separate unit in the exploration for structures, employs 16 geologists divided between the north and south sections of the State.

The value of geology to highway construction is now recognized and the services of geologists are constantly being applied to all fields of engineering geology.

Construction Materials

California, like other states, due to its geography, geomorphic provinces and geographic boundaries of its 11 highway districts, has difficulty in developing suitable construction materials in some areas. Urban development is becoming a factor in locating sources of material within economic haul. This is becoming more noticeable in the metropolitan areas where the construction of high type freeways and structures are contemplated. Therefore, one of the most important functions of the geologic staff is the continuous search for satisfactory construction materials in those areas which are deficient, and the development of new areas where proposed future construction is in the long-range stage.

After a possible source of materials has been located, the geologic factors at the site frequently control its economic usefulness. The extent of the deposit, the amount of overburden, the degree of weathering, the presence of deleterious rocks and minerals must all be known. The geologist can use several methods for exploring the site: (1) geologic mapping; (2) photogeology; (3) geophysics; (4) borings.

The geologist can provide valuable assistance in laying out a program of exploration; sampling and testing a source of material will give results which will be indicative of the quality of material that can be produced.

Geologic mapping is the basic tool used by the geologist. By working in the field, the geologist maps the areal extent of formations and their structures. In recent years photogeology, the technique of plotting geology from aerial photos, has increased in use and has saved many hours of arduous field work.

Geophysical methods are profitably applied in certain instances. Seismic surveys, using the principle of refracted waves, can determine depth of overburden or weathering, or when applicable can be used to quickly extend information from a boring. Quarry sites can be delineated in areas not accessible to equipment. Resistivity surveys, using the principle of electrical resistance of material, can sometimes be used to detect changes in lithology of sedimentary material. It, too, under favorable conditions, can rapidly extend the area of validity of borings.

In the laboratory, additional information can be obtained by various methods. For instance, a petrographic study will determine the percent of reactive material in an aggregate and, in the studies of riprap and quarry rock to be crushed for aggregates, thin sections are prepared and a complete petrographic analysis is made.

Cut Slope Design

The first step in a cut slope design is to get the broad picture. This involves soils and geological examinations of surface conditions, searches for evidences of stability or instability, and studies of the performance of any existing natural slopes and cuts in the area. It also includes a look at features that can change the stability of a slope, such as natural changes in ground water conditions, earthquakes, etc., and man-made changes such as aggravated ground water caused by infiltration from adjacent lawns, broken water mains, leaky sewer lines, etc.

The information needed for the economical design of cut slopes includes the nature and strengths of the materials that will be excavated, ground water conditions, the attitude in beds of sedimentary rocks, the degree of weathering, the extent of joints, bedding planes, fractures, and other surfaces of potential weakness, and the presence of landslides, active or ancient. To obtain this information several of the following methods commonly are utilized: visual inspection on the ground, study of topographic maps and airphotos, model construction, geological surveys, geophysical explorations, explorations by borings, either vertical or horizontal, evaluation of boring data, ground water observations, and testing of undisturbed soil samples or rock cores.

In the California Division of Highways an effort is made to avoid cut slope failures, but it would be necessary to be very conservative to avoid all failures. When any of importance occur they are studied very carefully to try to determine the causes. Sometimes the causes are obvious; other times they are very obscure.

The stability of cut slopes usually can be improved by one or more of the following methods:

1. Changing highway alignment or grade.

2. Flattening slope or unloading upper part of cut slope.

3. Adding support at the toe in the form of buttresses.

4. Drainage.

Obviously, for reasons of economy, cut slopes should be as steep as possible, consistent with stability. Typical slopes for various materials might be somewhat as follows: in cohesionless sands the slope should be no steeper than 1 1/2:1, since this is about the angle of repose for such soil; in cohesive soils, containing silt and clay along with sand, slopes of 1 1/2:1 or flatter are usually in order; in cemented sediments, such as sandstone, shale, and conglomerate, steeper slopes may be used, depending on the degree of cementation, bedding, jointing and ground water conditions; in weathered rock, slopes may vary from 3/4:1 to 2:1, again depending on degree of weathering, ground water, etc.; and in hard fresh rock slopes as steep as 1/2:1 or 3/4:1 are sometimes possible.

