PROCEEDINGS OF THE FIFTEENTH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Held
March 19, 1964
at the
Missouri Geological Survey
and
Water Resources
Rolla, Missouri

STATE OF MISSOURI
Department of Business and Administration
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GEOLOGICAL SURVEY
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CONTENTS

ACKNOWLEDGMENTS

The generous assistance of fellow staff members at the Missouri Geological Survey did much to make the Fifteenth Annual Highway Geology Symposium a success. Dr. William C. Hayes handled the greater part of the logistics and Mr. Jerry Vineyard, in addition to reading Mr. Migliacciao's paper, gave a slide-illustrated talk on Missouri's caves at the banquet. Mr. John W. Koenig guided the manuscripts and plates through the processes necessary to produce this publication. Those who presented papers made the Symposium a successful one, and their cooperation, promptness, and adherence to schedule is hereby acknowledged. The attendance and the geographical distribution of those attending was especially gratifying to the host and demonstrates that these symposia have established themselves as permanent and useful meetings.

Thomas R. Beveridge
Local Chairman

THE DUTIES AND TRAINING OF A GEOLOGIST IN THE OKLAHOMA STATE HIGHWAY DEPARTMENT

by

R. A. Helmer

The schooling of a civil engineer is to a certain extent directed toward highway work. Yet, a graduate engineer starting to work for a highway department has to become proficient in many new fields of study.

The duties of a geologist working for a highway department vary greatly from state to state. However, any highway geologist must also supplement his professional training with additional study.

In the Oklahoma Highway Department most of the geologists are in the Research Branch. Research requires knowledge in areas that are not usually covered in the training of either the geologist or the engineer. So, regardless of a man's formal education, it requires a rather long period of training and study to become a research scientist in the field of highway research.

To become indoctrinated with both highway work and research in highways is a rather large assignment. But, this kind of training is well worth the time and effort because it will greatly increase the value of a geologist not only in research but in all branches of highway work.

Our geologists make investigations and recommendations for the usual highway problems such as landslides, ground water, foundations, and material sources. These problems are closely related to geology. In the investigation of landslides we sometimes make geological surveys of new alignment to locate and avoid areas of potential slides. This involves a study of aerial photographs, investigation of the character and attitude of the rock strata, and a close study of the colluvial material, recognizing the possibility of this material sliding at the rock interface. We frequently investigate slides which have occurred on older highways to determine the cause. We then recommend a remedy. Since we find that ground water is almost always a contributing factor, the determination of the source of the water is necessary. We drill some test holes, but they are held to a minimum because of the difficulty of drilling in these sliding areas and the possibility that moving a dry hole just a few feet might have found the water. We supply the usual remedies of retaining walls, toewalls, and benching, but most of our effort is directed toward intercepting the water entering the slide area. We use subdrains and horizontal drilling for this purpose.

We have had excellent success with these horizontal drains. In the south-eastern part of our state the division engineer said he had solved about 90 per-cent of his landslide problems with horizontal drains.

Two inch perforated pipe is placed in 4-inch drilled holes. No backfill material is placed in the holes. This method has been used for many years in California, and I should give them credit for its development.

To the highway engineer, all material which reduces to a soil-like material under normal construction operations is considered to be soil. This includes many geological materials such as the softer shales and sandstones. Gravels and sands are also soils to the highway engineer.

Through the years engineers have developed many methods to classify and evaluate these so-called soil materials. All of these methods require testing of each material classified.

The development of so many methods is an indication that none of them has been found entirely satisfactory, probably because this kind of classification requires many tests which takes much time and is expensive. Some of the more common soils are encountered many times, and each classification requires new tests.

The principal tests we use in Oklahoma to classify soil material are the liquid limit, plasticity index, percent passing the No. 200 sieve, and the California bearing ratio is used to indicate the strength of a soil. We have developed an index number by means of which we can estimate the California bearing ratio from the plastic limit, liquid limit, and the percent passing the No. 200 sieve.

To hold the volume of testing to a minimum we have adopted a system of filing our test data for soil under the soil series name used by the Soil Conservation Service and the tests of the underlying geology under the geological name. The soil scientist and the geologist have accumulated much other information of value to the engineer. This system brings together in our files the information of the geologist, the soil scientist, and the engineer. When we know the name of a material, our records of previous tests can be used to evaluate it for highway purposes.

We frequently work from highway plans, and reading plans is a part of a geologist's training. For the engineering classification of materials, it is necessary to know the engineering tests, how they are interpreted, and how they enter into the specifications. Since our geologists classify both the geology and the soils, it is necessary for him to have a good working knowledge of all three methods of classification. Also, he should be able to identify by name the geology and the soil series in the field.

Basic to the study of pedological soils is the concept that all soils are the product of five soil forming factors. These factors are parent material, topography, biological life, climate, and time. Material, topography, and climate are major factors in the performance of highways.

A geologist must know what geological formations produce aggregate suitable for road building purposes. He must understand not only how the geology is reflected in topography, but how the topography is related to highway problems.

In the design of a highway pavement the principal governing factors are the material from which the pavement is to be constructed, the strength of the subgrade upon which it is to be placed, the traffic and wheel load it will carry, and a regional or environmental factor which is the combined effect of geology, soils, and climate.

The traffic data are available, but it is necessary to evaluate both the regional factor and the subgrade soils. This regional factor is very important and could change the required thickness of a pavement by as much as 100 percent.

The freezing and thawing and shrinkage and swell of engineering soils can rapidly destroy a pavement. The solving of this environmental problem has two parts; the identification of soils that are susceptible to damage by frost and study of the climatic records of the area to evaluate the degree days or frequency of temperatures that produce freezing conditions.

For our state the shrinkage and swell of plastic soils cause more damage to pavements than freeze and thaw. Here again it is a combination of soil type and wet and dry cycles of weather.

Our geologists spend a large part of their time doing research work. In fact, they are classified as research scientists.

Our research is of two general kinds. We investigate problems encountered by other departments, and we also carry on some rather large research projects in cooperation with the Bureau of Public Roads. We have had projects lasting several years and have investigated such things as flexible pavement, portland cement pavement, geologic materials, maintenance costs, and nuclear testing of moisture and density.

The collecting of data and the evaluation and analysis of the data are very difficult fields of study.

We use electronic computers in many of our projects, and an understanding of the equipment, preparation of data, statistical mathematics and methods is a part of a geologist's training in our organization.

In our research we use the performance of the pavement as the dependent variable and relate all the other factors to pavement performance.

The service of a pavement is determined by making a detailed field condition survey and noting all defects. Since this is a question of judgment, a man must be able to estimate the importance of the things he can observe in the field. Experience is required to become proficient in making these surveys, but we have geologists who can do an excellent job of making a condition survey.

Although it is rather unusual for a research department to design the thickness of pavement, this task is assigned to our research branch, and we have geologists who can make these designs.

Our geologists teach such subjects as soils, ground water, landslides, and geology in our training schools.

My purpose in giving this paper is not to paint a picture of the kind of a job a geologist will have if he works for a highway department, but to indicate the large variety of talents needed by the highway industry and to indicate the many areas in which the geologist can adapt himself to work.

I think there is a very narrow view held by both engineers and geologists of the place a geologist could occupy in the field of highways. For the good of the engineer and geologist, I want to correct this error.

Prospecting for and locating highway materials are similar to prospecting for oil and gas, and this is the first thing an engineer thinks of when considering the need of geological assistance.

It is true that materials are rapidly becoming depleted in many areas. Finding suitable materials is a major problem, but I have tried to indicate some of the many areas in which geology, supplemented by other knowledge, can be valuable.

There is work for the geologist in the planning, construction, and maintenance of highways. The larger a business gets the greater the variety of talent needed.

I recently heard an official of General Motors say that they had done extensive research on the shape of borrow ditches along highways. This research in the field of highway design was undertaken because cars are going to run off the road, and they felt that inasfar as possible cars should be designed to run into borrow ditches safely.

Highways are a multibillion dollars business. The highway problem covers an amazing number of areas commonly considered as functions of the doctor, lawyer, chemist, meteorologist, physicist, and many other professions.

The reasons for drivers going to sleep, the effect of an expressway on the abutting property values, the weight of a truckload of groceries, the origin and destination of a man going to work are all a part of the highway business. Most of these things enter into the highway problem in the form of observed data.

The geologist is trained in observation and the systematic recording of data, and this ability together with his training in the basic sciences such as mathematics, chemistry, physics, soils, and surveying gives him a good background for highway work.

I think those who are responsible for the teaching of geologists would do well to shape the education of these students a little less toward the production of oil and more toward the expanding field of furnishing the highways which are necessary to consume so much of the gas and lubricants which are products of the oil industry.

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GEOLOGY AND FOUNDATION PROBLEMS OF GLACIAL DRIFT IN EASTERN SOUTH DAKOTA

by

Walter W. Grimes

ABSTRACT

The eastern portion of South Dakota has been extensively glaciated. This glaciation covers about 34,000 square miles in the part of the state where two-thirds of the roads are built. The geology of the area is of real significance to the Highway Department because it governs the formation of the soils, gravel deposits as well as foundation material for structures.

The topography of the region is low and rolling with the drainage to the south by the James, Big Sioux, and Missouri Rivers. The area receives an average of 22.15 inches of rainfall and is covered with a Chernozem soil.

The pre-Wisconsinan glaciation is observed in rare instances and is identified chiefly by stratigraphic sequence. Four phases of Wisconsinan glaciation are identified and correlated with adjoining states. The Iowan substage is the most extensive in that it extends west of the present Missouri River. During the Tazewell and Cary substages a lobe was established down the James Basin and was divided from the Des Moines lobe in Iowa and Minnesota by the Sioux River.

Three examples of foundation problems and their solutions encountered in structure design and construction in Wisconsin till are discussed.

The first example in Marshall County is in old Lake Dakota where the structure design was changed to a box culvert because of the long piling required to support the structure.

The second example, in Codington County, used resistivity tests as preliminary information for design with final design estimates based on core drilling. The structure was built on spread type footings and is functioning satisfactorily.

The final example, on the Interstate Highway by Dell Rapids, was built on spread type footings after a plate bearing test indicated satisfactory bearing capacity of the Wisconsin till.

The use of geologic principals for analysis of soils structure and highway design will provide the most economic utilization of materials for construction as geology governs the formation of soils which provide the building material for highways.

INTRODUCTION

The Pleistocene epoch is represented in eastern South Dakota by cyclic glaciation. The action of the glacial ice and erosion during interglacial periods left a jumble of glacial geology over approximately 34,000 square miles. The study of Pleistocene geology of this territory represents a continuing study by the Department of Highways, for in this area live almost two-thirds of the total population of the state. For this reason a large portion of the total highway department construction and maintenance budget is spent in this area.

For the purposes of highway construction, the Department conducts a continuing search for gravel deposits, and performs soils studies and foundation investigations for structures. In the pursuit of these objectives, the need to understand the glacial geology of the area and to apply this knowledge to the solution of these problems becomes apparent.

SCOPE AND PURPOSE

No attempt will be made to discuss the separate problems of gravel exploration and soils studies for highway design and construction.

It is the intent of this paper to discuss the general geology of eastern South Dakota and attempt to relate it to the design and construction of three structures as located on the index map. Each structure is located in a different lithologic unit of the Wisconsinan stage of glaciation.

GENERAL PLEISTOCENE GEOLOGY OF EASTERN SOUTH DAKOTA

The Foundation Section does not actively engage in geological studies, except in instances where the geology and related features influence structure design. Therefore, the aspects of the regional geology must come from papers and reports prepared by others. These include the U. S. Geological Survey, South Dakota State Geologic Survey, and thesis work for advanced degrees by students. The data used in this discussion will be from these sources as well as pertinent data obtained from soundings made by the Foundation Section.

Topography and Drainage

A traveller from the west upon crossing the Missouri River is struck with the change in topography and vegetation. He has left a region of sharp gray shale hills, sparse grass, and void of trees and entered a rolling terrain of poorly defined drainage, heavy vegetation, and farmland. Speeding along, one notes the lack of relief which, in extreme, is approximately 1,000 feet. Average rainfall in this region is 22 inches per year, and the Chernozem soils of this area are subject to intense frost action in the winter months.

Two areas of relative high relief are indicated by the Missouri and Prairie Hills regions (Fig. 1). This is contrasted by the low lying James River Basin which formed the trough which provided a pathway for the glaciers to follow, and it was in this low trough that glacial Lake Dakota formed.

The present continental divide between Hudson Bay and the Gulf of Mexico is located in Roberts and Marshall Counties. The Missouri, James and Big Sioux Rivers provide the major drainage to the south and hence to the Gulf of Mexico. The Missouri River approximates the western reach of the glaciers where the drainage changes from north-south to east-west. Along the Missouri trench and immediately to the west of the Missouri trench, indications of the maximum extent of the glaciers during the Iowan substage of Wisconsinan glaciation can be found.

Pre-Wisconsinan Glacial Deposits

Glacial drift of eastern South Dakota is predominately a fine-grained sediment of silt and clay sized particles with aggregates of Niobrara chalk and Pierre shale. The pre-Wisconsinan tills are differentiated from younger tills by a scarcity of large rock fragments, a greater density, chemical alteration, distinctive coloring, and a possible greater thickness.

Boulders and cobbles of 6-inch size are rare and are not commonly seen. Flint (1955, p. 31) estimates that pebble and larger sizes constitute a small fraction, no more than 1 percent of the entire volume of the till.

On an outcrop of pre-Wisconsinan till, a blow with a pick leaves only a small dent while a similar blow in Wisconsin tills is likely to cause the material to shatter. This compaction leads to fracturing of a pre-Wisconsinan till around the included aggregate to leave a distinct mold.

Chemical alteration. - The nature of chemical alteration is to proceed from the surface downward and varies in intensity with depth. Therefore, it is reasonable to assume a greater depth of weathering on a low level, poorly drained area. Gumbotil, the residual product of this process, is rare in South Dakota in the pre-Wisconsinan tills. Gumbotil was never formed or was quite possibly stripped away by later glacial action or erosion.

Distinctive coloring. - The coloring of pre-Wisconsinan tills are of the darker hues. That is, in Wisconsin till the colors are buff, gray or yellow brown while the pre-Wisconsinan group tends to brown, chocolate brown with the older unoxidized portion of these tills being described as gray black or blue gray.

The difference between tills of different ages as revealed by ordinary geologic study methods is not great enough to permit correlation with any degree of confidence. In South Dakota there is no typical section that can be described and referred to as Nebraska or Illinois till. What constitutes pre-Wisconsinan tills are inferred by stratigraphic position or more accurately attributed to a certain age by "what they are not".

By stratigraphic inference Nebraska drift, Kansas and Illinois drift have been described. These have been described for this report in Table 1, "Classification Chart of Pleistocene Glacial Deposits in South Dakota."

Pre-Wisconsinan nonglacial deposits. - Study of Table 1, reveals that several formations recognized in adjoining states have been left out by this author, because several of these formations have only been tentatively identified in South Dakota and are indirectly correlated with their counterparts in Nebraska, Kansas, and Iowa.

These deposits consist of outwash sands and gravels, loess, and volcanic ash. Again, like pre-Wisconsinan tills these deposits are exposed in only a few places. Sands and gravels attributed to the pre-Wisconsinan age are located along the western edge of the glaciated region and in stream channels. Only the Loveland loess and the Sapa formation which are recognized in Nebraska and Iowa, are identified with any certainty. The Loveland loess is recognized by grain size and color which is a "reddish pink" or "pink", however, care must be exercised to keep from confusing it with loess of Wisconsinan age.

Along Skunk Creek in Minnehaha County, outcrops of a volcanic ash are found and correlated to the Pearlette ash in Kansas. The ash is unaltered,

TABLE I, CLASSIFICATION CHART OF PLEISTOCENE GLACIAL DEPOSITS IN SOUTH DAKOTA

Time Units			Lithologic Units	Dominant Event	Thick-ness *	Description of Material
Recent Epoch			Soil	Erosion	3+ 4+	Alluvium, Clay Silt & gravel Lake deposits clay and silt
Pleistocene Epoch	Wisconsin Age	Mankato Glacial Substage	Loess Drift	Deposition	+0 30	Brown, wind deposited Silt Brown, Silty Clay
		Interglacial Period	Outwash deposits forming plains, terraces & valley trains	Erosion	4+0 85	Sand and Gravel with some Clay and Silt.
		Cary Glacial Substage	Loess Drift	Deposition	20 to 45	Wind deposited Silt Till, Boulder Clay Gray brown Silty Clay
		Interglacial Period	Erosional Unconformity Outwash deposits	Erosion	5 to 20	Sand and gravel in terraces some Clay and Silt on top
		Tazewell Glacial Substage	Loess Drift	Deposition	15 to 50	Wind deposited Silt Till, Boulder Clay Brownish Gray Silty Clay
		Interglacial Period	Loess	Deposition	1 to 6	Brown or buff Wind blown Silt
		Iowan Glacial Substage	Drift	Deposition	20 to 60	Till pebble Clay, gray to brown-gray, compact, Silty Clay
	Sangamon Interglacial Age		Erosional Unconformity Weathering Profile oxidized Sediments and Gumbotil	Erosion	No thickness agreed on by S.D.S.G.S. and U.S.G.S.	Outwash Sands and Gravels
	Illinoian Glacial Age		Loveland Loess Drift Erosional Unconformity	Deposition		Pink, Wind deposited Silt Brown, Silty Clay
	Yarmouth Interglacial Age		Weathering Profile Erosional Unconformity	Erosion		Fossiliferous Clays Locally loess
	Kansan Glacial		Pearlette Ash Drift	Deposition		Volcanic Ash Brown, dense Silty Clay
	Aftonian Interglacial Age		Weathering Profile	Deposition		Outwash deposits of Sand and Gravel
	Nebraskan Glacial Age		Drift	Deposition		Blue Gray, Silty Clay dense
Pliocene Epoch			Ogallala Formation ?	Deposition		Western Sands and Gravels

From: Flint 1955, Tipton 1958, Lee and Powell 1961
* Approximate thickness in feet.

noncalcareous, and white or light gray in color.

Wisconsinan Glaciation

The most extensive period of glaciation in South Dakota is the Wisconsinan age. During this time the ice covered about 34,000 square miles. The retreats, advances, and complete removal of the ice is recorded in moraine and deposits of outwash material.

Iowan substage. - The Wisconsinan age of glaciation is composed of four recognizable glacial substages with relatively unimportant interglacial periods. The first and most extensive glacial substage was the Iowan. Evidence that this glacier extended beyond the confines of the Missouri River can be found in Stanley, Corson, and Dewey Counties.

At the close of the Sangamonian interglacial age, the Iowa ice entered northeastern South Dakota and spread south and west. The southward movement was facilitated by the James River lowland. The ice spread rapidly, filling the Missouri River trench throughout the state.

The dissected Missouri Hills region collected the bulk of the till. This till is a gray to gray brown compact silty clay with Niobrara chalk and shale fragments as inclusions. As the glacier wasted, the surface was deflated leaving a covering of pebbles. This exposed surface was abraded and polished by the winds and buried by the accumulating loess. This loess marks the interglacial period and no appreciable weathering profile was formed before the advance of the Tazewell ice.

Tazewellan substage. - The new invasion from the north formed well defined lobes known as the DesMoines and James lobes. As with the Iowa glacier, Tazewell ice followed the James Basin, however, the ice failed to cover the Prairie Hills. Apparently, this ice thrust was not as thick as the Iowa glacier.

Climatic influences evidently slowed the Tazewell advance, but evidently the margin stood long enough at the maximum extent to build an end moraine. The Big Sioux River functioned as an interlobe drainage and began to build the extensive gravel and sand valley train characterizing it today.

The till deposited from the Tazewell ice, although containing boulders of granite and gneiss, consists of a brown silty clay containing Niobrara chalk and shale aggregates. Again, as the glacier wasted, deflation occurred and loess was deposited.

Tazewell-Cary interstage. - The interval between the Tazewell ice sheet and Cary invasion was of longer duration than the Iowan-Tazewellan interglacial period. The loess deposits of this period are thicker and more extensive with the development of a Chernozem soil by oxidizing the loess. The development of a weathering profile would indicate that the ground thawed, plants were present, and that climatic conditions were such that deposition of loess was halted. In adjoining states this interval supported a diverse fauna, however, in South Dakota, no good evidence other than wood fragments and a small mouse (Flint, 1955, p. 163) extracted from possible Tazewell loess in Hughes County have been identified. At any rate, the conditions soon favored a third ice advance.

Cary substage. - For the third time, ice plowed up the previous loess deposits and cut into the tills and underlying rock. The Cary glacier followed the James Basin and covered part of the Prairie Hills and spread out over the Missouri Hills. The thin Cary ice formed a well defined lobe in the James Basin with the Sioux River acting as the drainage between it and the DesMoines lobe.

Cary ice must have reached the Missouri River from Chamberlain to the Nebraska border, however, no major diversion of the Missouri River occurred as is indicated by the missing end moraine associated with the Cary glacier.

Apparently the maximum extent of the Cary ice persisted for a long period, because many small retreats and advances may be mapped. During the retreats, loess was deposited and finally as the ice retreated a thin mantle of loess was deposited across the Cary till.

Cary till is similar to Iowa and Tazewell tills except that the Cary till has more boulders than previous ice sheets. These boulders are used as an identifying marker. The Cary-Mankato interval apparently was short-lived as no widespread weathering profile developed prior to the invasion of Mankato ice.

Mankato substage. - The final surge of ice of the Wisconsinan age formed a sharp lobe down the James Basin. The Prairie Hills, again, formed a wedge between the James lobe and the Des Moines lobe. The Mankato ice carried down the James River to the Missouri River between Yankton and Springfield. The lateral thrust towards the west was weak and short-lived although the ice remained in James lowlands for a relatively long period.

The tills of the Mankato substage resemble closely those of previous glaciers. Brown, silty clays with Niobrara chalk and Pierre shale as aggregates constitute the bulk of the till.

As the glacier retreated, a moraine blocked the James River at or about the northern border of Beadle County. The result as a glacial lake which at maximum extent covered 1,800 square miles in North and South Dakota. Today Glacial Lake Dakota is marked by near shore sand deposits and wave cut terraces covering Spink, Brown, and Marshall Counties.

DISCUSSION OF EXAMPLES

The three examples considered here are located in the glaciated area of South Dakota as shown on the index map (Fig. 1). Each example is in a different glacial substage. For example, the first site in Marshall County is located on Mankato till overlying old Glacial Lake Dakota sediments on what was the east shore of the lake.

Site One, Project S6141, Marshall County

The proposed structure is located on State Highway 25, 9 miles north of Britton. The original structure, a 20-foot roadway by 14-foot reinforced concrete bridge was to be replaced by a proposed 64-foot reinforced three-span structure with a 24-foot roadway (Fig. 3).

Field Work. - Data was collected using a 4-1/2 inch auger to make borings for a preliminary soils classification. A penetration test and cores were taken by using a 470-pound hammer dropped 30 inches to drive a 2-7/8 inch O.D. retractable plug sampler.