After all the factors and information have been obtained an evaluation is made by the different sections and a slope design based on the analysis of the data and opinions is made for submittal to the headquarters and district design departments.

Landslides

Diversity of formations, complex fault systems, and variable climatic conditions accentuate the problem of landslides in California. Almost every classification of slide can be found, and it is impossible to set up a standard method of correction for the many types of slides found in the state.

Preliminary geologic studies of proposed routes have been valuable in avoiding areas of unstability. When these areas cannot be avoided, a comprehensive study and investigation is made to determine what methods must be used to furnish the greatest stability to the roadway. Horizontal drains, stabilization trenches, stripping and struts are a few of the methods most frequently employed.

Approximately 70 percent of the slides occur in the Northern Coast Range Province, although a few of the more spectacular and complex of treatment have occurred in the seemingly stable Sierra Nevadas.

Geophysical Exploration

In 1935, the late E. R. Shepard, research engineer with the Bureau of Public Roads, demonstrated the use of the resistivity and seismic methods in highway exploration to personnel in the Division. Using plans furnished by the Bureau, the Materials and Research Department assembled seismic and resistivity equipment. This equipment was crude when compared to present-day instrumentation but the prinicple and interpretation were the same. The equipment did not receive widespread use and the lack of trained personnel in the interpretation of the records left much to be desired when a knowledge of the geologic conditions is essential.

In 1947 the Department pruchased a 12-channel Century seismograph, a Michimho vibroground, and a Bureau of Public Roads resistivity instrument. This equipment has had fairly continuous use in all sections of the state and is generally used to extend information from boreholes and to obtain subsurface data in areas not accessible to drilling equipment. It is used in the exploration of aggregate sites, quarries, cut slopes, fill foundations and in rippability studies.

In 1962 the Department purchased one of the hammer type seismographs, and a 12-channel ER-75 Porta-Seis interval timer. This new equipment has decreased field time and reduced personnel, resulting in a considerable saving in job costs.

In formations where good correlation with existing exposures or correlation with other data such as borings can be obtained, the seismic method of exploration is a valuable tool. It develops information along a continuous profile, whereas boring data are valid only at a specific location. Therefore, when the seismic method of exploration can be used in combination with other means of investigation the data thus obtained may be relatively economical.

Normally a three-man crew is required which will be able to make eight to twelve seismic tests per day at a cost of $100 to $150 per day.

In addition to the necessity for correlation with observed or known conditions, some other deficiencies are: poor results in areas of high organic content such as peat, the difficulties of using explosives in populated areas, and questionable results due to the heterogeneous nature of the formations being explored.

In the earth resistivity method of investigation, a crew of 2 to 4 men is used. This operation costs from $100 to $200 per day. Such a crew can made 15 to 20 tests per day. The data can often be interpreted in the field. Thus, the information obtained is frequently less expensive than that obtained by seismic studies or other means of investigation. It is almost always necessary to supplement such data with other types of exploration.

It should be kept in mind that optimum conditions are necessary in order to obtain satisfactory results from the earth resistivity method of exploration. These conditions are: appreciable differences in resistivity of soils involved, stratification parallel to the ground surface, and sufficient area of uniform soil profile in the zone to eliminate side effects.

Some conditions that may cause difficulties are: stray currents leaving cross-country pipe lines or emanating from electric railway systems in urban areas, or buried utilities such as water and gas pipes.

Conservative interpretation, geologic knowledge, and the use of borings have proven the value of geophysics as an aid to highway construction in California.

Photogeology

The use of photogeology is rapidly becoming one of the most important working tools of the geologist, saving many weeks of field work. Proper interpretation of air photos can, if correctly applied, save thousands of dollars in all phases of highway construction.

Photogeology, borings and geophysics in the order named, will be the means more frequently used to keep pace with the expanding highway program and the increased demand for greater quantities of construction materials.

Special Investigations

Another field in which geologists are frequently called upon is that of furnishing testimony and evidence as expert witnesses in condemnation and damage suits brought against the Division.

In one instance geology, geophysics, photogeology, and cores were all used as evidence which resulted in a verdict favorable to the state and an estimated saving of over $250,000.00. At the present time there are two court cases pending in which Division geologists are being used as expert witnesses.

Secondary Alteration

Considerable research is being made in the area of secondary mineral alteration of construction materials. It has been found that certain rocks, principally the volcanics, are subject to breakdown and progressive failure after being used in the roadway, stockpiled, or placed on slopes. Thin sections, and petrographic analysis, followed by further physical tests and field studies are a few of the methods being used to detect alteration before the rock is accepted or rejected for use.

Not all volcanics are suspect, and to date it has been found that certain granites, ultra-basics and sandstones are susceptible to secondary alteration or breakdown. As the locating, production and use of all types of construction materials are major items of highway development, continuous research is being carried on, and it is hoped that in the near future sufficient data will be obtained for publication.

Clay Mineralogy

During 1962 Differential Thermal Analysis has been extremely useful in the identification of clay minerals and good correlation has been obtained between the expansion pressure tests and the D.T.A. It is also used in the investigation of the clays from slip surfaces of landslides, gouge from fault zones and clay and soil cores from borings. The recognition of the presence of montmorillonite clays in base and subbase materials can readily be determined by the D.T.A. and has resulted in a closer control on suspect or borderline material.

Recently, X-ray diffraction equipment has been purchased and it is planned to continue research of the clay minerals in relation to construction materials using both D.T.A. and X-ray methods.

Conclusion

The engineering geologist should be ready at all times to furnish to the engineer any information he has or can obtain. In this manner the engineer can proceed in the preliminary stage to plan ahead on projects that might be delayed pending a formal report. A mutual understanding between the geologist and engineer is essential and this can only be obtained by recognizing each others problems.

Acknowledgements

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Fig. 1 - Aerial View of Completed Carquinez Cut and Southern Approach to Twin Bridges

The "Big Cut"
(A Case History)

One of the main highways giving the population of the San Francisco Bay area access to inland areas in U. S. Highway 40 which skirts the northeast shores of San Francisco Bay, crosses over the Carquinez Straits, progresses northerly a few miles and then heads easterly across the state. When the original Carquinez Bridge was completed in 1927 it was one of the nation's outstanding steel bridges, and the highway was very adequate for the traffic of the times. As the years passed, this highway became more and more congested because of insufficient lanes and innumerable bottlenecks in the many small towns through which the road passed. A bold solution to this traffic bottleneck was the construction of a second bridge parallel to the 1927 structure, and the rerouting of the highway across Tormey Valley east of the original highway. The chief obstacle in the way of the new routing was a large hill just south of the Straits. The projected line penetrated this hill to a maximum depth of nearly 350 feet. The natural terrain was standing on slopes approximating 3:1 or flatter, and many slide scarps were visible, indicating surface sloughing. The area was known to contain two active faults: the Franklin Thrust and the Mare Island Fault; and the region is subject to seismic activity. The earthquake record of the region since 1854 shows four shocks with intensities of X on the Rossi-Forel scale and 58 of lesser intensity. Studies were made of the relative cost of tunnel construction vs. open cut. The high cost and uncertainties of future permanence eliminated tunnel construction as a practical alternate. An investigation was then made in a study of construction by open cut methods, and subsequently the construction was made in this manner (See Figs. 1 and 2).