Geology. - The auger borings and cores obtained from the penetration test indicated a 10-to 18-foot thickness of brown to gray silty clay, identified as Mankato till. Immediately underlying this till is 40 to 50 feet of gray silt clays with lenses of sand representing sediment of old Glacial Lake Dakota.

The soft unconsolidated nature of this material as shown by the penetration test and the presence of varying in the cores indicate an old lake bed. None of the material at the footing zone was capable of supporting a 3.25 T.S.F. load and with the penetration test going to elevation 1,226, piling 55 feet long would be required to support the structure (Fig. 3).

Recommendations. - Since long piling would be required, it was recommended that the small bridge be dropped and some other structure be used. The final design called for a triple 10 by 4-foot reinforced concrete box culvert which would require 500 psf bearing capacity from the soil to support the structure. The box is in place and performing satisfactorily today.

Site Two, Project S4990(1), Codington County

Site Two is a county road project located 11 miles southeast of Watertown on Stray Horse Creek (Fig. 4). The old 82.7 by 17-foot, single span, pony truss bridge was to be replaced with a 184.75 by 30-foot, three span, continuous composite, girder structure. Old records indicated that the pony truss structure was on spread type footings, therefore, it would be necessary to determine if this was correct.

At the time the request for the study came to the Foundation Section, the drilling equipment was tied up on another project, therefore, as preliminary data, two resistivity tests were conducted. Later a complete drilling program was undertaken.

Resistivity surveys. - Figures 5 and 6 are the results of the resistivity survey conducted at the site. The tests were performed with a milliameter-potentiometer type direct current device. The interpretation is by R. W. Moore's (Bureau of Public Roads, Physical Research Branch) method of cumulative curve plotting. This method of interpretation uses the intersection of straight lines and the points of inflection on the field curve to indicate a change in material.

Test No. 4, indicated 8 feet of sandy clay, 8 feet of sand and gravel, and 6.5 feet of heavy gravel or boulders. Below 32 feet, the material was homogeneous and probably glacial till. Test No. 3, had 6 feet of sandy clay, 8 feet of sand and gravel, and 16 feet of gravel below which is glacial till. These data indicate the possible use of spread footings in the sand and gravel zone between 6 and 16 feet. Based on old records and these data the structure design was set to have spread footings at elevation 482.

Geology. - The data from core tests is presented here in Figure 4. These tests indicated 4 to 9 feet of road fill and undifferentiated recent alluvium overlaying 6 to 9 feet of a sandy gravel glacial outwash. At 18 feet, there is brown glacial till with alternating layers of sand and clay with a dark gray till at 32 feet. Unconfined compressive tests gave in excess of 3.25 tons per square foot. The alluvial silts and glacial outwash sands and gravels are lying on Tazewell and younger till with the underlying dark gray till representing pre-Wisconsinan till.

Construction. - When the footings were opened, the north bent was seated at 20 feet or elevation 480 in a brown clay. The south bent on the west side was seated in brown glacial clay at elevation 482. However, when the east pad of the south bent was opened a concrete deck of an old structure was encountered. Beneath this was old piling, timber, and rubble dating back to 1911. After cleaning out the excavation, the footing was set 2 feet below the west footing in a gravel layer.

Site Three, Project I-29-3(11), Minnehaha County

Example three is located 3 miles southwest of Dell Rapids. The structure was proposed as a local road over the Interstate Highway. The structure to be used is a 254-foot, continuous type, composite girder viaduct.

Geology. - Borings at the site indicated a brown plastic silt clay which represents Wisconsin till probably Iowan. Below elevation 550 is a stiff brown glacial clay which is probably Illinois till. While at elevation 535 on the west and dipping east, the borings showed Sioux quartzite (Precambrian).

Construction. - The footing elevations were set at 544. Unconfined compressive tests had a maximum value of 21,258 psf which indicated the use of spread type footings, however, owing to the erratic nature of glacial deposits, 15-foot, 8BP36 piling were included in the plans. When the footings were opened, all were set in stiff brown till. Figure 8 has the results of a plate bearing test conducted at the site, used to confirm laboratory tests.

Two loaded trucks were used as the reaction load. A 50 T. capacity jack supplied the load and the settlement was measured with an Aimes dial guage. The data was plotted in a stress-strain relation. Allowing 1/4 inch displacement a safe bearing capacity of the soil was determined as 13,200 psf. The bearing required to support the structure is 6,500 psf. The piling was eliminated and the structure built using spread footings.

CONCLUSIONS

The study of glacial geology in South Dakota is in its infancy. Current literature deals with only a small part of the total area covered by the glaciers. Each structure site is a problem to be considered individually. The wide variation of glacial soils in a short horizontal range demands that foundation investigations be thorough. To be thorough, the drill data must have geological principals applied for proper interpretation.

A structure is only as good as the foundation on which it rests. The design and construction of it depends on accurate and complete field tests which must be interpreted and used wisely.

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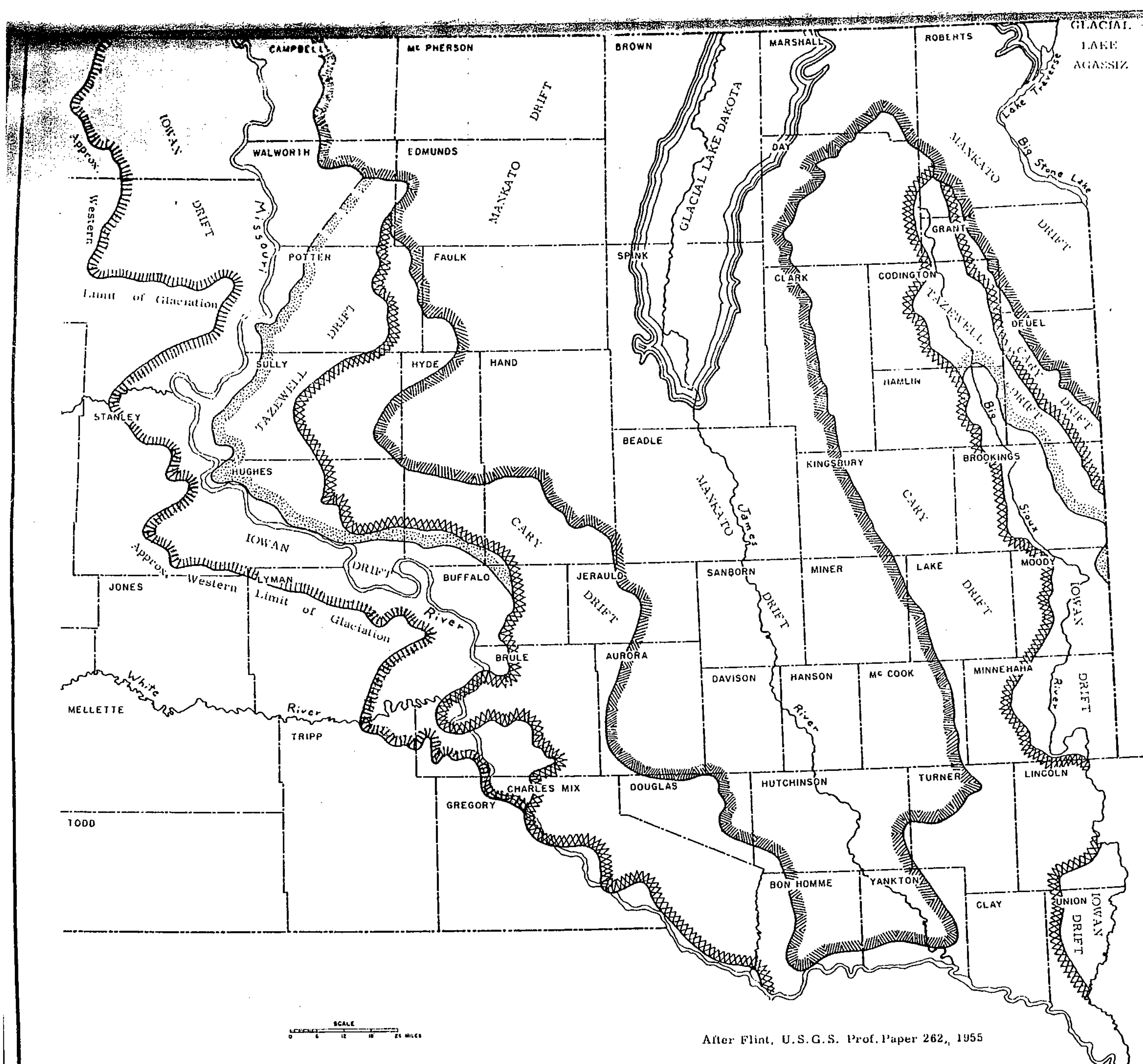
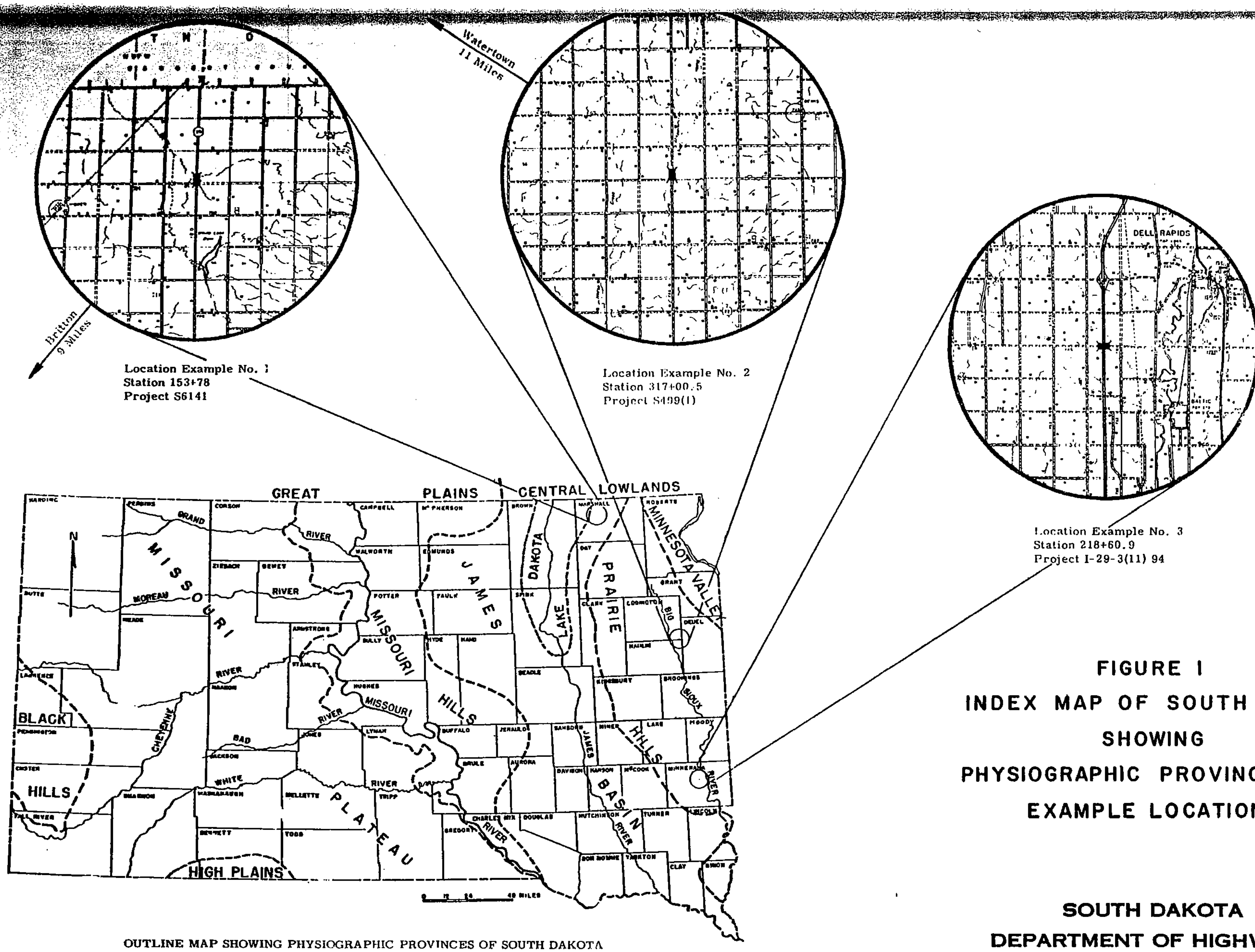


FIGURE 2

SKETCH MAP SHOWING GLACIATED REGIONS OF EASTERN SOUTH DAKOTA

SOUTH DAKOTA DEPARTMENT OF HIGHWAYS

DESIGNED BY	DRAWN BY	CHECKED BY	APPROVED
	G.L.		



OUTLINE MAP SHOWING PHYSIOGRAPHIC PROVINCES OF SOUTH DAKOTA

after Rothrock, 1943

FIGURE I

INDEX MAP OF SOUTH DAKOTA SHOWING PHYSIOGRAPHIC PROVINCES AND EXAMPLE LOCATIONS

SOUTH DAKOTA
DEPARTMENT OF HIGHWAYS

DESIGNED BY	DRAWN BY	CHECKED BY	APPROVED
	G.L.		

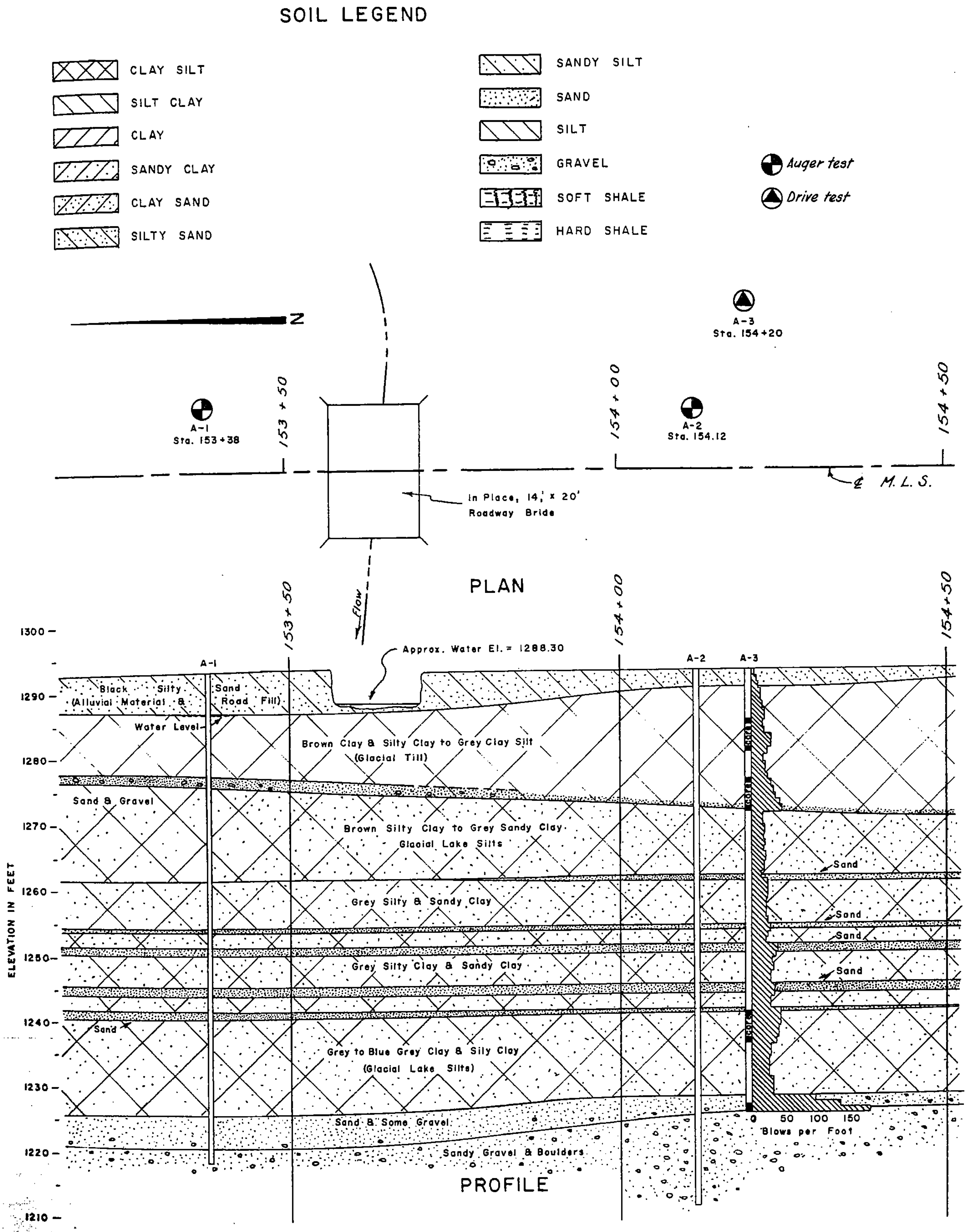


FIGURE 3

SITE PLAN & SUBSURFACE PROFILE

Bridge in Sec. 13 & 14, T. 128N, R. 57W Station 153+78
Location A Marshall County Project No. S6141
Scale: Vert: 1" = 10' Hor: 1" = 20' Drilled: Sep. 2 & 10, 1959

South Dakota Department of Highways

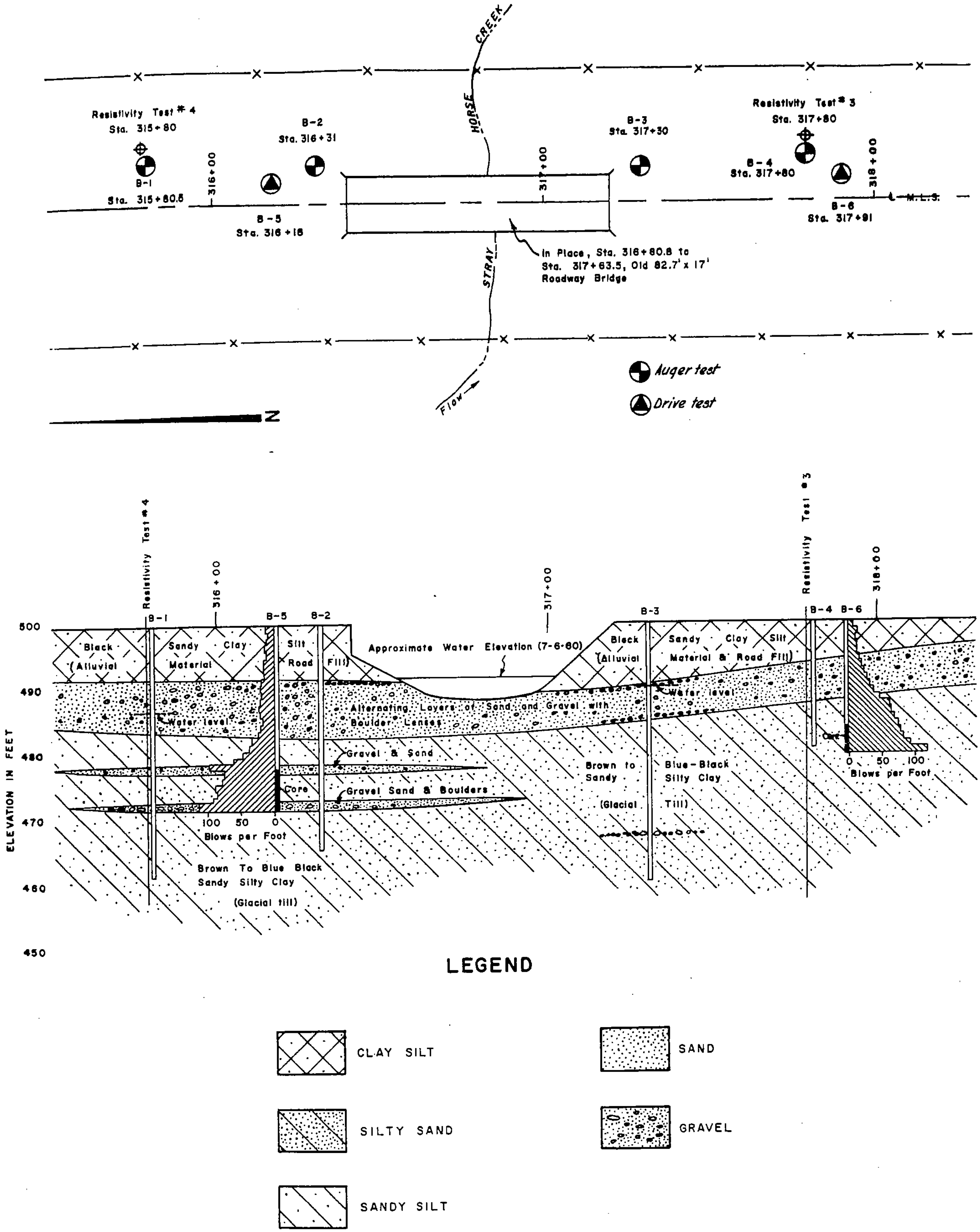


FIGURE 4

SITE PLAN & SUBSURFACE PROFILE

Bridge in Sec. 34 & 35, T. 116N, R. 51W Station 317+00
Location B Codington County Project No. S4990(1)
Scale: Vert: 1" = 10' Hor: 1" = 20' Drilled: July 6&8, 1960

South Dakota Department of Highways

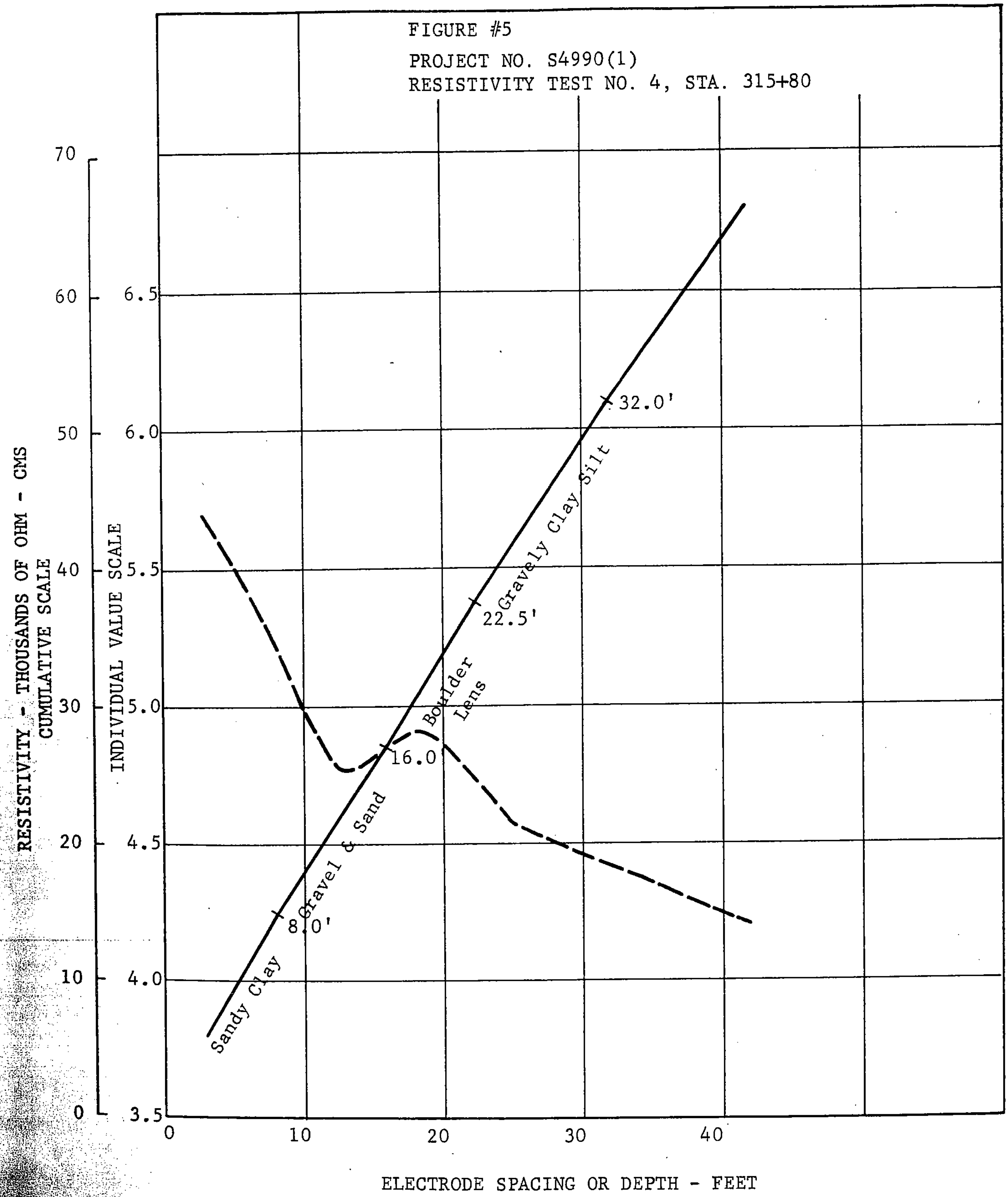


FIGURE #5
PROJECT NO. S4990(1)
RESISTIVITY TEST NO. 4, STA. 315+80
RESISTIVITY - THOUSANDS OF OHM - CMS
CUMULATIVE SCALE
INDIVIDUAL VALUE SCALE
ELECTRODE SPACING OR DEPTH - FEET
Sandy Clay
8.0'
Gravel & Sand
16.0'
Boulder
Lens
22.5'
Gravely Clay Silt
32.0'
0
10
20
30
40
50
60
70
3.5
4.0
4.5
5.0
5.5
6.0
6.5

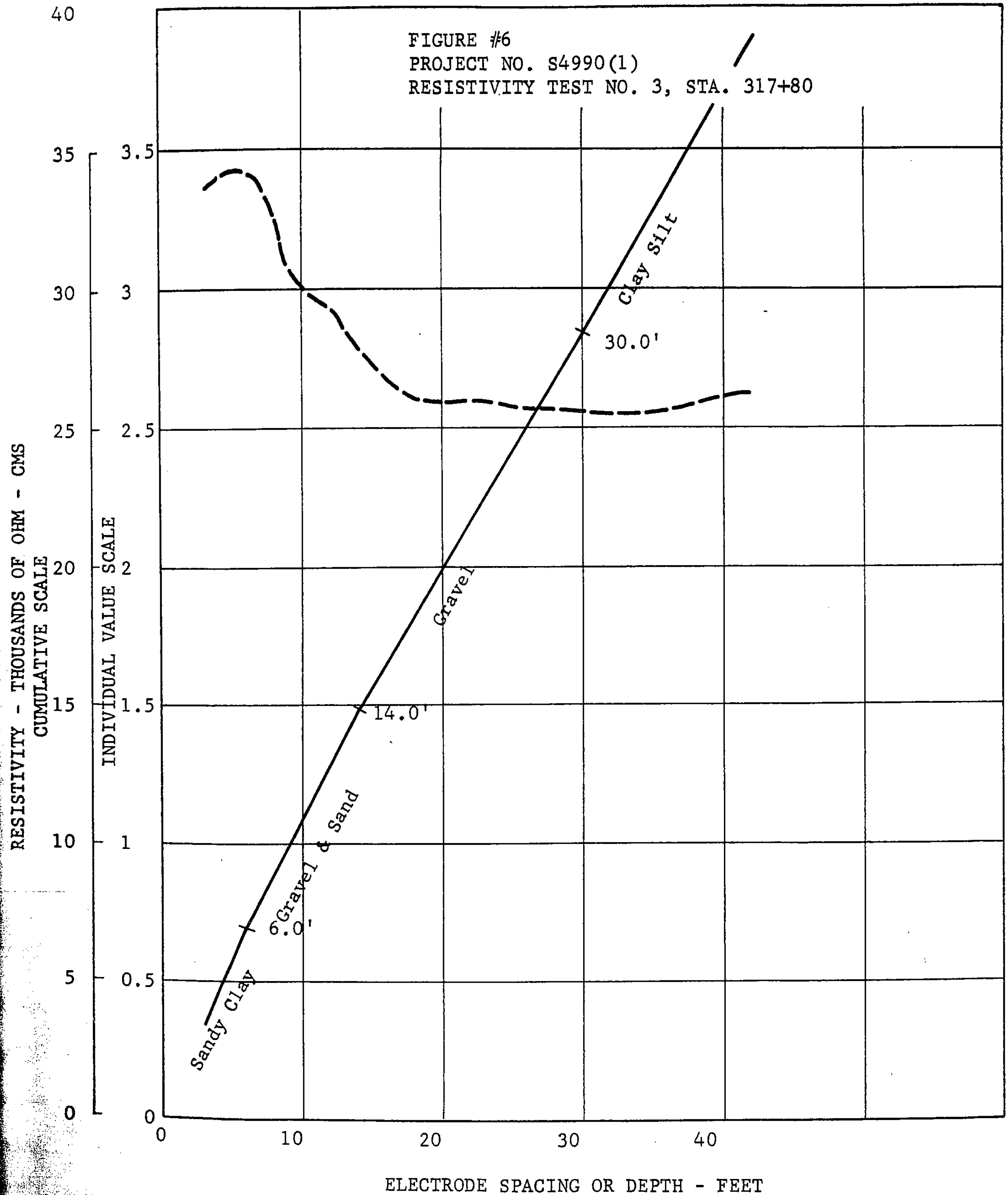


FIGURE #6
PROJECT NO. S4990(1)
RESISTIVITY TEST NO. 3, STA. 317+80
RESISTIVITY - THOUSANDS OF OHM - CMS
CUMULATIVE SCALE
INDIVIDUAL VALUE SCALE
ELECTRODE SPACING OR DEPTH - FEET
0
5
10
15
20
25
30
35
40
0.5
1
1.5
2
2.5
3
3.5
0
10
20
30
40
Sandy Clay
6.0'
Gravel & Sand
14.0'
Gravel
30.0'
Clay Silt

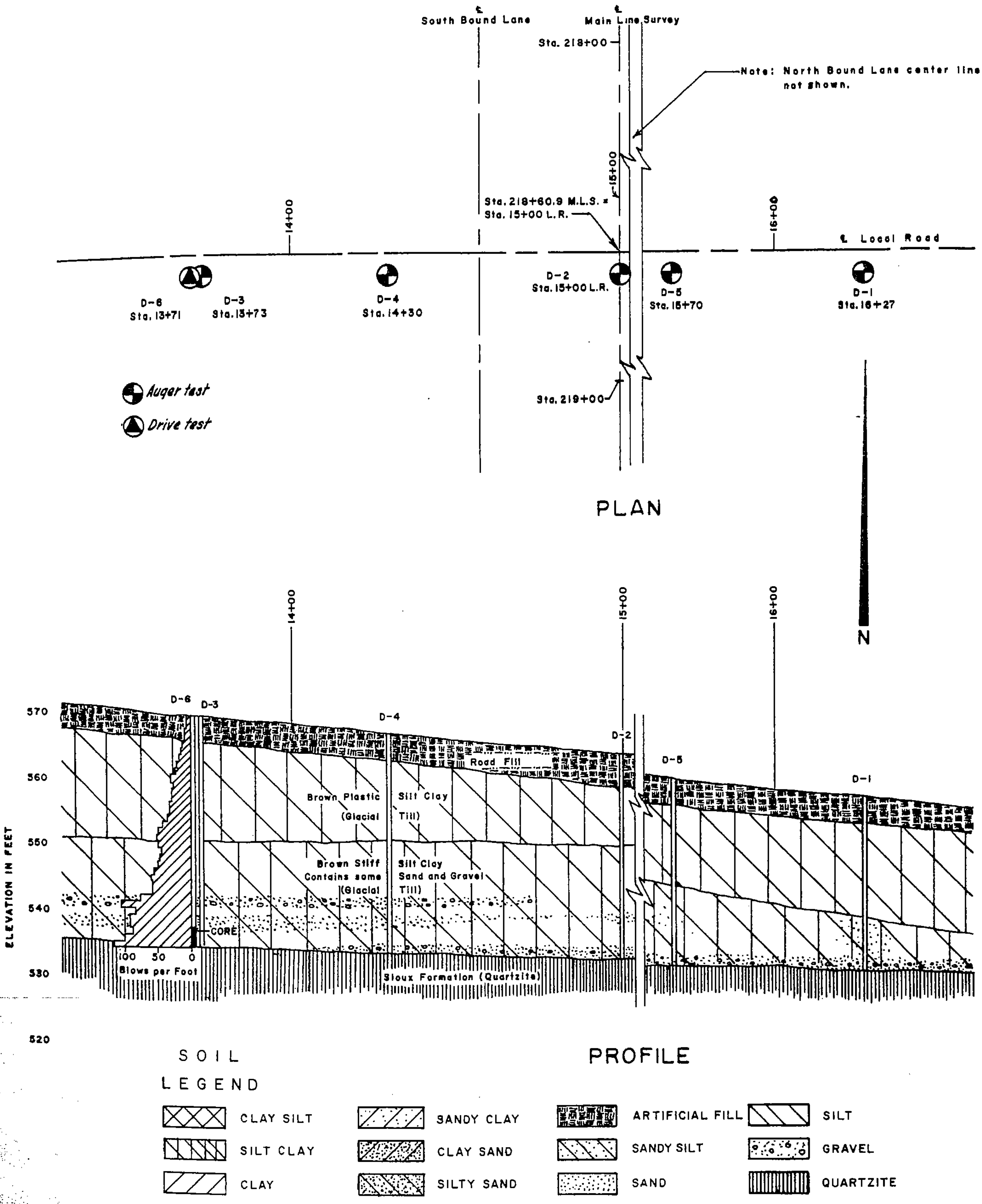


Figure 7

SITE PLAN & SUBSURFACE PROFILE

Viaduct in Sec. 24 & 25, T. 104N, R. 50W
Sta. 218+60.9 M.L.S. Sta. 15+00 Local Road Minnehaha County
Project No. 1-29-2(11)94 Location D
Scale: Vert: 1" = 10' Hor: 1" = 20' Drilled: Aug. 17-18, 1959

South Dakota Department of Highways

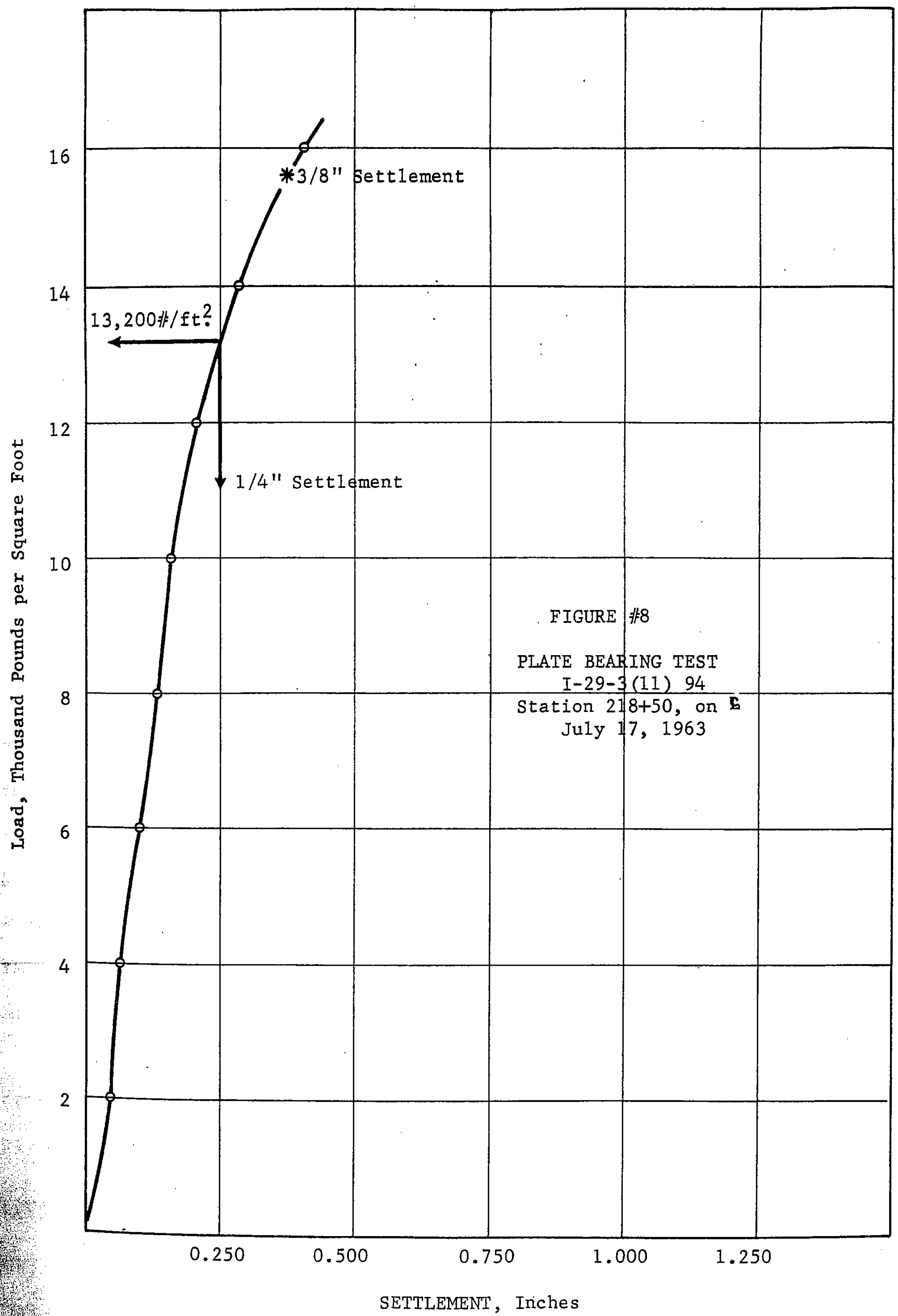


FIGURE #8

PLATE BEARING TEST
I-29-3(11) 94
Station 218+50, on ℄
July 17, 1963

ENGINEERING AND CONSTRUCTION PROBLEMS IN THE VALDEZ DISTRICT, ALASKA

by

Ralph R. Migliaccio*

INTRODUCTION

The Valdez district is one of five highway districts into which the State of Alaska has been divided (Fig. 1). It is located in south-central Alaska and encompasses an area of some 50,000 square miles. In fact, the district is larger than 24 of the 50 states. Total mileage of existing highway, both primary and secondary, is approximately 900 miles. Geographically, the district may be divided into two general categories. These are the coastal, or southern portions, and the interior, or northern portions. The interior regions are separated from the coast by the Chugach Mountains, which parallel the southern boundary of the district. At present, only one highway, the Richardson, traverses the range. Another highway is now under construction, and portions of this highway will be discussed later in the paper. The Chugach Mountains are a labrinyth of steep dead-end canyons, sharp peaks, innumerable glaciers, and swift glacier-fed streams. Figures 2, 3, and 4 are glaciers common in the range. Almost without exception, these mountains drop abruptly into the Gulf of Alaska leaving little room for any sort of development. Valdez is situated on a glacier outwash plain adjacent to the Gulf of Alaska (Fig. 5). The ruggedness of this terrain is one of the principal reasons why coastal highways do not exist within the Valdez district.

Interior regions of the district may be further divided into two provinces (Fig. 6). These are the Copper River Basin and Tanana River valley. The Copper River Basin lies immediately north of the Chugach Mountains. It is bounded on

*All figures by R. Migliaccio unless otherwise indicated.

the east by the Wrangell Mountains, on the west by spurs of the Chugach Mountains, and on the north by the Alaska Range. The entire basin, which encompasses an area of some 25,000 square miles, is drained by the Copper River which flows southward into the Gulf of Alaska. Figure 7 is an aerial view of the Copper River Basin. The Copper River is in the foreground, and the Alaska Range is in the background. The Tanana Valley is a broad intermontane valley within the Alaska Range. It varies between 50 and 100 miles in width and trends northwest. This area is drained by the Tanana River which flows northwest into the Yukon River.

Climate within the Valdez district is varied. The coastal regions receive precipitation ranging between 70 and 90 inches per year, while the interior portions of the district are semiarid with less than 13 inches per year. Average summer temperatures are 50 degrees Fahrenheit on the coast and zero degrees in the interior. These averages are somewhat deceptive especially for the interior, as large fluctuations do occur. As an example, temperatures occasionally reach 100 degrees in Fairbanks, and during winter a minus 60 degrees. Temperatures may also rise from minus 60 degrees to plus 30 degrees in 24 hours.

Interior regions make up by far the most accessible portions of the district. It is in these regions that our most complex highway engineering problems usually occur. The remainder of this paper is devoted to a discussion of the problems which are common within the Valdez district. They are not, with one exception, unique to the Valdez district.

PERMAFROST

The subject of permanently frozen ground is discussed first since it is one topic about which most engineers and geologists have at least some knowledge. This is not to say that anyone knows exactly how permafrost was formed, how to build lasting structures on permafrost, or for that matter, how the term is best defined. For purposes of this paper, all soil or rock which remains frozen throughout any one year and from year to year, is considered permanently frozen and is referred to as permafrost.

One of the major problems one faces when dealing with permafrost is determining its location. There are numerous methods in use, the most common being drilling. There are other methods, such as air photo analysis, observations of vegetative types or patterns, and geophysical methods. All require on-site subsurface investigations for confirmation. In the Valdez district, the current technique involves the use of solid and hollow stem continuous flight augers. These are powered by a Mobile B-38 mounted on a track vehicle. Figure 8 shows a 6-inch, solid auger upon retraction from the permafrost zone. The large block on the auger head is frozen silt which must be removed with a pick before the auger will penetrate further. Wash borings are sometimes desirable, but locating the frost level requires continuous sampling. Temperature recording instruments are occasionally installed to either confirm or show conflict with original conclusions. It is sometimes impossible to determine whether the frost encountered is permafrost, or relict annual frost left over from the preceding winter. This is especially true if subsurface temperatures are consistent with depth, the condition exists over a large area, and frost extends downward for great depths with no break. There is seldom time for a complete analysis since construction schedules must be met. During winter months, locating the permafrost level is a virtual impossibility since frost extends from ground surface to untold depths. Unfortunately, it is seldom possible to avoid winter field operations, and a return to the site is then mandatory for permafrost deliniation.

After the frost has been located it must be decided whether or not the condition will be detrimental to the proposed construction. This depends on numerous factors. Generally speaking, clean coarse materials, although frozen, are not considered detrimental. These materials are generally well drained and have low natural moisture contents. In addition, their high permeability and low capillarity prevent, to a large extent, the formation of ice lenses. Fine grained materials are the exact opposite. These materials, with their low permeability and high capillarity, retain large quantities of moisture and provide an excellent environment for ice lenses and ice crystal formation. The size of ice lenses which have been encountered in the Valdez district vary from fractions of an inch to as much as 20 feet in thickness. Ice lenses are by no means uncommon and can usually be anticipated in poorly drained areas. The thickest ice lense encountered in this district was located directly below centerline of the Alaska Highway, approximately one mile west of the Canadian border. The section is to be reconstructed and a cut had been proposed at this point; hence, borings had been extended to greater depths. If a cut had not been proposed, it is quite likely the lense would never have been discovered. The ice body was lense-shaped and some 75 feet long. It was located directly below centerline of the existing roadway, with no surface indication of its presence. To make matters worse, the ice was surrounded by frozen organic silts with moisture contents up to 1200 percent. Other than the thickness of said ice lense, this is not an unusual problem in the Valdez district. Natural moisture contents over the 100 percent mark are encountered on nearly every project, and in some cases they are the rule rather than the exception. This condition is due to large amounts of free ice in the permanently frozen soils. In some instances, the ice particles make up the bulk of a particular sample, and visual classifications such as "Silty Ice" are common.

It is generally agreed that design and construction must be based on what are termed "passive" methods. That is, it is seldom possible or practical to remove all the permanently frozen soils, and experience has shown that retention of the permafrost is desirable if it can be accomplished economically. When working in areas underlain by permafrost, every effort is expended to see that permafrost will be retained with as little disturbance as possible, or that whatever disturbance occurs is uniform. Using this line of thought, deep cuts as well as high fills are avoided. This is not always possible due to other factors. In the case of deep cuts in frozen fine grained soils, backslope problems can always be anticipated during the design stage. Anticipating and solving such a problem are two very different things. First, it is seldom possible to obtain undisturbed samples, and if they are obtained, it is a vertual impossibility to transport such samples to the laboratory without further disturbance. Normally, such samples can be poured from the sample tube on arrival at the laboratory. This is a good indication of what will occur when the cut is opened. Backslopes, which were smooth and clean when cut, degenerate while thawing and flow downslope in a semiliquid mass. This same condition exists in the cut base and a mudhole develops in which equipment operation is nearly impossible. These conditions must also be tempered by considering many other factors. For instance, a cut running east-west will have one backslope facing south and one facing north. The south facing slope will receive more sunlight and may be expected to thaw deeper and more rapidly than the north facing slope. It should be apparent that a solution must depend on judgement and experience to a great extent. It should be pointed out that in permanently frozen cuts, backslopes are only half the problem. Once cutting is complete, subsurface thawing will move downward to a depth which is for the most part, unpredictable. Placing thick base courses and/or embankments over the cut base may inhibit the rate of thaw, but in most cases will not halt its downward movement. In some instances, it may increase both the rate and depth. Various means of insulating the frozen subsoils are now being utilized in European countries, but they have not been attempted by the Alaska Department of Highways. It has been our experience that cuts in permanently frozen fine grained soils usually result in a nonuniform riding surface.