In the fall of 1953 and the spring of 1954 several deep exploratory core holes were made in the proposed cut area. In general the deposits were soft interbedded shales and sandstones of the Cretaceous-Paleocene Age, but the sediments ranged from hard sandstone to soft friable sand, from firm silty shale to soft clay-shale. Substantial evidence of ground water was disclosed by the borings. When deposited, the sediments that form this region were uniform and competent, but intensive folding and faulting has greatly weakened the masses. The locations of borings are shown on the plan of the cut and on the typical cross-section (Fig. 4). All of the borings were bailed to within a few feet of the bottom of the casing, and the rising water level recorded for a period of time after the bailing to obtain an indication of the general permeability of the formations.

The character of the formations as determined by the explorations may be summarized by the following excerpts from the report of this foundation investigation:

> "In summary, it is believed that it is feasible to construct the proposed road...without serious risks. It should be recognized that the cuts would be high and the soil far from ideal. Even with proper slope design,

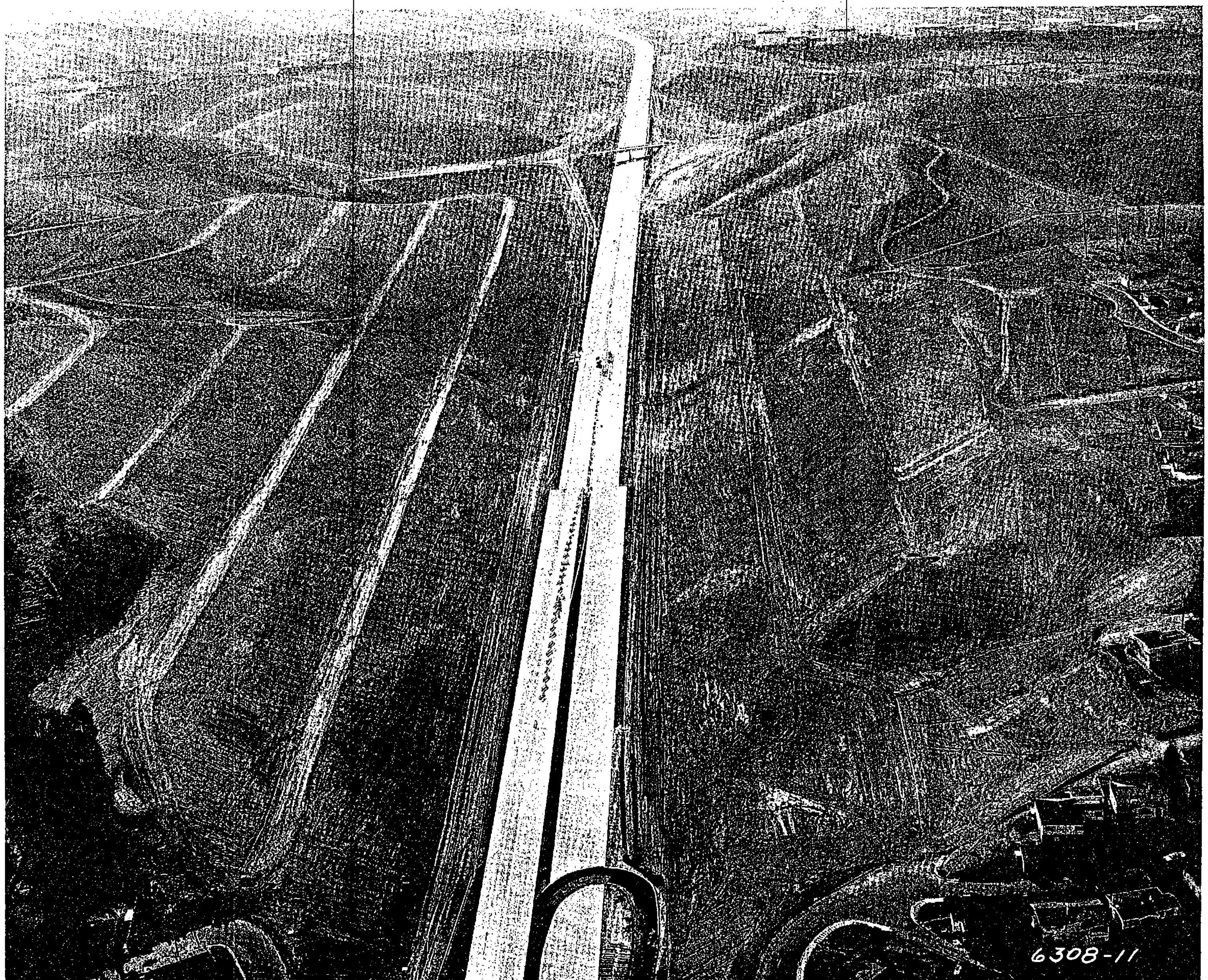

Fig. 2 - Aerial View Showing Magnitude of Carquinez Cut

some surface sloughing and minor slides can probably be expected. The possibilities of a major slide, one that would close all or even two or three lanes of the road, are very remote. It is believed that the risks involved...are not seriously greater than they are on numerous major roads where foundation design problems are complex."

To aid in visualizing the variations in conditions within this cut a scale geological model was constructed, and the deduced and actual geological conditions were shown on various surfaces of the model. A photograph of this model is reproduced in Fig. 3.

On the basis of all available information about the soils and geological conditions in the area, it was determined that this road could be designed as an open cut through the hill south of the Straits. The typical cross-section reproduced in Fig. 4 shows approximately the maximum section through the cut. The cut was designed with slopes of 2:1 and 30-foot wide benches at 60-foot vertical intervals. This cut was widened about 30 feet on each side at roadway grade to provide protection against the blocking of traffic lanes in the event large slides should take place after the road was opened to traffic. A substantial number of horizontal drains were to be installed at various levels in the cut.

As designed, the "Big Cut" had a length of about 3000 feet, a top width at the crest of 1370 feet and a maximum depth of 350 feet. The total volume was calculated to be more than 9,000,000 cubic yards.

Excavation of the "Big Cut" was started in late March, 1956, and completed in June, 1958. As the excavation was deepened, ground water levels were recorded in numerous wells and horizontal drains were drilled into areas where the water level did not drop rapidly with the deepening of the cut. On the whole, the designed and constructed slopes have been stable. From time to time small slides have occurred at various points on the faces of the cuts. One slide of rather major proportions took place at the north end of the cut in an area known locally as Valona. The extent of this slide in relation to the magnitude of the cut may be seen in the photographs reproduced in Figs. 1 and 2.

In February, 1958, some cracking was observed above the cut in the area on the west side of the cut, noted above. Subsequently, a retaining wall was badly cracked and noticeable cracks showed up in the basements of two of the houses at the top of the slope. The condition grew progressively worse, and several houses had to be removed and the slide mass removed. After this treatment this area showed no further evidence of instability.

In relation to the total valume of this cut, the slide was of rather nominal size. Nevertheless, approximately 125,000 cubic yards of earth were removed in correcting this slide. Had this cut been designed initially on a slope sufficiently flat to guarantee 100% security against slides, a rather enormous additional quantity of roadway excavation would have been required (several million cubic yards).