In areas where high fills are proposed other problems arise. Of primary importance is the question of whether or not the subsoils will support the proposed fill without danger of embankment failure. In permafrost areas this is a question which cannot usually be answered by what are termed "conventional" methods. If the subsurface thermal regime is left undisturbed, the frozen soils provide a stable, strong foundation. This is something that is difficult, if not altogether impossible in highway construction. Whenever an embankment is placed, the thermal conditions are altered in nearly every case. Whether a deep or shallow thaw will occur beneath the constructed embankment is something which cannot be accurately predicted. This is especially critical when subsoils consist of silts, organics, or any mixture thereof. Under normal conditions, these soils could possibly be removed or surcharged in such a manner to eliminate long term detrimental settlements. This is definitely not the case in permafrost areas where the soils may be frozen to depths of 100 feet or more. The removal of the upper layers does nothing more than expose the lower layers and promote a deep thaw. Usually, every effort is made to obtain undisturbed samples from the proposed fill areas. These samples are then thawed, and if possible they are subjected to triaxial compression and consolidation tests. Analyses are then made under what are assumed to be the most critical conditions likely to occur beneath the finished embankment. It is, as previously mentioned, very difficult to obtain undisturbed samples. If such samples are obtained, thawed and tested, there is no assurance subsoils beneath the proposed embankment will behave in a like manner. Indeed, they may never even thaw. This is especially critical in settlement predictions. One can never be sure the subsoils will thaw and consolidate according to the theory. It is seldom possible to perdict the thickness of soils which can be referred to as "compressible" since they are frozen. There are various formulae which can be used to predict the depth of thaw induced by various heights of fill, but these are dependent on so many variables that they are used as guide lines only.

For the reasons mentioned, every effort is made to assure retention of the frost table at as high a level as possible. This entails clearing the fill area by hand, keeping tracked or heavy equipment off the area, and end-dumping the initial lifts. If possible, the initial lifts are made up of clean sand which has several purposes. First, it will prevent puncturing of the organics overlying the frozen subsoils, second, it will provide a moisture cutoff for capillary water, and third, it will prevent intrusion and contamination of the embankment by fine grained soils. The Norwegian Railroad is now utilizing compressed peat, leaves, tree bark, or other organics as insulators over permafrost. This has not been tried as yet, on highways in Alaska.

SURFACE ICING

Another problem which is common in permafrost areas is "surface icing". This phenomenon has been defined as "a mass of surface ice formed during the winter by successive freezing of sheets of water that may seep from the ground, from a river, or from a spring". It is especially critical since it usually develops in ditches, culverts, or beneath highway bridges. When occuring in ditches or culverts, icing commonly builds up until it begins to cover the highway. This creates a condition which is hazardous and which causes embankment failure during the spring thaw. Figure 9 shows a typical icing condition. Note the chimney which marks the culvert inlet. The photo was made in January when the ice was just breaking onto the pavement. Ice at this site was 8 feet thick.

When forming beneath highway bridges, icings have been known to lift the deck off the supports. It can easily be seen why this is a critical problem. All that is needed is a steady seep or flow of water coupled with below zero temperatures. Under ideal conditions, masses of ice miles long and depending on site conditions, many feet in width, can be formed. Icings will spread laterally until natural restrictions are encountered.

Adjacent to roadways and on sloping ground, this condition is apparently connected with ground water seeping through the subsoils above the permafrost table, and below ground surface (Fig. 10). In the fall, frost begins to penetrate the ground, and because the roadway and right-of-way have been stripped of insulation an impermeable mass of frozen soil is formed that locks the subsurface flow. Water is then forced to the surface and icing begins. From then on it is only a matter of time before the ice builds onto the roadway. Figure 11 shows icing on a cut which is dry during summer. The entire phenomenon is dependent on average temperatures, snow cover, and amount of moisture available. This has been exceptionally obvious during the present winter.

The late fall months were cold with vertually no snow, and icing conditions developed in areas where they had never been noted in the last 10 to 20 years. Yet, during the present winter, seeping water froze into successive layers of ice totaling 3 to 4 feet thick on the Richardson Highway. Figure 12 shows an icing adjacent to the roadway.

Current methods of control are ice fences, culvert heaters, steam points, and diversion dikes. Ice fences made of cardboard or tarpaper are erected on the edge of the roadway and are meant to contain the ice. They are partially effective but are commonly damaged by snow removal equipment or traffic, they are topped, or they leak. Culvert heaters keep culvert inlets open in the hope that water will continue flowing rather than freeze and build up. Figure 13 shows a culvert heater buried in ice which is approximately 10 feet thick. The drums are the 55 gallon type. These units require constant refueling which is time consuming and expensive. Steam points are utilized in conjunction with culvert heaters and in instances where ponded water requires an immediate outlet. They are sometimes effective in providing a tunnel through which water can flow under an ice body. Diversion dikes are by far the most effective means of control, provided sufficient storage space is available. A bulldozer is used to construct a berm or dike which induces ponding and icing in a predetermined area. It should be noted that all of these methods are control rather than prevention. Current practice in the Valdez district is to design for <u>prevention</u> rather than control. First the trouble spots must be located. Of primary importance is locating the moisture source. Following location, suitable means of diverting or intercepting said moisture must be devised. This includes stripping a zone outside the cleared right-of-way of all insulating cover (Fig. 14). This zone, if kept free of snow, will allow a rapid freeze of subsoils and the formation of a relatively impermeable barrier. Water is then forced to the surface where it is allowed to pond and freeze in a predetermined area. This method is known to be effective in areas of slow, steady seepage, but is not intended for use in well defined channels which carry appreciable volumes. Under ideal conditions it can be used effectively above cut sections. Other methods being utilized are channel improvement and steepening culvert grades.

Icing is a problem we have yet to solve. Indeed, none of the methods of control or prevention mentioned herein are 100 percent effective.

COPPER RIVER HIGHWAY

As mentioned in the opening paragraphs of this paper, a second highway from the coast to the interior is now under construction (Fig. 15). This route, now known as the Copper River Highway, will connect the Port of Cordova with Chitina, Alaska. The roadway will be some 130 miles long, and will follow the Copper River through the Chugach Mountains.

During the early 1900's this route was used by the Copper River and Northwestern Railroad as a means of transporting copper ore to the Port of Cordova. Alignment of the highway is nearly identical to that of the railroad, and in several instances existing railroad bridges are being utilized.

Of primary importance, for purposes of this paper, is the section lying between 57 and 61 miles north of Cordova. In this zone, the proposed alignment will cross <u>over</u> the outer margins of a large stagnant ice mass or moraine. This, as far as we know, is the first attempt to build a highway over a glacier under climatic conditions such as exist in the Copper River

Canyon. There is a vast difference between this area and one such as Greenland (Fig. 16). As can easily be seen, a change in alignment is a virtual impossibility, with the Copper River on one side, and the Allen Glacier on the other. We are forced into crossing a mass of glacial ice which is melting.

Recent studies showed that some 3 miles of roadway will be underlain by glacial ice. The ice varies between 10 and 150 feet in thickness with the thicker deposits being some distance left of centerline. The ice is buried by 10 to 20 feet of overburden in the zone of the proposed roadway. Said overburden thins towards the center of the mass and in some areas is not more than one foot thick. This condition exists over an area of some 4 square miles. The ice cannot be detected from the proposed alignment due to heavy vegetation.

During summer months, this area resembles a rain-forest and is almost impenetrable. Figure 17 is an aerial view showing one of the many craters which are forming in the mass. Note the heavy vegetation, the thin overburden and the ice walls. Ice exposed in the walls is between 30 to 50 feet in height. Figure 18 was taken from the back or west side of the moraine. The low hills in the background are some 100 to 150 feet high and are solid ice. Figure 19 is a close-up of the ice; note the vegetation.

Design is being based on "passive" methods. This will mean avoiding cuts, utilizing natural drainage channels, and avoiding disturbance of natural insulation. Figure 20 shows a D-9 Cat working at mile 57 of the Copper River Highway. Note the ice exposed in the roadbed and the backslope. This area is breaking up almost as rapidly as it is constructed. "Passive" methods will probably inhibit the rate of melting, but a stable condition will not be attained until all ice has melted. This statement is based on the fact that although the ice is buried by as much as 20 feet of overburden, its upper surface is slushy and rotten having every indication of melting. Ice removal was and is being considered, but at this time seems impractical due to the tremendous quantities involved. It has been estimated that a total excavation of well over one million cubic yards would be required.

Drainage control in this area is a special problem. During the early 1900's, major streams in this area ran almost due north. They now run due south. Should the Allen Glacier advance, and this is regarded by the writer as a probability rather than a possibility, drainage channels would again be disrupted. In addition, new channels are constantly forming along the outer portion of the ice mass.

It is a gross understatement to say that design and construction present critical problems in an area such as this. The problem was <u>not</u> solved by the Copper River and Northwestern Railroad. Indeed, this section required continual maintenance and reconstruction almost yearly. Admittedly, their problem was more complex than ours since they were laying ties on ice.


ARCTIC OCEAN
Pt. Barrow
Fairbanks District
Nome District
ALASKA
CANADA
Bering Strait
Nome
Fairbanks
Valdez District
Anchorage District
Anchorage
Valdez
Prince Wm. Sound
Juneau
Juneau District
BERING SEA
Alaska Peninsula
Gulf of Alaska
PACIFIC OCEAN

Figure I

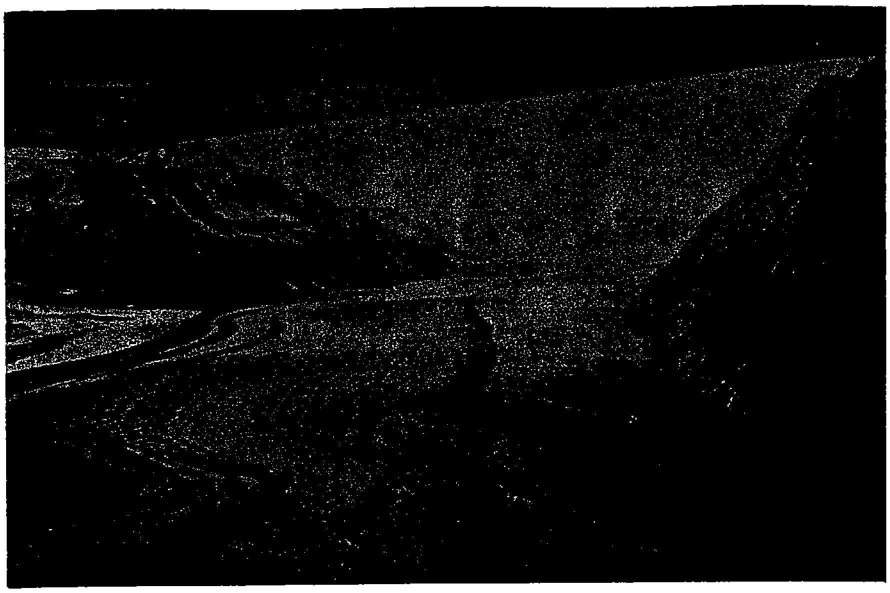


Fig. 2
Woodworth Glacier

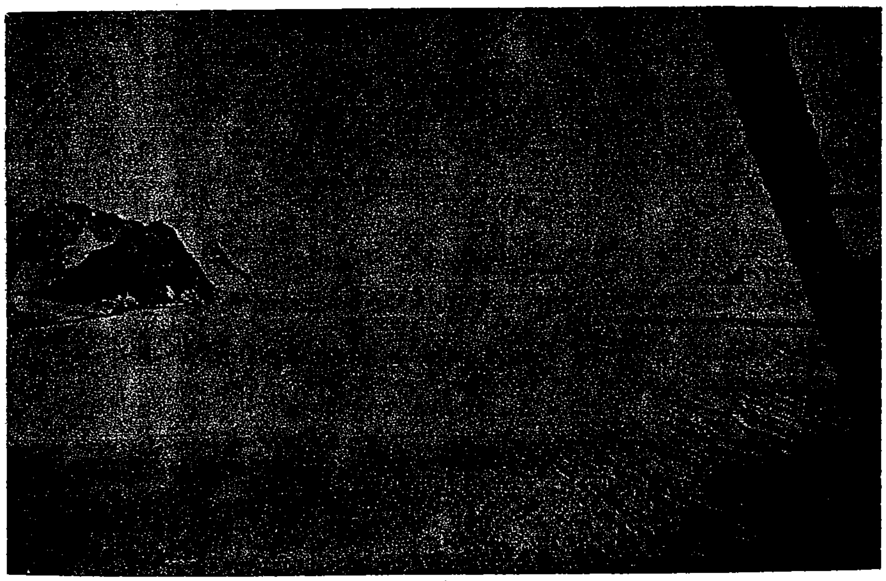


Fig. 3
Tazlina Glacier

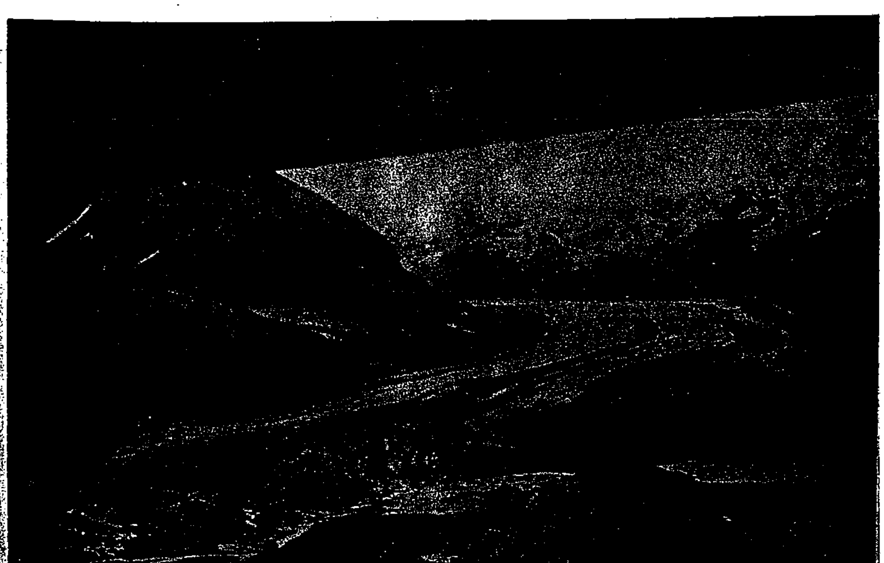


Fig. 4
Schwan Glacier

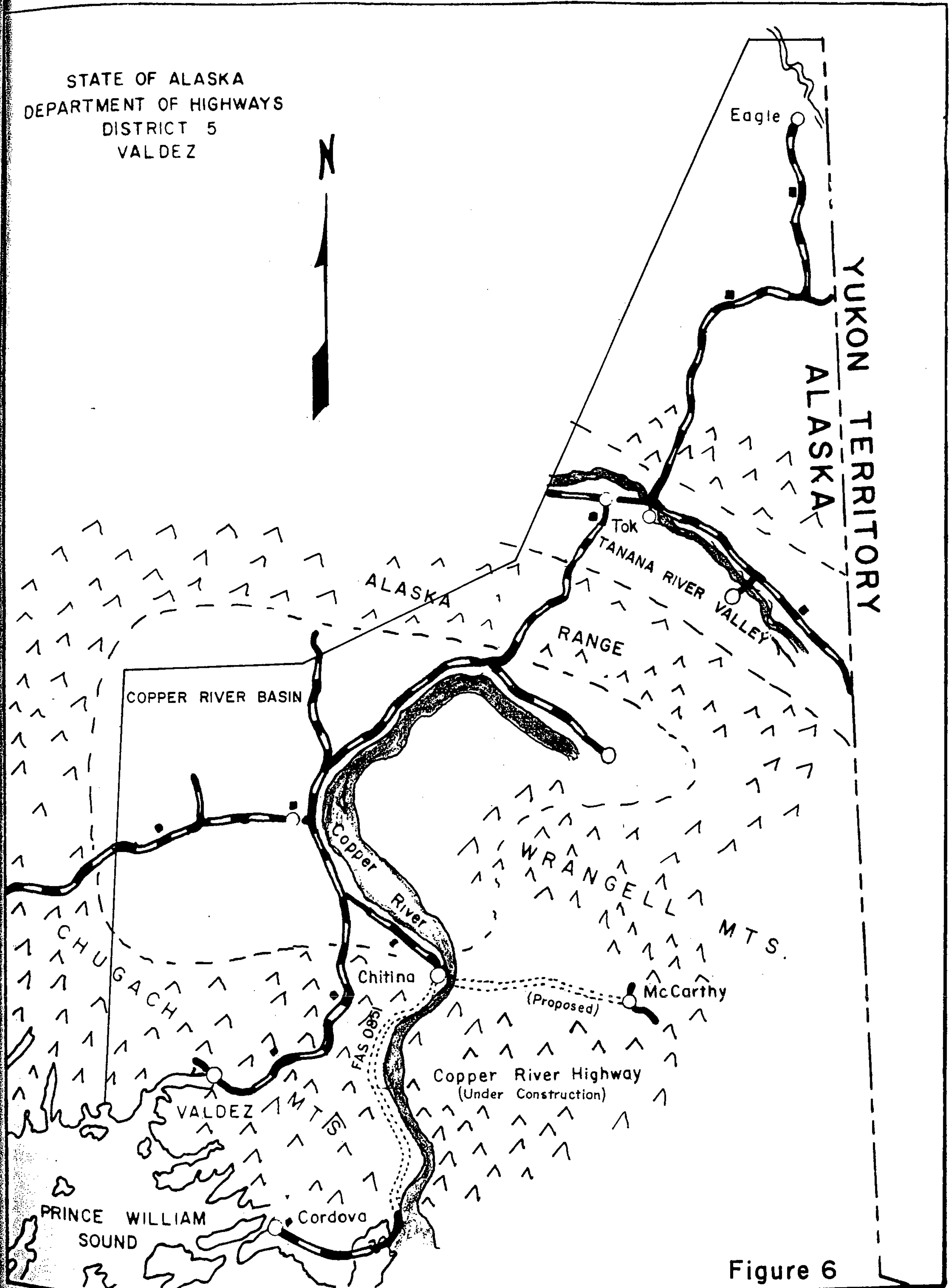

STATE OF ALASKA
DEPARTMENT OF HIGHWAYS
DISTRICT 5
VALDEZ
N
Eagle
YUKON TERRITORY
ALASKA
Tok
TANANA RIVER VALLEY
ALASKA
RANGE
COPPER RIVER BASIN
Copper River
WRANGELL MTS.
CHUGACH MTS.
Chitina
McCarthy
(Proposed)
FAS 851
Copper River Highway
(Under Construction)
VALDEZ
PRINCE WILLIAM
SOUND
Cordova

Figure 6

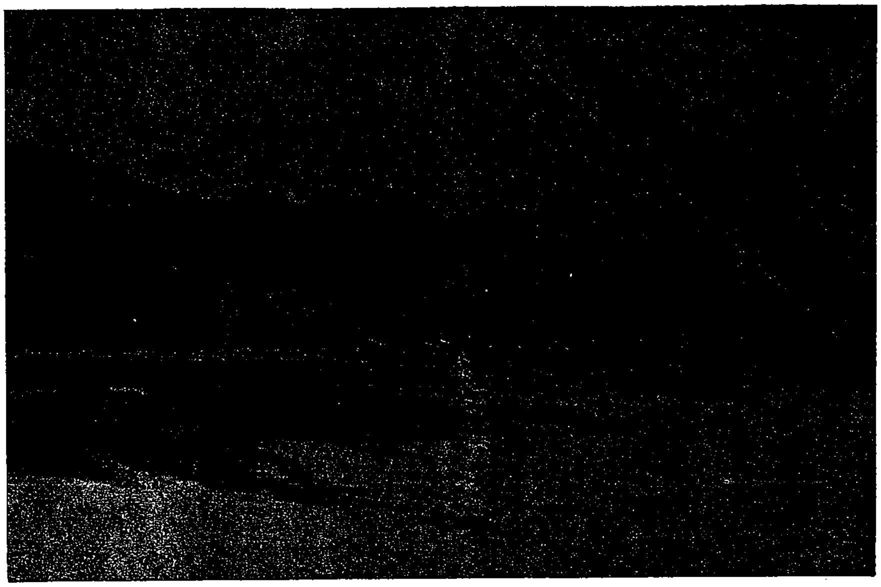


Fig. 5
Air View of
Valdez, Alaska

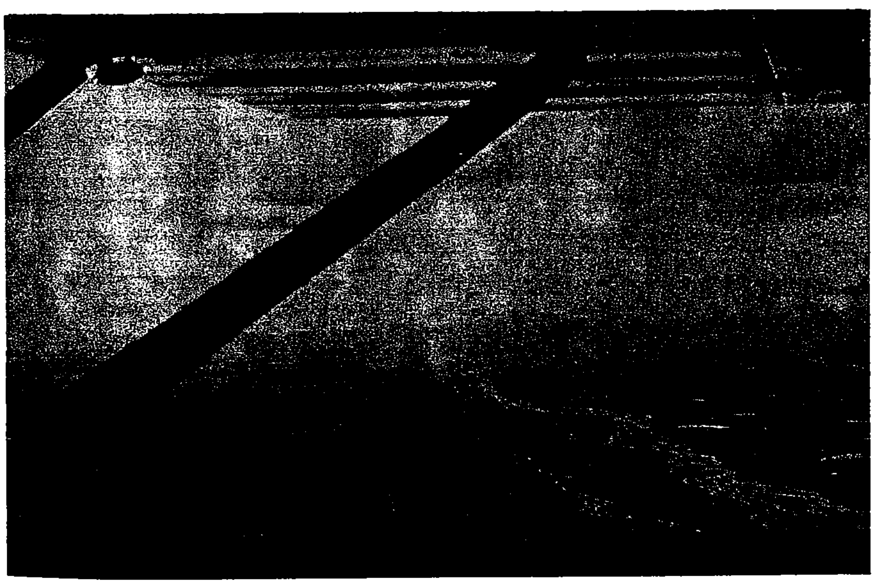
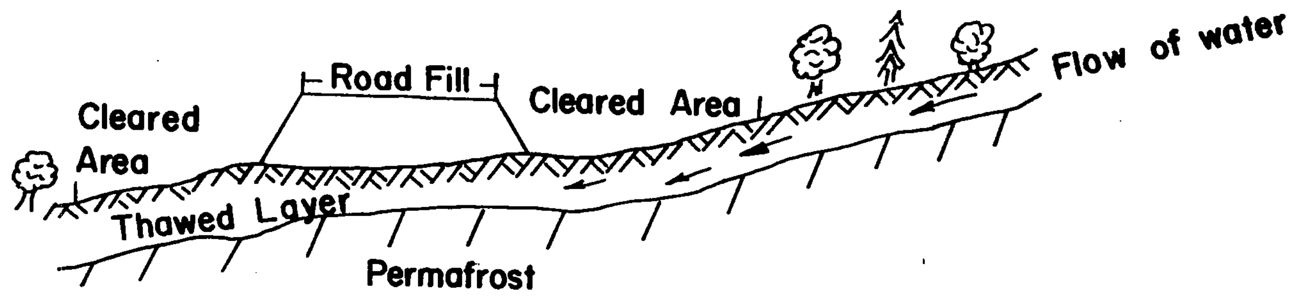


Fig. 7
Air View of
Copper River near
Slana, Alaska

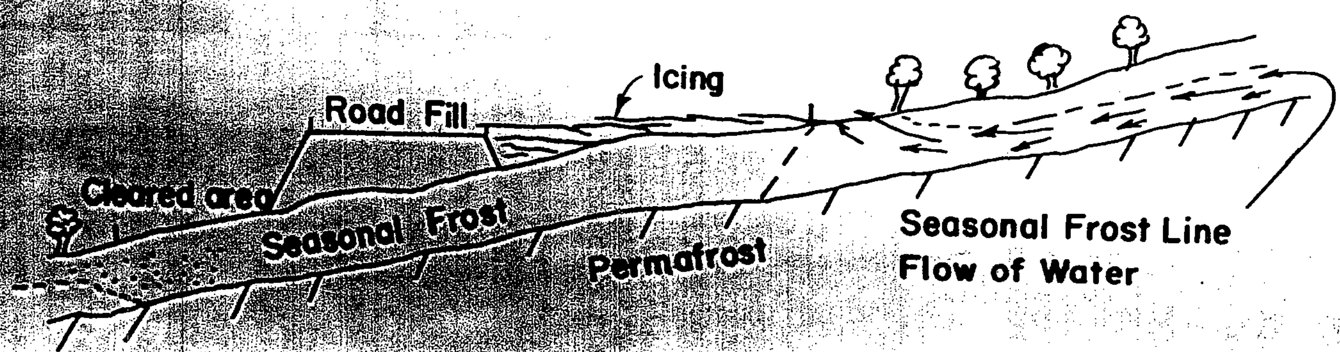
Fig. 8
Frozen Silt on
6 inch solid
Auger

Fig. 9
Surface Icing near mile 73, Richardson Highway.