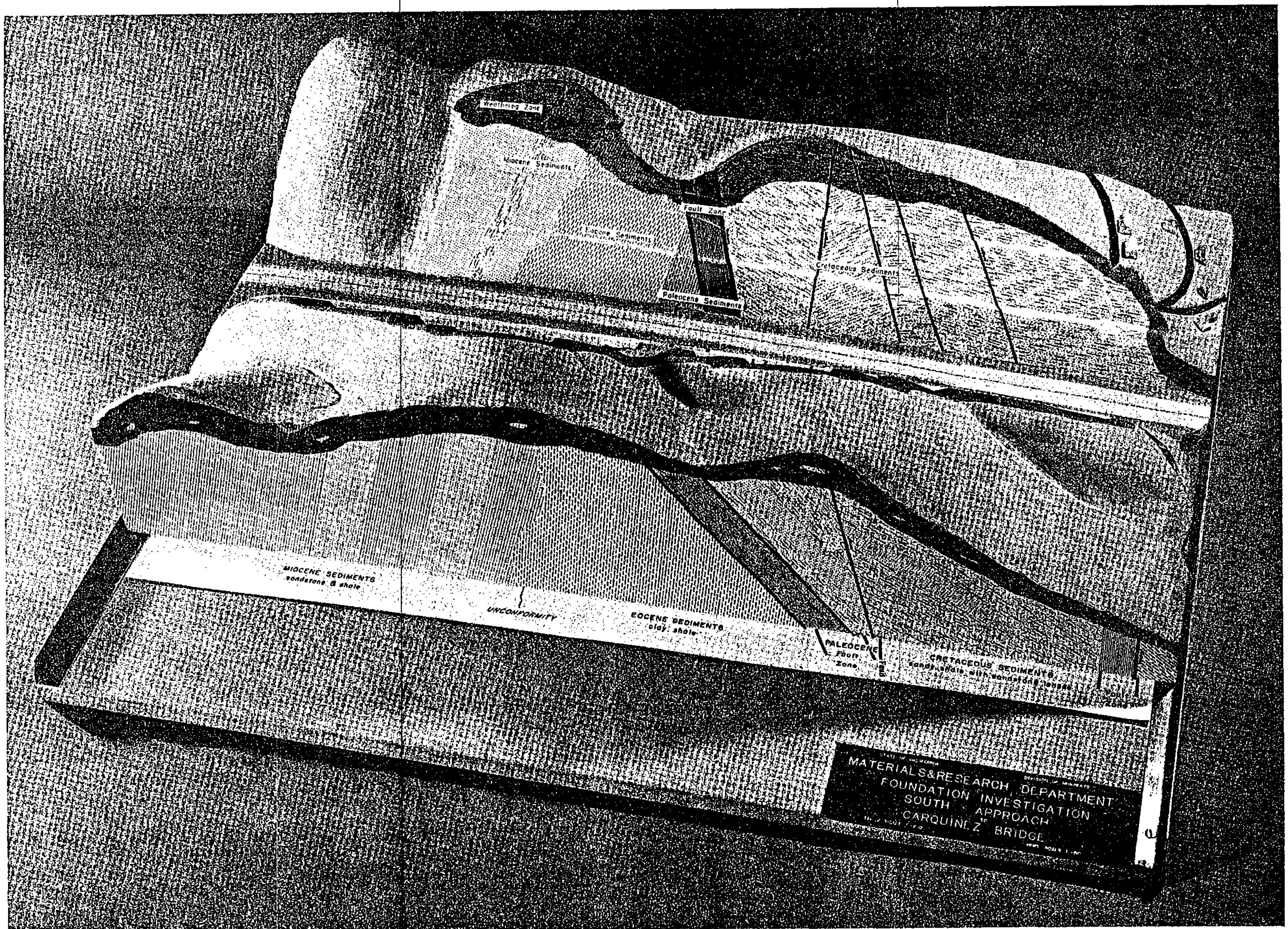

Fig. 3 - View of Model Showing Preliminary (Front Profile)
and Actual (Rear Profile) Geology of Carquinez Cut

This large cut is another example of a highway project in which soils and geological explorations and test borings provided very essential design information. The knowledge (information from the borings) furnished to prospective contractors, placed all bidders on a relatively even basis in judging the rippability of the formations and in estimating the costs involved in handling the excavation materials. Knowledge of the character of the formations and ground water conditions were extremely valuable in designing this cut.

On the whole, this excavation has been exceedingly successful. It is a tribute to the combined experience and judgment of the engineers and geologists who developed and executed its design.