CONDITION DURING SUMMER



CONDITION DURING LATE FALL AND WINTER



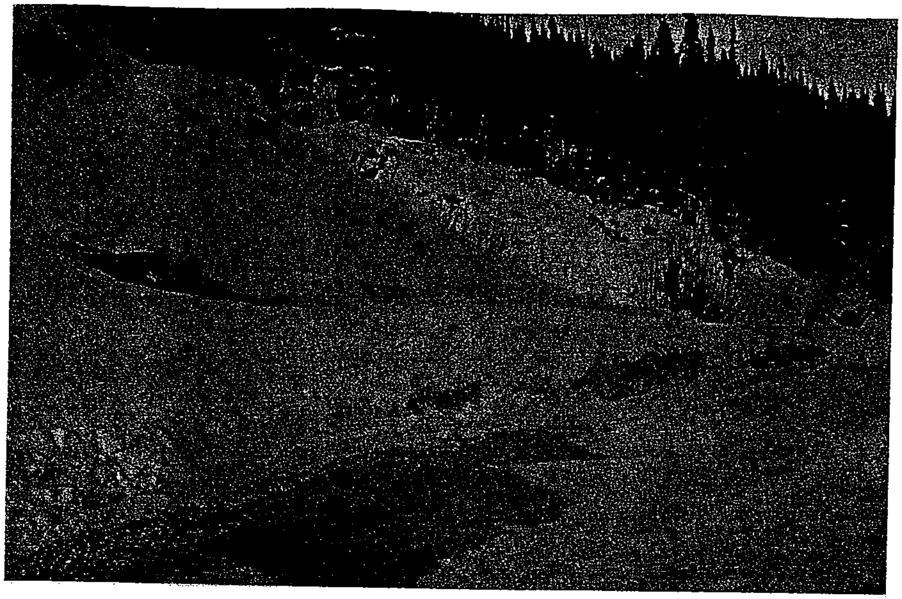


Fig. 11
Surface Icing
near mile 53,
Richardson Highway.

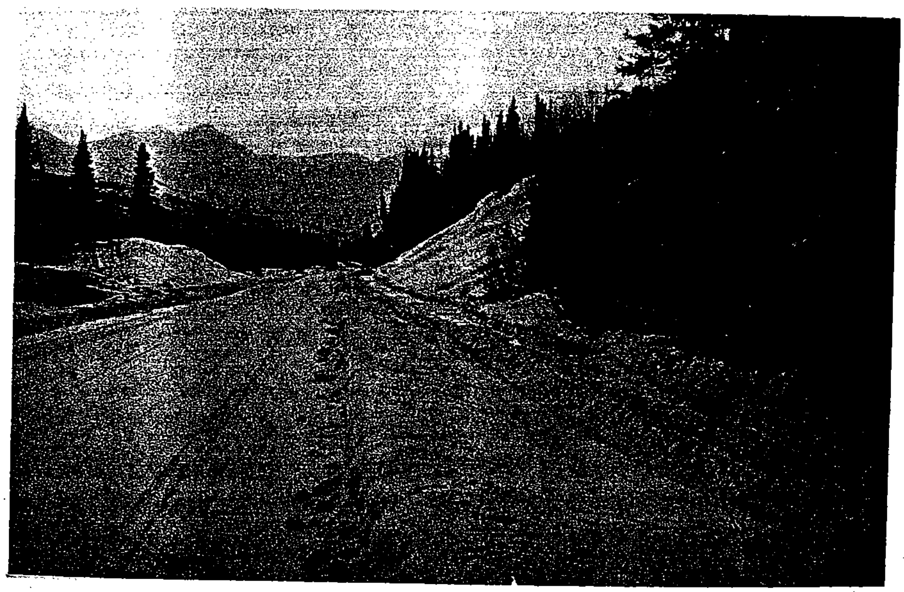


Fig. 12
Surface Icing
near mile 59,
Richardson Highway

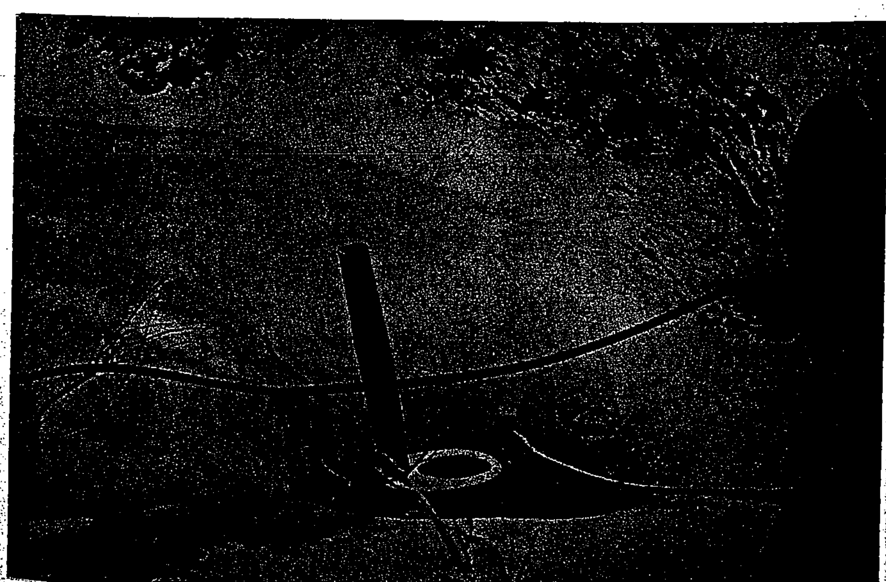


Fig. 13
Surface Icing
near mile 54,
Richardson Highway.
Culvert is under
10 feet of ice.

METHOD OF PREVENTION

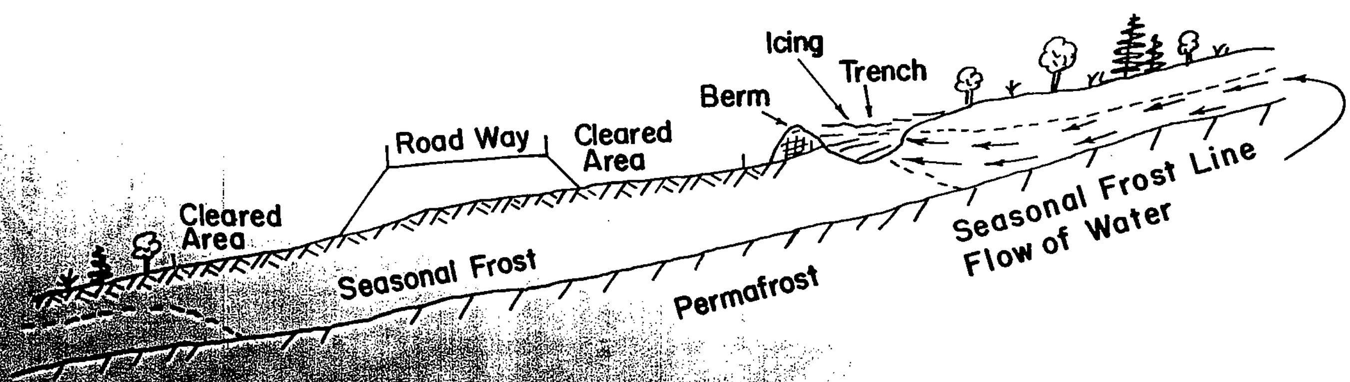
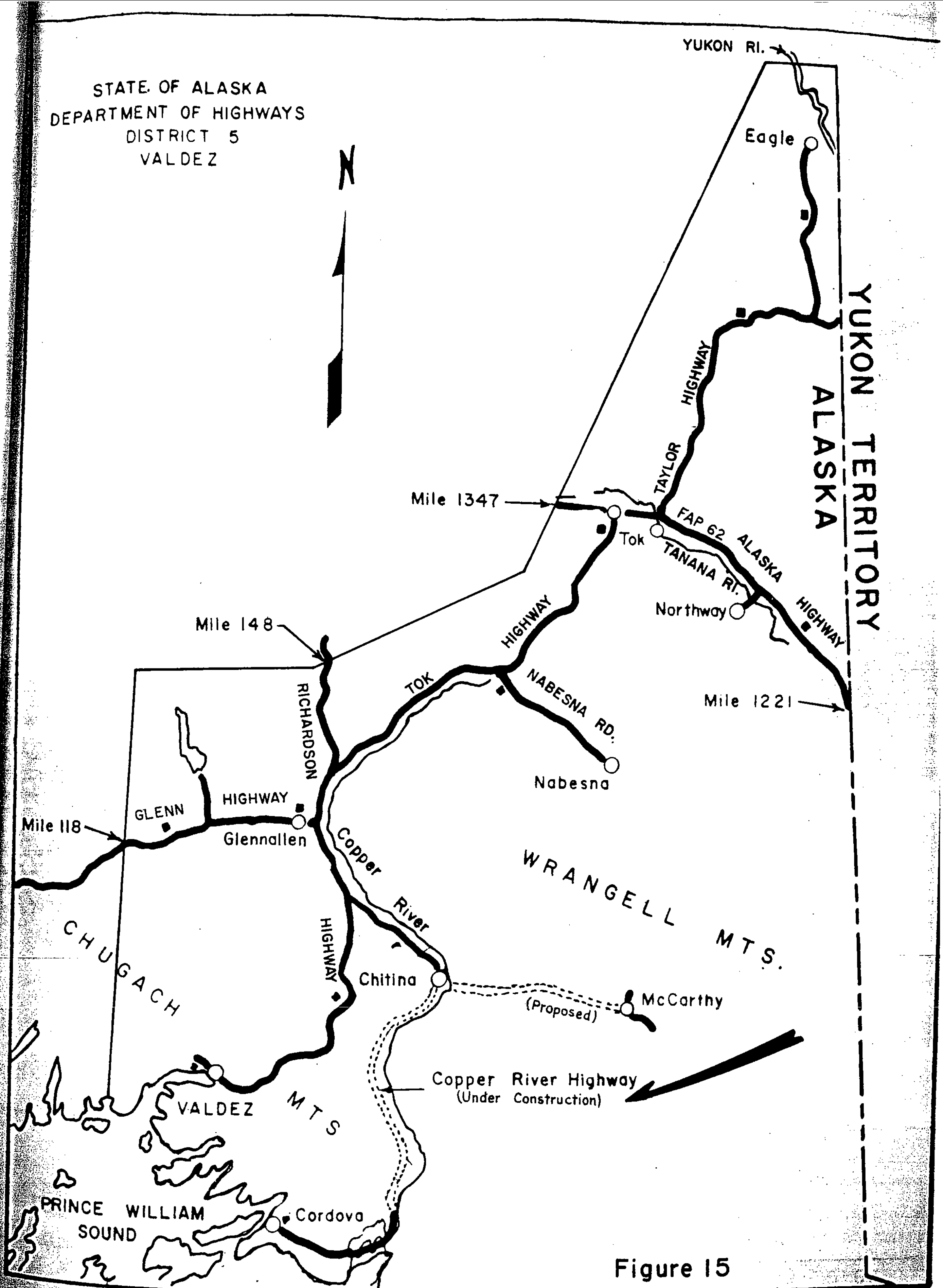


Figure 14



STATE OF ALASKA
DEPARTMENT OF HIGHWAYS
DISTRICT 5
VALDEZ
N
YUKON RI.
Eagle
TAYLOR HIGHWAY
YUKON TERRITORY
ALASKA
Mile 1347
Tok
FAP 62
TANANA RI.
ALASKA HIGHWAY
Northway
Mile 1221
Mile 148
TOK HIGHWAY
NABESNA RD.
Nabesna
RICHARDSON HIGHWAY
GLENN HIGHWAY
Mile 118
Glennallen
Copper River
WRANGELL MTS.
CHUGACH
Chitina
McCarthy
(Proposed)
Copper River Highway
(Under Construction)
VALDEZ
MTS
PRINCE WILLIAM SOUND
Cordova

Figure 15

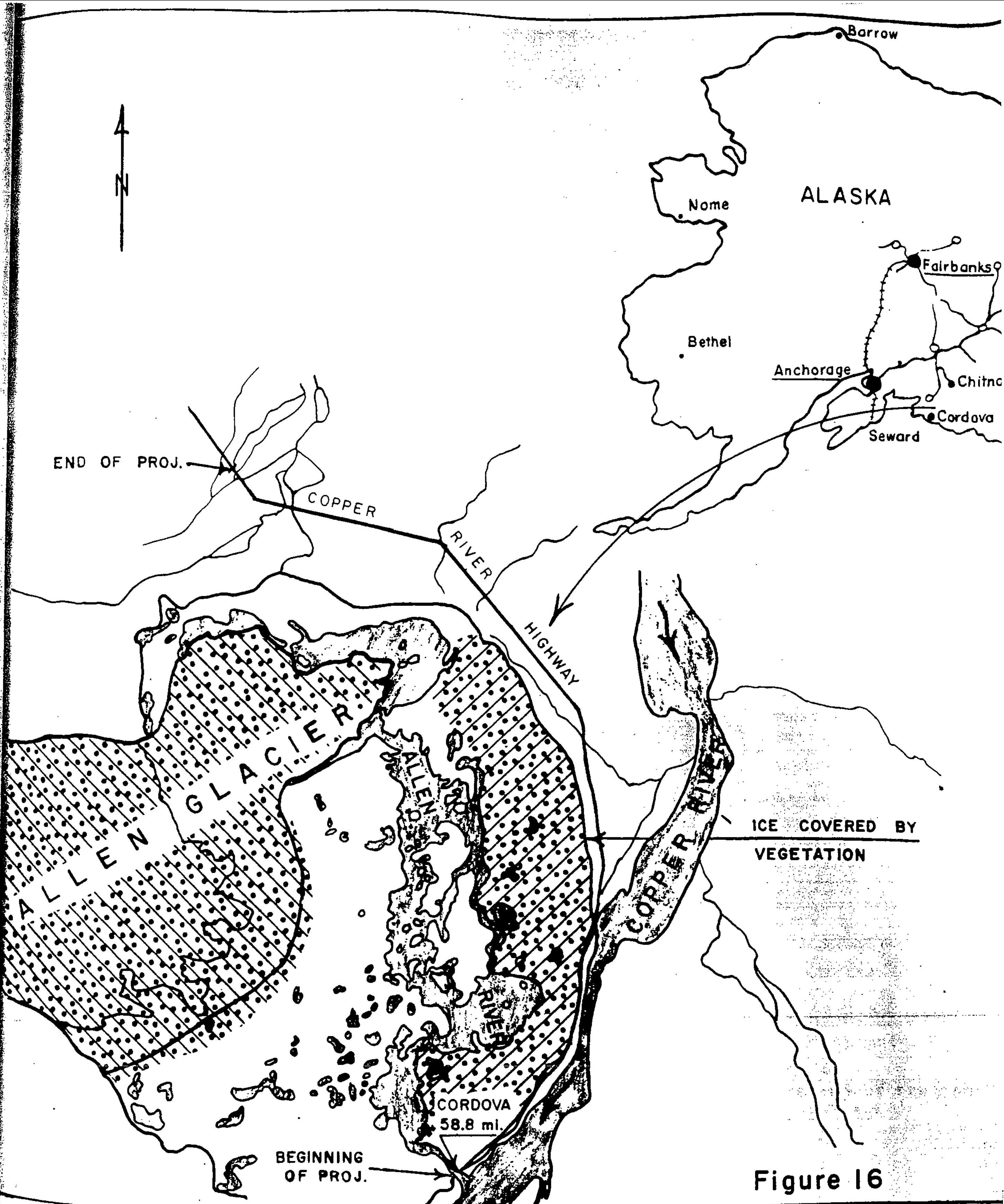


Figure 16

MAP OF ALLEN GLACIER IN 1963 ICE BOUNDARIES APPROXIMATE	STATE OF ALASKA DEPARTMENT OF HIGHWAYS VALDEZ DISTRICT MATERIALS SECTION
FROM AERIAL PHOTOGRAPHS AND TOPOGRAPHIC MAPS	SCALE 1:63,360 DATE 12-26-63 DATA R. R. M. DRAWN A. D. APPROV R. R. M.

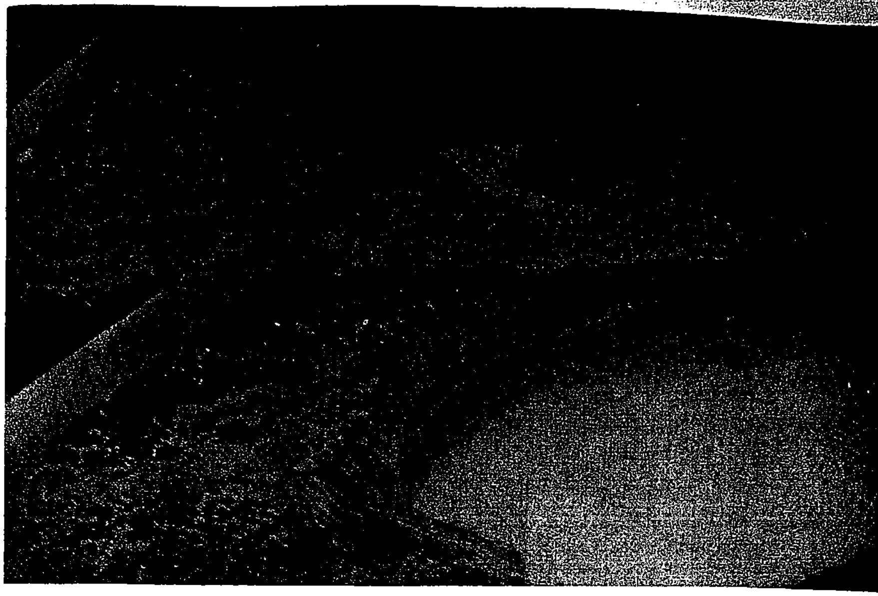


Fig. 17
Crater or kettle in stagnant glacial ice. Exposed ice is approx 100 feet thick. Photo by R. Felland, July 1963, near mile 60, Copper River Highway.

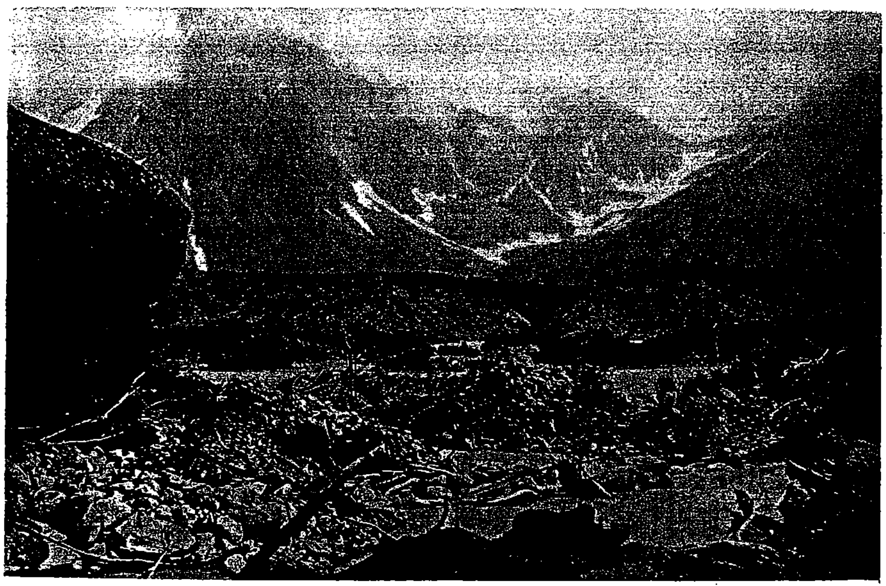


Fig. 18
Back or west side of stagnant ice mass, near mile 59, Copper River Highway.

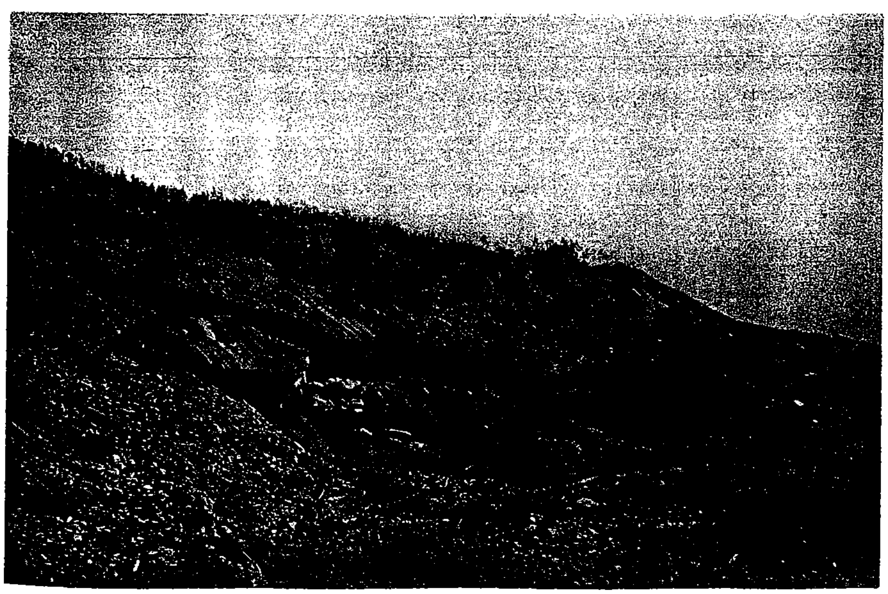


Fig. 19
Close-up of stagnant glacial ice.

Fig. 20
Highway construction in stagnant glacial ice, near mile 57, Copper River Highway.

A FOUNDATION INVESTIGATION OF CHEROKEE CAVE UNDER ROUTE I-55, CITY OF ST. LOUIS

by

J. D. Laundrum

Probably few highways in Missouri have been more affected by problems that are associated with karst topography and solution activity than has the Missouri portion of I-55 through St. Louis and southward toward Cape Girardeau. This route generally parallels the Mississippi River near the eastern limits of the Ozark Uplift through Mississippian limestones and Ordovician dolomites. Through most of one county, the originally selected line has recently been rerouted in order to avoid an almost interminable series of sinks and interconnecting solution channels. Foundation investigations of solution structures in these regions involve considerable core drilling at great expense and frequently provide meager results.

Recently, in the St. Louis area, we have had an opportunity to investigate a solution structure under conditions that permitted detailed work and observations and where the recent history was well documented. This investigation provoked considerable interest within the Department, and it is presented here with the hope that it may be of interest to others as well.

The St. Louis portion of I-55, to be known as the Ozark Expressway, has had its full share of foundation problems. Certainly one of the most interesting was that posed by crossing Cherokee Cave with a 30-foot fill near the intersection of Cherokee and Broadway Streets in the heart of old southside St. Louis. At this point, the line sweeps back into the city after skirting the river bottom and its location is rather closely limited by a large concentration of industry.

Cherokee Cave is the best known of St. Louis's many caves, and it has a history of use unique in this state. This history, to a great extent, is associated with that of St. Louis's brewing industry. Early brewers sought natural caverns or built huge cellars in which to store and lager their wares. It was for this purpose that Cherokee Cave was first developed, about 1850, by a firm called the Minnehaha Brewery. A natural sinkhole over the intersection of two major corridors was enlarged to form an entrance shaft to a large chamber some 35 feet below.

Later, other portions of the cave were developed for the same purpose by the Lemp Brewery. This company, the founder of the Falstaff label, went out of business after the advent of Prohibition. Long before this time, the Minnehaha portion of the caverns was in disuse and tenement houses occupied a landfill over the original ground surface. Openings into the cave were used for disposal of garbage and wastes.

After the second World War, the tenement house area was cleared for the commercial development of the cave as a tourist attraction. An elaborate stairwell and entrance corridor were built, providing access to the cave through another sink area. The accumulation of soil, waste and garbage, which even included a mule, was removed with most of the mess being jetted and washed away through an unexplored channel. (It is speculated that this channel enters the Mississippi River not far away). Soil and rock were removed to enlarge the cavern to an extent of perhaps three times its original volume. Supporting columns were constructed about the stairwell area to shore rock made unstable by excavation. Graveled walkways were constructed and lighting was installed. Guided tours were conducted during the summer months while enlargement and improvements were continued during the winter.

About 1957, heavy rains commenced causing flooding of the cave to near its roofline and pumping was necessary to maintain commercial operation. The operators speculated that flooding was due to blocking of natural drainage channels by grouting under new construction at the Busch Brewery, several blocks north. Eventually, the cave waters found a new channel or enlarged their old one for the cave is now, and has been for several years, draining freely with a moderate flow of water.

Cherokee Cave has been studied by a number of geologists and is described in Caves of Missouri (Bretz, 1956), a publication of the Missouri Geological Survey. It occurs in the St. Louis limestone, a formation of Mississippian age. It is believed that it was originally created by solution enlargement of joints and bedding planes under phreatic conditions of total immersion while the nearby Mississippi River was at a relatively much higher elevation. With lowering of the river into its present course, the water table lowered and air entered the cave. During this period of vadose conditions, enlargement and deepening of corridors continued with joint enlargement and sinkhole development.

Probably as a result of the sink development, the cave was filled with deposits of silt and gravel. Walls of certain corridors are composed now of these deposits with interbedded flowstone. Locally, seepage has caused slumping of this material since commercial operation ceased. It seems logical that continued erosion may occur over the years with consequent corridor widening, at least until present drainage exists are blocked. Bones of peccary and other Pleistocene animals are found in the silt and gravel deposits and are of considerable interest to naturalists and paleontologists. It is speculated that these deposits were flushed into the cave during inter-and post-glacial stages after the animals become entrapped in ponds or marshes developed over sinks. An appeal was made to the Highway Department to preserve these bone deposits and to provide access to them for future study. This was one factor in prompting a fairly extensive study and drilling investigation of the structural soundness of the cave.

During a preliminary survey, a traverse was carried down the entrance corridor and stairwell and tied to the older entrance shaft, now used for ventilation. This survey indicated some 35 feet of cover above the cave but provided no reliable estimate of rock thickness except at cave openings. Therefore, a preliminary drilling program was carried out to determine just how serious the problem really was. Auger holes were advanced to rock and cores cut through the limestone roof.

At the stairwell, a thickness of only some 4 feet of limestone could be observed. Several feet away, a drill cored only 1.3 feet of rock before breaking through into the cave through a mud filled joint. Other cores showed 8 to 11 feet of rock over this largest corridor, which had clear spans at the ceiling of up to 35 feet. From observations made in the cave, information from core and auger holes tied to the survey and with rock thicknesses estimated at cave openings, these tentative conclusions were made:

1. The area around the stairwell had to be considered structurally unsound for additional loads. In addition to uncollapsed rock shored by columns, a completely collapsed zone had been partially excavated, shored, and capped with concrete.

2. The large room adjoining the stairwell area on the west, with its large spans of up to 35 feet and its thin, jointed and seamed rock cover (8 to 11 feet), would probably be unsafe under the added load of a 30-foot fill. As a matter of judgment and without methods of rational analysis, it would appear that this area is now dangerously near failure.

3. Certain other regions of the cave could probably be considered safe. With much smaller corridor spans, thicker roof rock and less solution development of joints, the possibility of collapse seems remote. Here, the thickness of roof rock approaches or exceeds width of corridors.

4. In areas where fill load will be lightest, and approaching a cut section, roof rock thickness continues to increase to an obvious and comfortable margin of safety.

Determination of the thickness of roof rock was the primary object of the preliminary drilling phase. But other questions were posed which required answers. Were there additional undiscovered caverns or rooms under the right of way? Was there a possibility of additional levels below those known? A suspected sink, previously unnoticed, was pointed out, and local residents recounted legends of numerous other sinks supposed to exist in the neighborhood.

What lay beyond the collapsed zone at the stairwell? Conversations with people who had helped clear the cave during its development brought out the fact that heavy planks had been uncovered leading into the collapsed zone. This indicated early use of additional rooms in the cave by the Minnehaha Brewery and that the collapse was historically recent.

An overpass spanning Broadway Street was to be constructed immediately east of the cave grounds. It now seemed desirable to make a preliminary foundation investigation for this structure.

A second and more extensive drilling program was started to clear up these problems. Additional cores were cut within right of way limits and outside the limits of the known cave. In every core, one or two silt and gravel filled seams were found within elevations bracketing the cave floor and ceiling, with some seams located at irregular elevations above the ceiling. The limestone generally was hard and dense but with parting planes irregularly and closely spaced. Sylolite growth was heavy throughout.

Several cores were taken, to a depth of some 20 to 30 feet below the known cave floor. Within this zone almost no evidence of solution work was found except for a very few stylolites. One hole, near the stairwell region, found soil but no rock to a level near the cave floor. Others found 5 to 8 feet of limestone covering a 4.5 to nearly 6-foot thickness of silt and clay. East, across Broadway and Seventh Streets, borings encountered a jumbled mass of clay and limestone boulders indicating total collapse and filling. This evidence clearly confirmed that the easterly extent of this major corridor had been originally much greater and that the collapsed zone had not been completely reexcavated by the latest developers. Fortunately, no open cavities were found through this area.

Near the suspected sink, a pair of core holes were drilled. These holes encountered a 0.5-foot open cavity above cave roof elevation with an additional 1.5-foot thickness of silt and gravel. Fill height is negligible here, and no special treatment is planned for this area.

This second and more extensive drilling program served mainly to still fears and suspicions of other and unknown caverns. It did affect the conduct of a later foundation investigation for the adjacent overpass structure over Broadway. There, increased use of cores was made to a depth below the known level of solution activity. At least one of these later cores also encountered the filled extension of the collapsed corridor previously described. Steel H-beams, carrying substantially reduced loads, will be used for the overpass structure foundation.

No precedents in Department experience existed for knowingly crossing a shallow cavern with a high fill. Opinions were sought from geologists, engineers, and an explosives expert-- those outside the Department who were qualified by experience in mining and tunneling to offer advice. Predictably, there was some range in the suggested remedial measures, but no one who inspected the cave, so far as is known suggested that nothing should be done.

The possible courses of action which could be taken with this problem fall into these general categories:

1. Build the fill and do nothing. This course of action accepts the risk of collapse, the consequence of which would be a loss of support under the fill with possible long term subsurface erosion of foundation soils by drainage waters. It seems apparent that some collapsing action has occurred within the last one hundred years, possibly as a result of land filling and leveling. It seems probable that additional loads will cause further collapse.

2. Shoot and collapse the cavern with explosives. The explosives expert who inspected the cave recommended against this as a general method of treatment. With the degree of urban and industrial development in the neighborhood, litigation as a result of blasting damages, real or imagined, is a considerable hazard. It is known that the cave developers had to contend with this as a result of blasting done while enlarging and deepening corridors. A further disadvantage is the possibility of imperfect collapse and subsequent erosion of foundation soil.

3. Construct supporting beams and columns. This would permit keeping the cave open. Disadvantages include a difficult design problem posed by badly jointed and fissured roof rock. Periodic inspection and maintenance would probably be required. Fortunately, it appears that most of the bone deposits in question are outside the area which will be affected by highway construction. Needless to say, the Department has no desire to be in the "cave business" in any form if it can be avoided.

4. <u>Fill Unsafe Rooms</u>. Possible methods include grouting, lowering of materials through the ventilation shaft to be packed by hand or some mechanical procedure, and depositing saturated chat or gravel through holes drilled in the roof.

The details of treatment which will be used are not complete at this time but probably will include the following:

1. The entrance corridor, stairwell and collapsed sink area will be excavated, and refilled with select backfill. This form of treatment may be extended to the stairwell area. Some collapsing of unsound rock after excavation and prior to backfilling will be required.

2. Corridors considered unsafe or possibly unsafe will be enclosed with bulkheads and filled with grout. Present drainage will be maintained through grouted or filled areas with pipe strapped to, or entrenched in, the floor to prevent buoyant rise in plastic grout. Maintenance of present drainage is believed particularly important to prevent flooding of corridors and erosion of new channels.

The solutions to be used are, of course, not unique. Under different conditions, other approaches might be more applicable. The procedures used here are perhaps more elaborate and more expensive than would be justified under other than a major traffic artery.

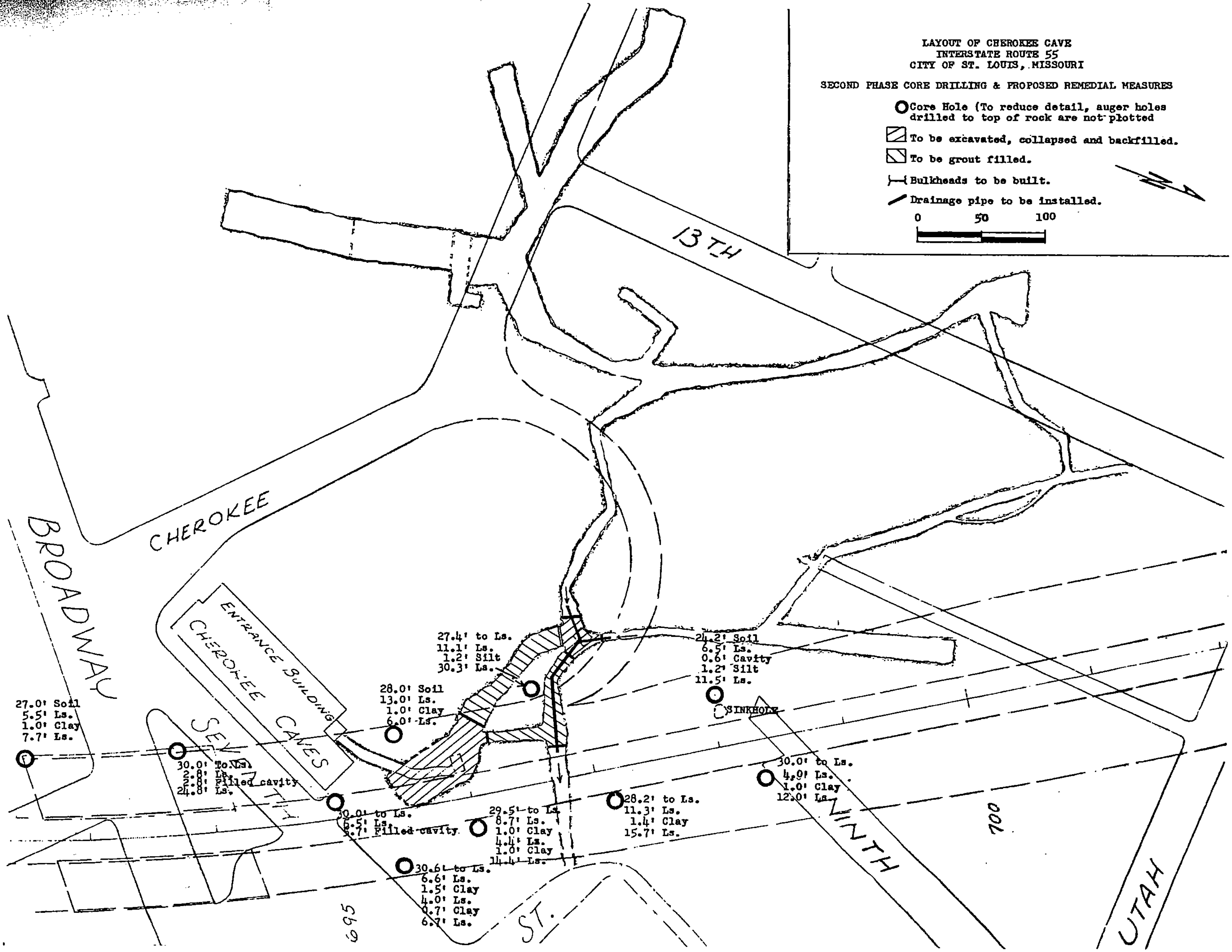
SUMMARY

From this study of Cherokee Cave, the Geology and Soils Section has gained a new appreciation of the complexity and inter-relationships of solution developed structures. Specific episodes in the cave's recent history point up the two major hazards that confront road builders in regions of solution activity. Blocking of solution channels at one point, as is believed to have

occurred under the Busch Brewery, may have unforseen consequences elsewhere in a subterranean drainage system. An example of the second and greater hazard, that of collapse, has already occurred, probably triggered in this instance by excavation and loading with landfill. The proposed remedial measures, hopefully, will prevent reoccurrences of these events.

LAYOUT OF CHEROKEE CAVE
INTERSTATE ROUTE 55
CITY OF ST. LOUIS, MISSOURI

SECOND PHASE CORE DRILLING & PROPOSED REMEDIAL MEASURES



PETROGRAPHY APPLIED TO THE DETECTION OF DELETERIOUS MATERIALS IN AGGREGATES

by

R.W. Lounsbury and R. L. Schuster

INTRODUCTION

Many undesirable or deleterious aggregate materials are noted in specifications for highway construction. These range widely in nature and composition, but share the common property of producing concrete of poor performance record.

Specifications delineate such materials and set limits on properties and amounts permissible in aggregates. Quality control is established by visual examination, standard acceptance tests, and recommended practices, such as ASTM, AASHO and others. The tests and practices are often modified. Frequently used are the Los Angeles abrasion test, sodium and magnesium sulphate and freeze-thaw tests for soundness, potential alkali reactivity, specific gravity and absorption, and other tests.

Some of the deleterious materials listed in specifications include chert, coal, clay lumps, shale, nondurable particles, ocher, shells, soft particles, coated grains, alkali mica and objectionable matter, among others.

Visual inspection and the tests mentioned above serve to indentify the undesirable constituents. The use of geologic methods, particularly petrologic and petrographic analyses, may profitably supplement the more familiar practices. Petrographic examination of aggregates has been employed for nearly two decades.

Some clarification of the meaning and scope of petrology and petrography should be made. These terms vary with authorities, but petrology may be regarded as the science or study of rocks. Petrography deals with the details of description and classification of rocks, and is generally microscopic in character. It is of interest to note that within the ASTM Committee C-9 on Concrete and Concrete Aggregates, the subcommittee on Petrography of Concrete and Concrete Aggregates has been currently discussing the scope of petrography applied to aggregate examination, whether it should be restricted to microscopic means or should include the tools of x-ray, differential thermal analysis, electron microscope, infra-red and other analyses. Reversing an earlier stand, the present authors will include the latter, broader scope of petrography in this paper.

UTILITY OF PETROGRAPHIC EXAMINATION

Petrography may detect potentially deleterious aggregates, which escape visual examination and standard tests. Frequently important correlations between standard test results and engineering properties and petrographic characteristics may be established. These are helpful to explain past performances, predict future behavior of aggregates, and to weed out undesirable constituents.

In addition to the deleterious effects of common nondurable and unsound stone particles, aggregate problems arise through cement-aggregate reactions (siliceous and certain carbonate aggregates), other chemical reactions, and low specific gravity cherts. Also some aggregates adversely affect the skid resistance properties of bituminous concrete. Frequently some of these aggregates may only be identified through petrographic examination.

HISTORY

The use of petrology in evaluating aggregates goes back to the late 1920's (Laughlin, 1928). Petrographic examination of aggregates was utilized by the

Bureau of Reclamation as early as 1936. This method was also used by the Corps of Engineers before 1940. By the late thirties field and laboratory studies had demonstrated that alkali-high cements and certain siliceous aggregates react to produce deterioration of concrete. Stanton (1940) published the first report of this reaction. Rhoades and Mielenz (1946) described the petrographic examination of concrete aggregates. In 1947 McConnell, Mielenz, Holland and Greene summarized the work to date on cement-aggregate reaction in concrete, and showed the utility of petrography in identifying deleterious aggregates.

Increasing use of petrographic examination of aggregates was made in subsequent years, and a Tentative Recommended Practice for Petrographic Examination of Aggregates for Concrete (C 295) was adopted as standard by ASTM in 1954. Mielenz (1954) wrote a comprehensive procedure for the petrographic examination of concrete aggregates. Other contributions to petrographic analysis of aggregates were made by K. and B. Mather (1950) and others in following years.

With the need for more detailed information in the 1950's, the use of differential thermal analysis, x-ray diffraction, the electron microscope and other tools was included or supplemented microscopic petrographic examination of aggregates. Petrography was linked to chemical and physical tests. Lemish (1963) has pointed out that prior to 1955 microscopic methods were chiefly employed by petrographers. Following recognition of the carbonate aggregate reactions, further refinement and expansion of analyses, as noted above, were made. Lemish and his associates in Iowa and Swenson in Canada have been active in pursuing these investigations.

Currently petrographers are seeking to advance the knowledge of relations between aggregate composition, structure and texture on the one hand,

and engineering properties and service performance on the other. This may be accomplished through a better understanding of siliceous and carbonate aggregate-cement reactions, degradation studies, polishing characteristics of carbonate aggregates in bituminous and Portland cement concrete, deleterious effects of clay alteration products on bonding properties of bituminous mixes and other phenomena.

SOME APPLICATIONS OF PETROGRAPHY TO THE DETECTION OF DELETERIOUS AGGREGATES

Detection of potentially deleterious aggregates and those which have reacted chemically in concrete can usually be determined by microscopic techniques, if visual observation cannot identify these. The research and literature on cement-siliceous aggregate reactions are voluminous. Since the identification of carbonate aggregate reactions, interest, research and bibliography are rapidly increasing in this area. Petrography applied to these aspects of aggregate problems will not be considered in detail here. Some glacial aggregates of the midwestern states present special problems in the climatic conditions imposed on them, and examples of these will be given.

Limonitic concretions, ocher, soft particles, such as shale and clay lumps, can usually be readily identified by visual observation. For example, porous ironstone or limonitic concretions in glacial gravels have been troublesome in Indiana highway construction. Certain deleterious rock types, unfavorable textures and microstructures, and other features cannot.

Petrographic examination of fine grained rocks using binocular and polarizing microscopes can be used to identify cherts, siliceous and dolomitic limestones, acidic volcanic rocks responsible for cement-aggregate reaction. Reaction rims about aggregate particles in deteriorating concretes can often be seen in both polished and thin sections of these latter aggregates. Pattern cracking is common in affected Portland-cement concretes made with these aggregates. Microfractures are larger and more abundant in these distressed concretes. The problem is most serious in the western and Great Plains states.

Gravels of western states containing rhyolitic and andesitic glasses and tuffs, some chalcedonic cherts, opal, and opaline shales seem most prone to the cement-siliceous aggregate reaction. Distressed concretes using Platte and Republican River gravels have been reported in Kansas and Nebraska. Even midwestern gravels have been suspect. A recent study of Weigel (1963) which employed petrographic examination of northern Illinois glacially derived gravels exonerated these gravels suspected of reactivity. It should be pointed out that this investigator concluded that though petrographic examination and chemical tests were most rapid to obtain evidence of reactivity, these were not conclusive evidence in this case.

Cherts and shales are abundant in glacial gravels, potential aggregate sources in most midwestern states. Schuster (1961) and Schuster and Lounsbury (1960) investigated the durability of concrete containing glacial cherts and shales. In this study, varying percentages of shales and cherts were incorporated in concrete beams which were subjected to up to 300 cycles of freezing-and-thawing. The remainder of the coarse aggregate in these test beams was crushed limestone of good service performance record. Cherts used were separated into bulk specific gravity categories, including chert greater than 2.55 and chert less than 2.45. Figure 1 after Schuster shows the distribution of cherts of different specific gravities used in this work.

From this study it was learned that beams containing more than 6 percent of chert with specific gravity less than 2.45 suffered significant deep-seated and surface deterioration during freeze-thaw testing. Typical deterioration shown by the Ohio River gravel is illustrated by Figure 2. Six to ten percent of porous shale produced severe "popout" damage of the beams without deep-seated failure. The investigation indicated that a combination of pore characteristics, chiefly volume of microvoids, is probably the main factor in determining chert durability.

Petrographic examination, including thin-section, x-ray and differential thermal analyses, were made of materials used in the study. The durability of the cherts has been chiefly attributed to the pore characteristics as noted above. Thin section study demonstrated a consistent relation between low durability cherts (minus 2.45 specific gravity) and the presence in section of abundant voids in the range of a few-hundredths millimeters in diameter. These were abundant only in the minus 2.45 fraction. Further, thin sections showed that voids frequently originated from solution of carbonate grains in the chert.

Petrography also offers an effective approach to the detection of deleterious carbonate aggregates which are potentially reactive. Petrographic analyses, including x-ray and D.T.A., probe the composition and properties of these rocks, and supplement chemical analyses and physical tests employed in research on the cement-carbonate aggregate reaction.

Lemish (1962) postulates that this reaction is cause by argillaceous dolomitic rocks with relatively high insoluble residues, and emphasizes the point that the manner in which the carbonate reaction contributes to concrete failure is not yet fully understood. Hadley (1961) has attributed cement-carbonate aggregate reaction to stone with 40-60 percent dolomite in the total carbonate fraction, 10-20 percent clay, and testures of calcilutites (consolidated carbonate muds). The partially dolomitized calcilutites consist of dolomite rhombs in a fine-grained matrix of calcite and clay.

The composition and textures of these rocks can be determined by the petrographic means mentioned in the proceding paragraph. The means of estimating the calcite/dolomite ratios in these rocks through differential thermal is illustrated by Figure 3 after Aughenbaugh (1963). Comparison of aggregate samples with these prepared control mixtures gives a semi-quantitative analysis of composition, the area under the curves is approximately proportional to the reacting substances.

The x-ray approach to identification of aggregate constituents is shown by Figure 4a and 4b from Aughenbaugh. These are analyses of fines produced by compaction and of insoluble residues from Indiana crushed stone aggregates. The positions and size of the reflection peaks are characteristic of the kinds and amounts of diffracting materials in the aggregates.

Textures including grain size determinations can be made from thin sections of the aggregates with the petrographic microscope, and correlations between certain textures and composition can be made. From the total petrographic analysis aggregate performance can be predicted. Where service records are available, correlations are established between petrographic findings and performance of concrete.

One other application of petrographic examination to evaluation of aggregate performance may be briefly mentioned. This concerns skid-resistance qualities of aggregates in Portland-cement and bituminous concrete mixtures.

Shupe and Goetz (1959) reported a laboratory method for evaluating pavement slipperiness. Essentially this device measures slipperiness through the application of a rubber shoe to rotating laboratory test samples of concretes which have been subjected to simulated traffic wear.

Shupe and Lounsbury (1959) utilized this method in studying the polishing characteristics of mineral aggregates. Figure 5 from their paper shows the relative skid resistance values (RRV) of various aggregates which were tested. A Kentucky sandstone with an RRV of 1 was used as the reference material for comparison purposes with the other aggregates. The letters in the figure indicate the following aggregates: L dolomite and limestone, C chert, R rhyolite, Q quartz, S sandstone, D diabase, G granite and SL slag.

Note samples L2 and L4 which show good resistance values for carbonate aggregates. These Indiana aggregates are crystalline granular dolomitic limestones with sutured textures. L7 and L11 are respectively a very fine grained argillaceous limestone from Virginia and a limestone from Indiana consisting chiefly of oolites in a calcite matrix. Both exhibit poor skid resistance properties. All of these features are easily identified by microscopic petrographic examination, and skid resistance may be predicted by thin section analysis.

SUMMARY

Petrographic analyses offer relatively rapid means for detecting potentially deleterious materials in aggregates. Thin sections under the polarizing microscope and binocular microscopic studies of rocks permit identification of compositions and textures of rocks that cannot be determined by visual observation. Aggregates which have reacted chemically in concrete may also be detected.

D.T.A. and x-ray analyses afford further pertinent data for the evaluation of aggregate composition and prediction of performance. The combined petrographic analyses may be correlated with chemical and physical tests and the service records of concretes. These analyses are widely used now, and research applying petrography may hold the key to some of the unanswered problems of aggregate phenomena in concrete.

ACKNOWLEDGEMENTS

The authors wish to thank Mr. R.C. Parks for assistance in preparing the figures and to Dr. J.F. McLaughlin for helpful suggestions.

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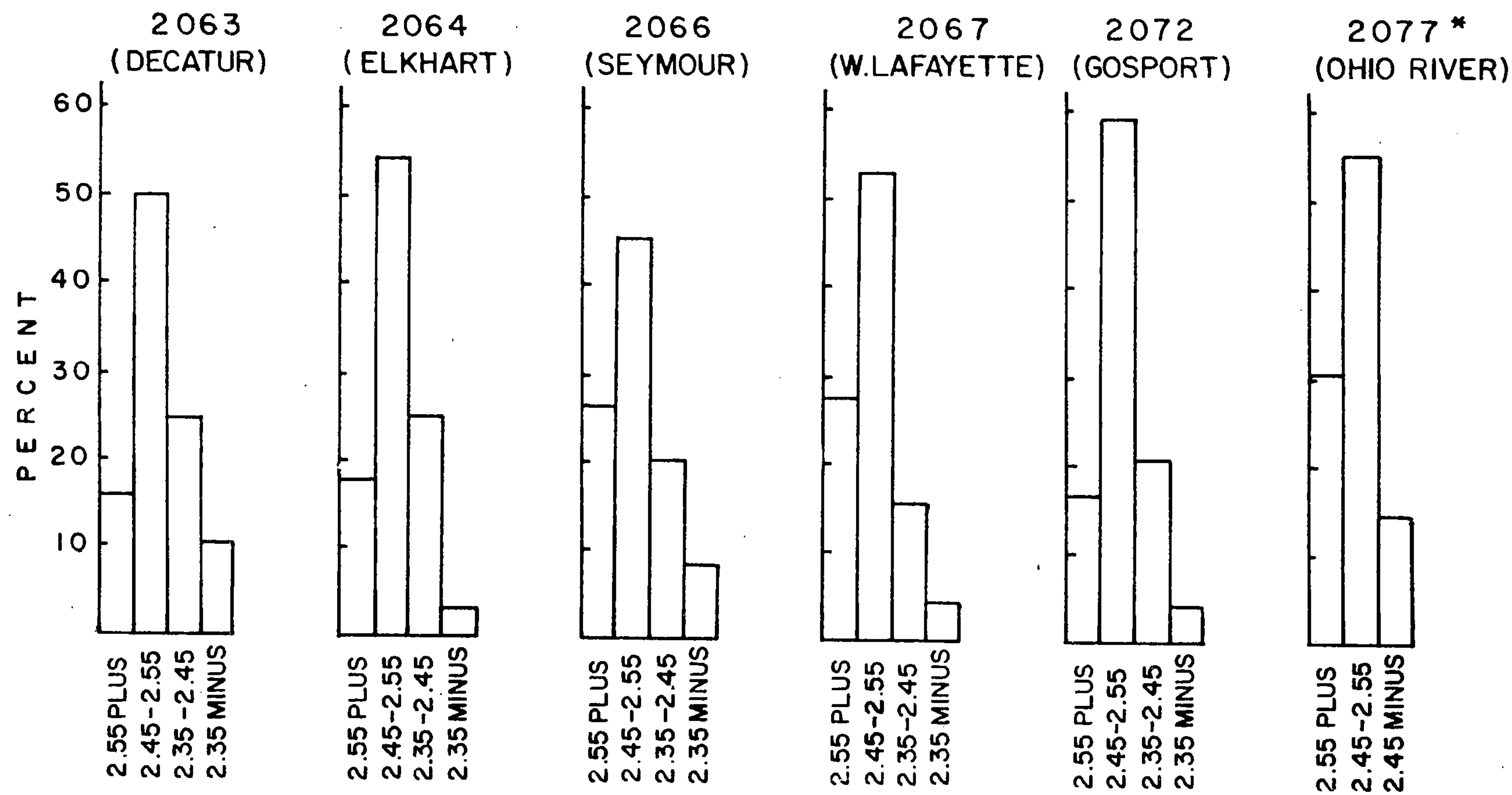

* NOT SEPARATED AT 2.35 LEVEL.

FIGURE I. PERCENTAGES OF CHERT SAMPLES IN DIFFERENT BULK SPECIFIC GRAVITY RANGES (SATURATED SURFACE-DRY BASIS)

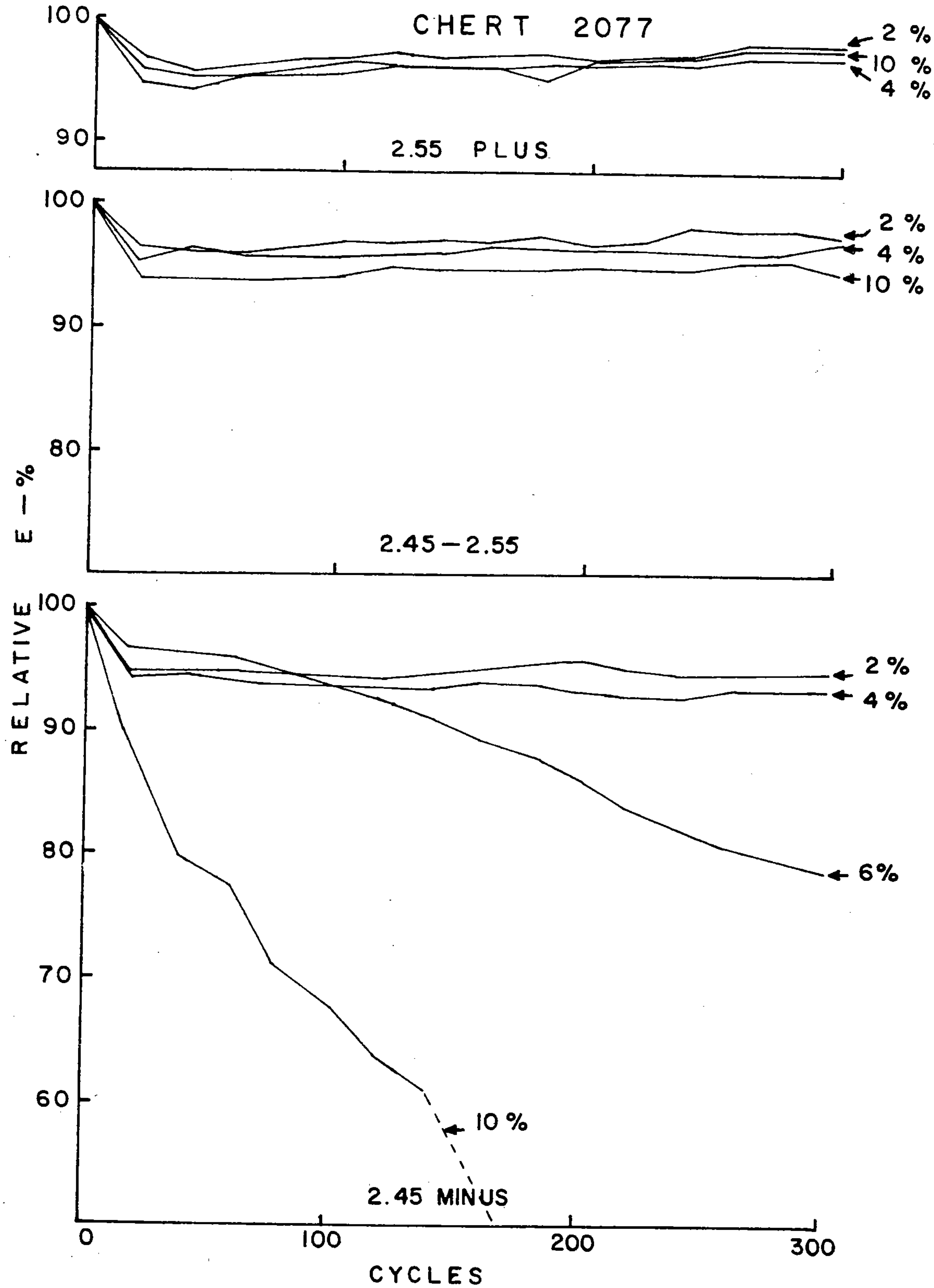

FIGURE 2. AVERAGE FREEZE-THAW CURVES FOR CONCRETE BEAMS CONTAINING CHERT 2077

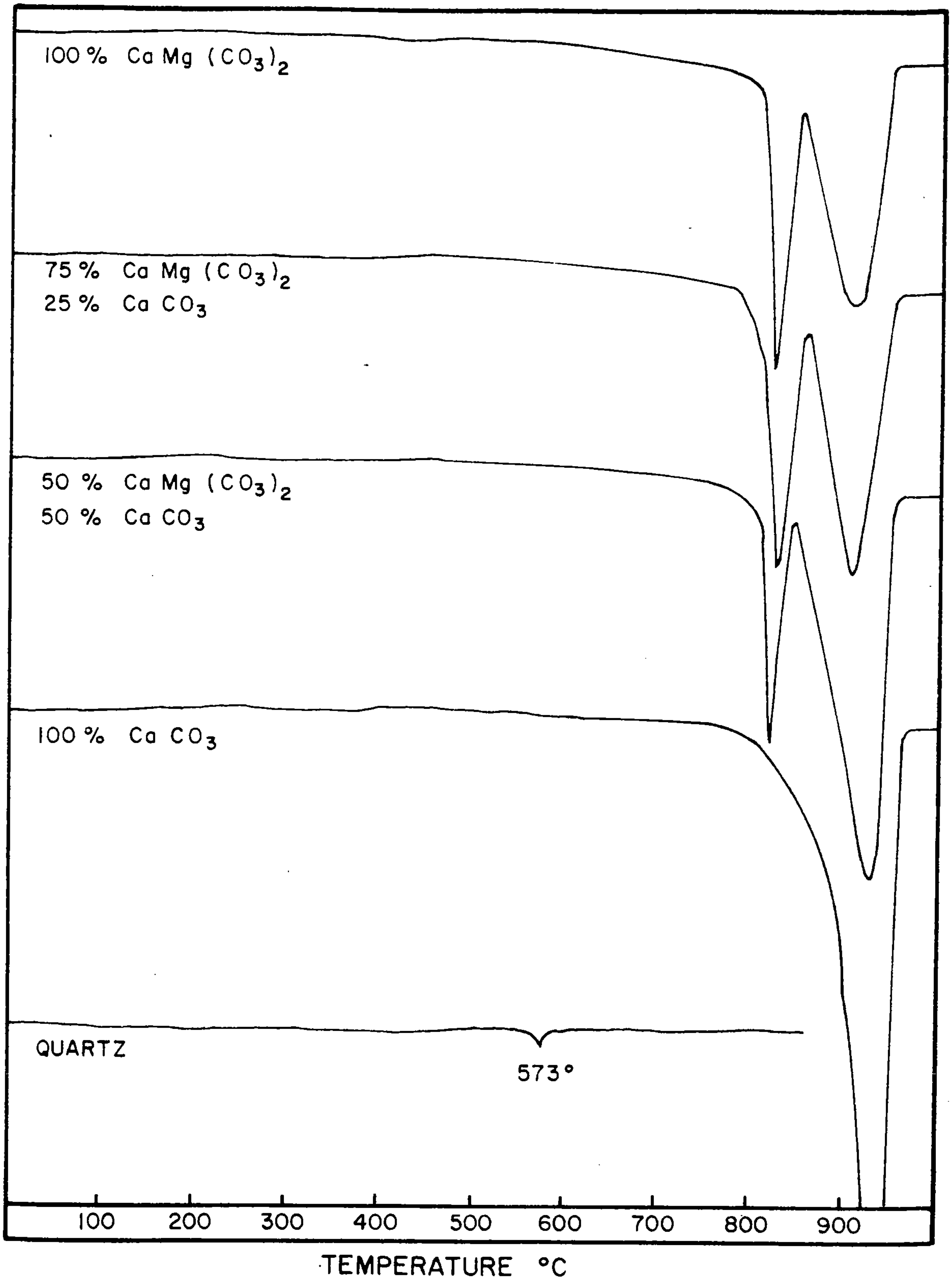

FIGURE 3. THERMOGRAMS FOR CONTROLLED COMPOSITIONAL RUNS.

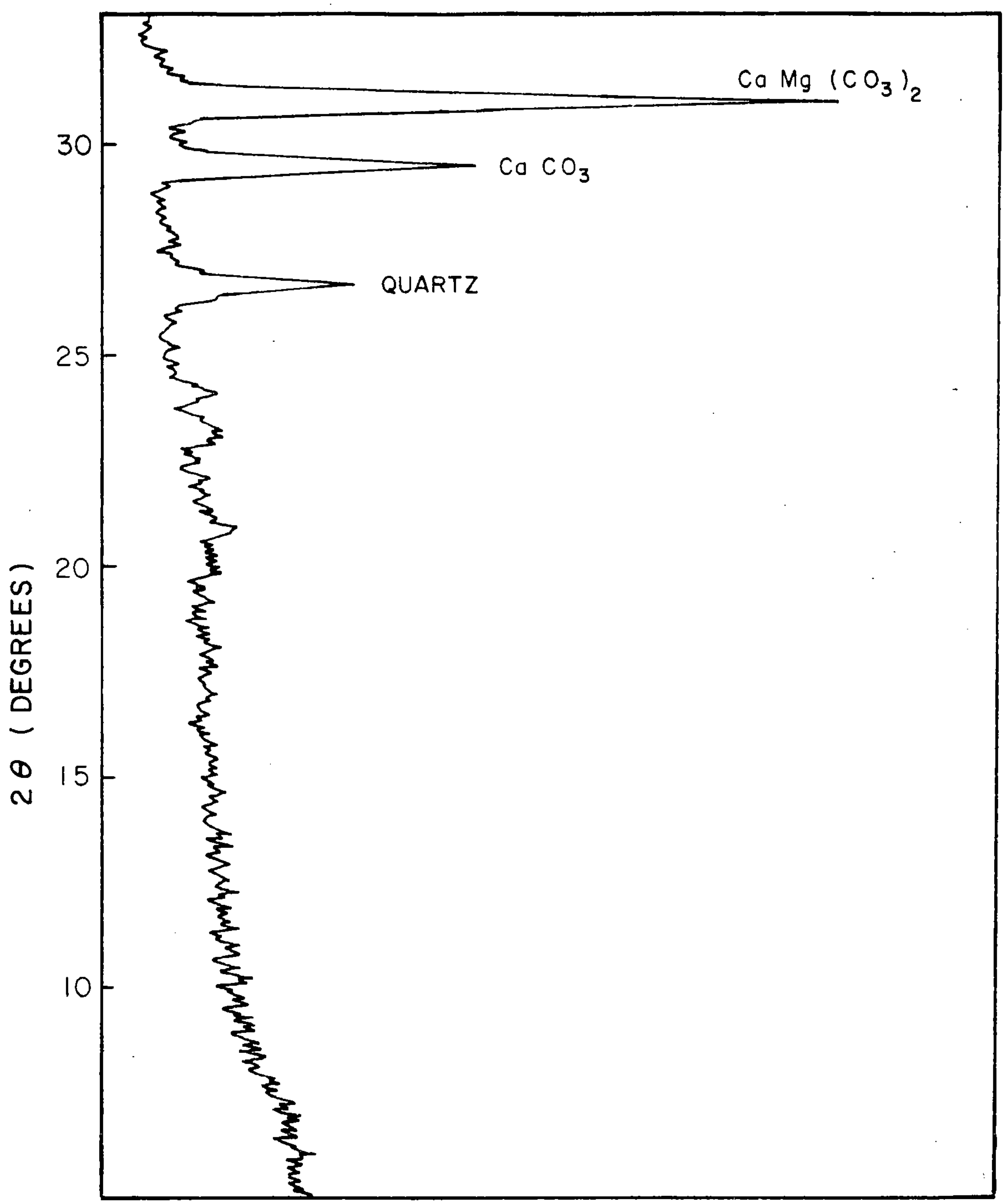

FIGURE 4A. X-RAY DIFFRACTOMETER TRACING OF C2 FINES FROM FIELD COMPACTION DEGRADATION.

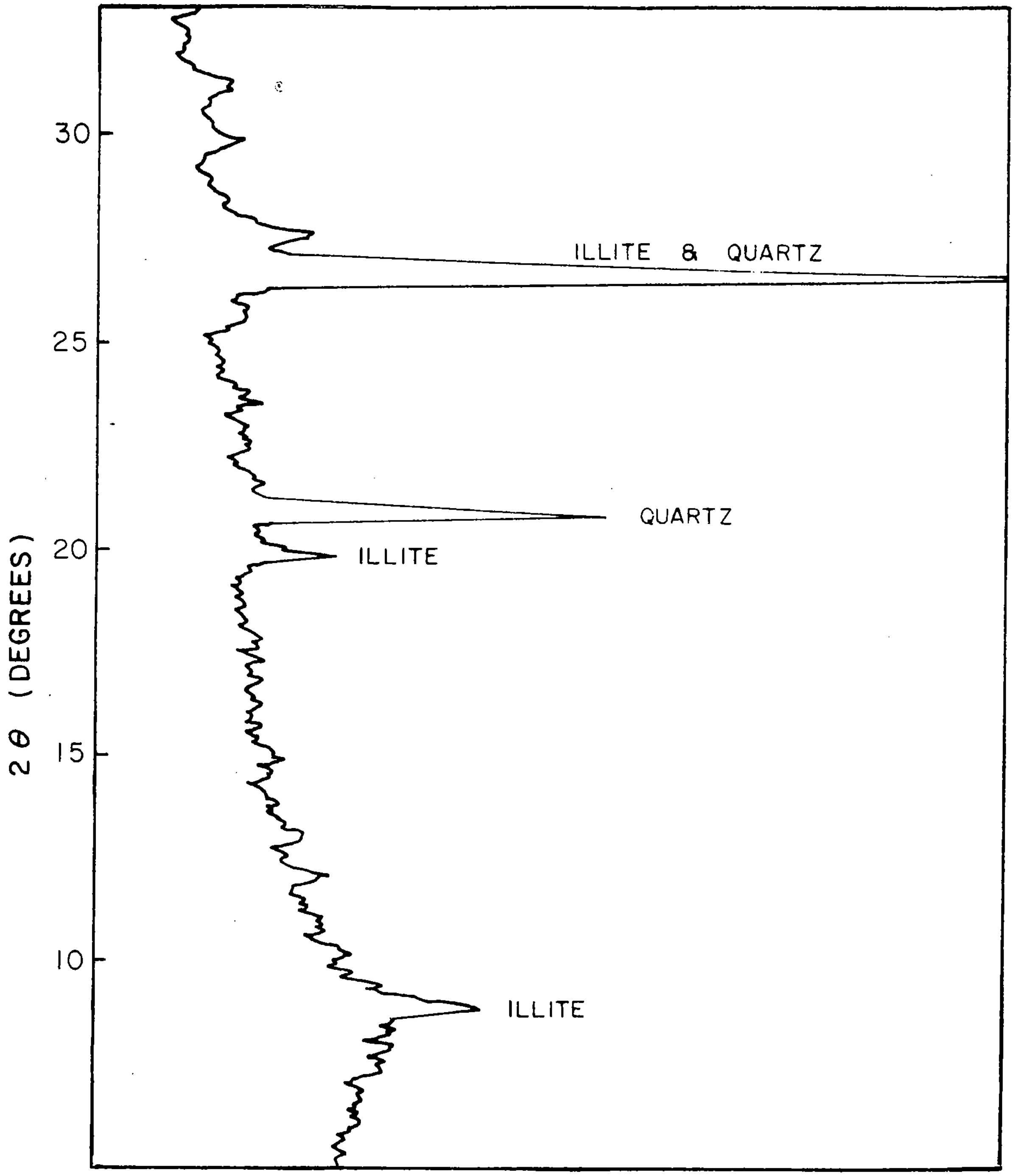

FIGURE 4B X-RAY DIFFRACTOMETER TRACING OF CLAY SIZE INSOLUBLE RESIDUE FROM C2.

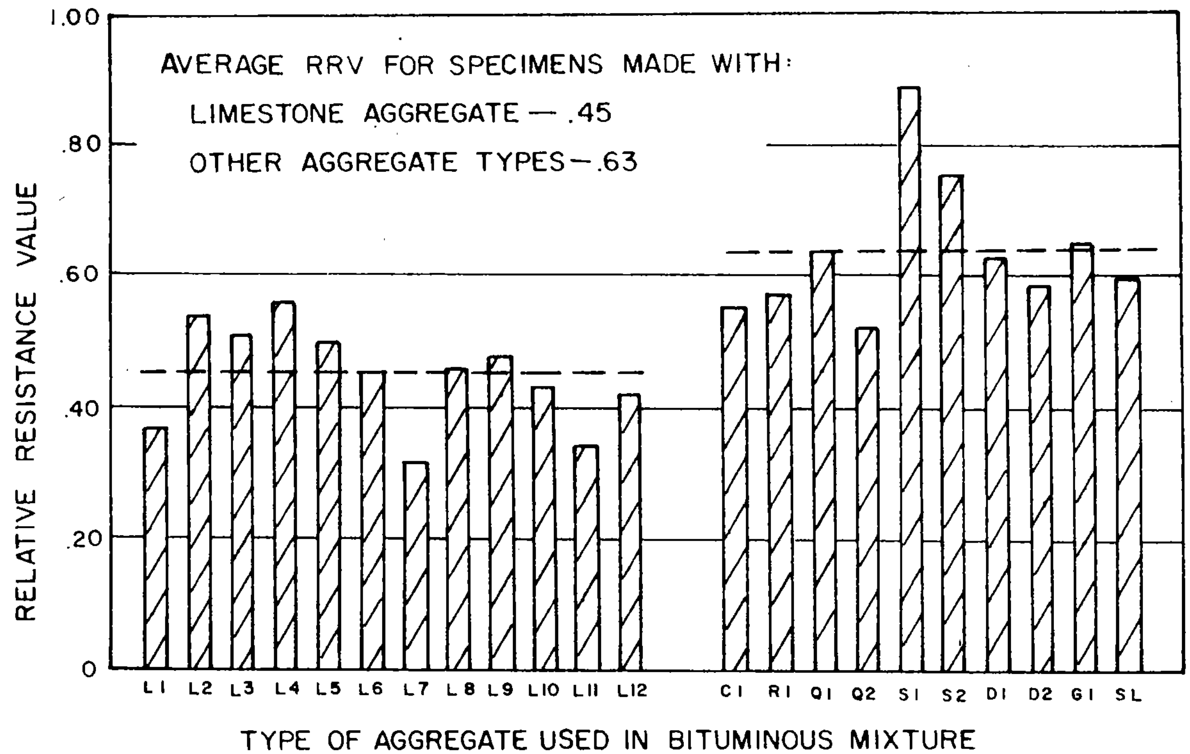

FIGURE 5. SKIDDING RESISTANCE OF BITUMINOUS MIXTURES AT THE COMPLETION OF THE FINE POLISH CYCLE.

THE ROLE OF AGGREGATE DEGRADATION IN HIGHWAY CONSTRUCTION

by

T. R. West and N. B. Aughenbaugh

INTRODUCTION

Aggregate degradation studies are currently under way in a number of state highway departments, universities, and governmental agencies in this country and elsewhere in the world. The widespread interest in aggregate breakdown is of fairly recent origin although the phenomenon has been recognized and studied to a limited extent for many years.

Aggregate degradation is by nature a large subject because aggregates are used in many different ways as an engineering material. Some degradation investigations are concerned only with base and subbase materials, some deal with bituminous mixes and surface courses, and still other studies pertain to concrete aggregate uses. Since the various studies originated from different problems of aggregate use, it is often difficult to relate the results to a common base. Consequently, a moderate amount of information has been obtained on the subject, but it has not been fully correlated or digested.

Several factors account for the recent increase in aggregate breakdown studies. Modern highway and airfield designs require greater densities in the aggregate masses. To achieve this end, supercompactors have been introduced which create greater stresses in the aggregate during the construction phase. In-service demands on modern pavements also are more stringent, with high speed travel and increased traffic volume requiring better pavement performance over longer periods of time. To insure adequate performance, the original aggregate gradation must be maintained throughout the life of the pavement.

It is the intent of this paper to review briefly the literature on aggregate degradation, to define the problem as it currently exists, and to discuss the present and possible future paths of research.

HISTORY

Aggregate degradation was first recognized near the turn of this century when it was noted that horses hoofs and buggy wheels caused significant break-down of certain surface roadstones. The first detailed studies of aggregate properties and performance were made at this time by E.C.E. Lord (19) and F.H. Jackson (15) whose results and evaluations still generally apply today.

The next important period of activity in aggregate studies was in the mid 1930's when investigators attempted to reproduce field conditions in the laboratory by model rollers and roadways. The designs were many and varied. Goldbeck et al (12) used a circular track apparatus, whereas, Tremper (31) designed a movable rectangular box with a stationary roller to simulate actual conditions. These studies notably advanced the knowledge on aggregate and pavement interactions, but because of the bulky and time-consuming nature of the equipment, they could not be adapted to routine aggregate evaluation. It was during this same period that the Los Angeles abrasion test was developed and investigated as a practical laboratory test for aggregate quality (26,33).

The effects of aggregate degradation in bituminous mixtures were investigated by Macnaughton (20) in 1936 by laboratory compaction tests and by observation of the in-service performance of pavements. Although he considered many factors such as amount and type of compaction, gradation, aggregate shape, amount and type of bituminous binder and position in the pavement (traffic line or near the edge), he did not take into account the effects of different aggregate types on the breakdown.

One of the first field studies on aggregate degradation was made in the late 1930's by Shelburne (27). He investigated the crushing effect of steel wheel rollers on the aggregate particles for surface treatments and compared the field aggregate performance with the Los Angeles abrasion test results.

In the early 1940's two studies were made at the University of Witwatersrand, South Africa, on the road making properties of aggregates in which degradation was a main consideration (7 and 18). B.H. Knight, (17) who supervised these studies, later conducted a three-year field and laboratory investigation on the factors that affect aggregate breakdown in asphalt mixes. In his book "Road Aggregates," Dr. Knight notes the problem of aggregate degradation and emphasizes its importance to highway engineers (16).

The British Road Research Laboratory has been an active agency in aggregate research and has considered degradation in its studies. In 1948 Shergold (28) made a literature search on the evaluation of physical tests to predict the in-service behavior of road building aggregates. Degradation was one of the factors considered. He used statistical analysis to rate the reliablity of physical tests to performance, quality, and aggregate type. The Laboratory later conducted a three-year field investigation of degradation in surface-dressings (29). Some of the variables considered were: a) weight of roller, b) number of roller passes, c) rock type, d) grading, e) particle shape, and f) binder effect.

The late 1940's and early 1950's was a period of increasing aggregate quality investigations, however, only a few studies considered any aspects of aggregate breakdown. Among the more notable of these were Melville (21), Metcalf and Goetz (22), and Philippe (25). Melville investigated the possibility of devising a special laboratory test that would indicate the susceptibility of aggregates to in-service degradation. Failures of several surface-treated, waterbound macadam pavements in Virginia after a few years' service led to the study. Metcalf and Goetz made a special study to see if sandstone could be used as an aggregate in bituminous mixes in regions where other desirable aggregates are lacking. Degradation of the sandstone was one of the main factors considered. Philippe investigated nine different types of laboratory compaction apparatus to see how well they reproduced the amount of aggregate breakdown and the degree of density as found in the field.

The northwestern states should be given a great deal of credit for initiating the present stimulus for aggregate degradation research. Washington State in the mid 1950's experienced considerable trouble with new highways showing undue amounts of distress owing to obvious aggregate degradation. An investigation was made by Turner and Wilson (32) to determine the cause of the aggregate failure and to develop a satisfactory routine laboratory test capable of identifying unsatisfactory materials. This investigation was the first of many recent studies conducted in the states of Washington, Idaho, and Oregon on the problem. Some of the results and information are summarized by Sibley (30), Erickson (9), Eske and Morris (11), Minor (23), Collett et al (5), Day (8), and Harra (14). The results of these investigations have established that chemical breakdown is the prime degradation factor in this area.

Degradation research in other parts of the country has proven physical aggregate breakdown to be an equally important phenomenon. Two studies recently completed at Purdue University demonstrated that physical breakdown of aggregates does occur in significant amounts and can cause serious detrimental changes in design gradations. Moavenzadeh and Goetz (24) studied the effects of gradation in bituminous mixes using a gyratory compactor. Aughenbaugh et al (3,4) made a three-year field and laboratory study on construction compaction breakdown of aggregates and the effects of the fines on the frost susceptibility of the resulting base coarse material. A current investigation is being made at Purdue University for the National Cooperative Highway Research Program (NCHRP). This study is concerned with the regional aspects of degradation in base and subbase material.

Other investigations on the subject are known to be currently in progress. The Bureau of Public Roads has indicated in a personal communication that 14 states presently are conducting research on the subject. Questionnaires distributed in the past (1, 10, and 34) also have disclosed varying amounts of activity by many agencies on the subject.

THE DEGRADATION PROBLEM

The term "degradation" means a reduction of rank or the wearing down of a substance. When applied to aggregates, degradation has the same general meaning, although some investigators have defined it more rigidly. The broadest definition of "aggregate degradation" is the breakdown of aggregate pieces into smaller particles by chemical and physical processes. This is the definition used in this paper so that all aspects of the phenomenon can be considered.

The physical and chemical processes that cause the breakdown can act singly or in combination. Aggregate degradation generally results from the interaction of both processes, but one usually plays a much more dominant role. Investigators who have dealt with problems where only one has predominated, have erroneously considered all degradation the end product of that process. This explains why several different definitions have been proposed for aggregate degradation.

Aggregate degradation is not always detrimental, and the general definition as used in this paper does not imply this. Several states rely on limited aggregate breakdown during construction of base courses to yield the desired density. At the present, detrimental considerations far outweigh any beneficial ones, but future investigators may study what valuable aspects degradation offers.

Degradation is detrimental if it alters adversely the design of function of a road or structure. Some of the harmful effects that degradation can create are:

1. Raveling and instability in bituminous mixes
2. Loss of base course support
3. Excessive and differential settlements
4. Reduction of drainage
5. Creation of frost susceptible material
6. Distress in Portland cement concretes

The harmful effects of aggregate breakdown were first noticed in bituminous surface courses. The change of the design gradation by degradation caused an increase in aggregate surface area which resulted in raveling and instability of the affected pavement. As a result, much of the early research conducted on degradation was done by investigators in the field of bituminous pavement design. The problem has never been eliminated, but engineers in this field are aware that the aggregate mass does undergo some crushing by the compaction roller, causing an increase in the amount of surface area the bitumin must cover.

Investigators in the northwestern states of Washington, Oregon, and Idaho have found degradation of certain aggregate types create excessive amounts of plastic fines in base courses and overlays. The resulting plastic material causes a loss of pavement support which leads to surface cracking and break-up in the bituminous surface coarse.

Degradation allows a further densification to occur in the aggregate mass in relation to its original density state. Continued aggregate break-down and densification after the aggregate has been placed into use can cause differential settlements to occur. This detrimental effect has not been positively identified as the principal cause of any highway settlements because other phenomena can produce the same end result and investigators have not considered it. However, loss of grade due to degradation and sub-sequent densification has been documented for railroad ballast (13).

The permeability of a base course can be appreciably altered by aggregate breakdown. Some coarse, granular, freely-drained base courses develop pump-ing after a few years' service, and this pumping is not due to subgrade in-trusion in all cases, but rather to the development of fines and a reduction of drainage in the course itself. A reduction of drainage also can cause a reduction of strength in the base course.

In regions of frost action,degradation can produce a frost susceptible base. Only a small amount of silt-size fines need to be created to render a base course frost susceptible. The Corps of Engineers have encountered this problem in some of their jet runways (4).

Aggregates for Portland cement concrete are known to degrade in the mixer, and the breakdown is similar to that produced by the tumbling and ball mill action of the Los Angeles abrasion machine. The increase in aggregate surface area can cause a reduction in strength, a decrease in workability, and a change in the concrete pore size and interconnection. An interesting thought recently raised by Corlett and DeGast (5), is the possibility that aggregates could continue to degrade even after the concrete has hardened, resulting from stress relaxation within the rock pieces. This mechanism would expose more surface area of the aggregates and perhaps would hasten alkali aggregate reaction.

It is possible that aggregate degradation can be detrimental in other ways which have not been noted in this paper. With the interest in aggregate quality increasing, such harmful effects should not remain unknown for long.

RESULTS OF PAST RESEARCH

More emphasis has been placed on developing tests to indicate the quality of aggregates than on defining the mechanics of degradation. The early researchers concentrated almost entirely on developing laboratory tests, but more recently investigators have included in their studies the definition and analyzation of factors that influence aggregate breakdown. With an understanding of the mechanics of degradation, they feel it is possible to develop more significant standard tests.

Aggregate degradation tests should serve a definite purpose. The test results should reflect those properties of the aggregates which are affected most under field conditions. The ideal tests would approximate or simulate actual field conditions, but this is difficult to achieve because of the size, complexity, and time limitations that are imposed in obtaining a practical test. Most existing tests are empirical because they do not satisfy the above criteria. These empirical tests can be and are useful if their results can be correlated to field performance, and if they are simple, fast, and inexpensive operations. This explains why there are so many different tests being used and why these tests give satisfactory results in some areas and unsatisfactory results in others. Owing to the type of breakdown and the factors involved in a region, the empirical tests differ in their degree of correlation. As far as it is known, no one physical test adequately evaluates the susceptibility of an aggregate to degrade under all conditions. Some states have developed a set of tests which they have found reliable in revealing unsatisfactory aggregates with respect to degradation.

The tumbler-type tests have been the most successful and widely used of all tests. The Los Angeles abrasion test is the most common and best known of all the tumbler tests, but it has been unreliable as a degradation test in many cases. The "Standard" or usual routine laboratory tests have been very unreliable although they are useful at times for trends and for distinguishing very undesirable aggregate types. Among the tests that fall in this catagory are the sulfate soundness, absorption, toughness, scratch hardness, and specific gravity tests. Many existing compression and dynamic tests have been tried as routine degradation tests, but they have given poor results to date. Petrographic analysis, if it can be considered a routine laboratory test, seems to offer the best approach in establishing a partical and standard universal test for aggregate degradation.

The factors that influence aggregate degradation are many, interrelated, and vary in their importance. Mechanical degradation will be influenced by a different set of factors than chemical degradation. All the factors that influence degradation are not known and of those that have been recognized and studied, only a few have been shown to govern degradation in most cases.

Listed below are the factors that have been found to influence aggregate degradation.

1. Aggregate type
2. Compactive effort
3. Gradation
4. Aggregate size
5. Aggregate shape
6. Time
7. Water
8. Weathering and secondary minerals
9. Subgrade and/or subbase
10. Aggregate structures
11. Particles orientation
12. Thickness of layer
13. Climate
14. Aggregate use

All investigators agree that aggregate type and compactive effort are major factors, but they disagree on the influence of gradation and aggregate size and shape. The remaining factors have not been studied by many investigators and hence less is known about them for all situations.

SUMMARY

The harmful effects of aggregate degradation may take many different forms. Aggregate breakdown should be considered detrimental if in any way it adversely affects the design or function of a highway. The undesirable effects resulting from the breakdown may occur in the construction phase or after the road has been placed in service. The degradation can occur during stock piling, transportation, placement, compaction, or after the aggregate mass has been put into service.

It is often difficult to recognize if degradation has occurred. For one reason, it is hard to control the blending within certain limits and, at times, wide variances can occur in the resulting gradation. Pit-run gravels have the poorest control of all aggregate blends, and any suspected degradation occurrences in them should be viewed with caution. One-size and open-type gradations of crushed aggregates offer the best control for

for degradation evaluations. The more dense-type gradings with their multiple particle sizes suffer segregation effects and make it more difficult to define, by normal sampling procedures, the amount and type of aggregate breakdown. For example, in a stockpile, a natural segregation will occur so that random samples across the pile will give different gradations, although the total gradation of the pile is intact. Any handling will cause segregation. In one investigation (4), it was noted that significant segragation occurred in dense-graded mixtures even though the material was hand placed. If the segregation and blending variations are so large that they exceed the degradational changes, aggregate breakdown cannot be positively identified even though it may occur in significant amounts.

In addition to the factors that influence or cause the actual particle breakdown, other aspects must be considered in determining what constitutes detrimental degradation. The aggregate use dictates the degree and kind of degradation to be tolerated. As noted earlier in the paper, the harmful effects of aggregate breakdown are many and varied. An aggregate may undergo the same amount of degradation in two different uses, but the degradation would be detrimental for only one. What constitutes an excessive amount of breakdown in bituminous mixes may not be excessive for base courses.

Intimately associated with aggregate use are the tolerances that are allowed of the aggregate performance. Many states have placed different tolerances on the same aggregate use. These tolerances depend mostly on the amount of high grade aggregate available in the state. Also, within a single state, different tolerances are allowed for the same type of construction use. Aggregates used in base courses for interstate highways must be of higher quality than those used in second class or country roads. Economics dictates this apparent conflict. In considering any engineering project, the money available will influence the final evaluation for the tolerances to be allowed in aggregate quality.

Regional setting is a final and important aspect of the problem. Varied climates, rock suites, and construction practices in the different geographic regions result in degradation problems unique to those areas. Those states south of the frost line need not worry about degradation producing frost susceptible material. Likewise, states in semi-arid regions are less concerned with problems created by excess fines in continued association of water, such as pumping and loss of support due to plastic fines. Other examples could be presented to emphasize the importance of the regional aspects to detrimental degradation. The regional concept has clouded the broad considerations of aggregate degradation more than anything else, primarily because investigators studying problems unique to their area tend to believe that no other degradation problems can exist.

After considering the many ways aggregate degradation can cause distress, the different uses and tolerances imposed upon the aggregate, and the influence of regional aspects, it is easy to understand why no one laboratory test successfully evaluates aggregate degradation for all cases. Good routine laboratory tests can and should be developed on a regional basis where conditions are similar. Petrographic analysis offers a possible way to integrate inter-regional degradation considerations.

In summary, aggregate degradation is a many-phased phenomenon which has been recognized since the turn of the century but has received widespread attention only in recent years. Both physical and chemical processes are involved in the breakdown, but usually one dominates over the other. The factors that control the breakdown are many and vary in their influence, depending upon the dominate process involved. Actually, there is an interdependence among the processes and factors. The degradation of aggregates can cause different harmful effects depending upon the aggregate use and the regional aspects, therefore aggregates should be evaluated for their intended use. This limits routine laboratory testing except on a regional basis. As additional information is accumulated on the mechanics of aggregate breakdown from more geographic locations, better use of aggregates can be made in highway design.

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CLAY MINERALOGY AND SOIL STABILIZATION

by

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INTRODUCTION

The fact that colloid chemistry, mineralogy, and soil technology are inseparably linked and that the properties of clay soils are governed to an important degree by the rules of colloid chemistry has been accepted reluctantly by the practicing engineer. As early as 1935 Hans F. Winterkorn was emphasizing the need for a fuller understanding of physical and colloidal chemistry for investigators in the area of soil mechanics. Interest mounted slowly and only a few investigators began to apply these sciences to soil systems. A few papers began to appear in various publications on clay technology and its application to soil mechanics during the following years. Finally, in the April 1959 issue of the Journal of Soil Mechanics and Foundations of A.S.C.E., a collection of papers on the "Physico-Chemical Properties of Soils" was published, and a marked change in the acceptance level of this important subject was seen to occur across the country. Certainly it became evident that stabilization of soils or at least important modification of soils could be accomplished by moderate and economical additions of various chemicals. In this paper, the nature of the clay mineral and surface energy concepts will be presented with emphasis on their relation to the physical properties of the bulk system.

NATURE OF THE CLAY COLLOID

Fundamentally there is nothing very special about the interaction of a clay particle and water when compared with interactions of other inorganic materials with water. There is very little difference in the orders of magnitude of adsorption energy on clays or other inorganics nor are there larger differences in charge density. The behavior of the clay materials are quite different, however, in an engineering sense because of the unusual shape of the clay platelet and the dual character of its surfaces.

When we speak of clay having dual surface character, we see two particular features, one is the platy nature of the basal or upper and lower faces and the very thin fractured edge which gives the clay at least one colloidal dimension. Secondly, we note that the fractured edges have exposed unsatisfied positive ions, corner exposed positive and negative ions, and basal exposed negative ions and that the pyropholite and kaolinite crystal taken as a unit has a balanced internal change. The montmorillonite structure has an internal charge unbalance. Thus the edges are the seat of residual unsatisfied bonds, and the basal surface ions are strained due to eccentricity of stress, analogous to surface tension in water, and the basal surfaces are the site of super-structural ions necessary to balance the shortage of charge within the clay crystal lattice. The overall result of this appears to be that an attraction for ions from the liquid phase is exhibited by the clay crystal. This demand or attraction can be expressed as a surface potential or a surface energy condition. To achieve a smooth transition from a high potential state to a potential equivalent to that of the electrolyte solution, ions of opposite charge to that of the clay crystal are adsorbed from the electrolyte and these in turn attract ions of an opposite sign. The overall result of this attraction is the formation of a complex double layer of absorbed ions through which the potential energy of the surface is dissipated to an amount equal to the potential of its atmosphere. If an electric current is impressed on a suspension of clay having a double layer of absorbed ions a portion of the adsorbed ions are stripped off and will move in the direction of the anode or cathode, the clay with the remaining adsorbed ions will move to the opposite pole. In the case of clays, the particle usually moves toward the anode and the more loosely held diffuse layer ions move toward the cathode. This infers that the clay crystal and its more tightly held ions have a preponderance of negative charge and the diffuse layer ions are predominantly positive in charge. From this concept, an atmospheric distribution of ions as conceived by Gouy can be shown, Figure 2. Through this atmosphere of ions the surface potential of the clay crystal is dissipated exponentially until it has reached the value of the potential of the atmosphere in which it exists. One method of expressing this potential magnitude is by the so-called Zeta Potential equation. This is shown in Figure 2B as the potential drop in the diffuse layer of the double layer system and is expressed by the equation shown.

SURFACE ENERGY (ZETA POTENTIAL) AND ITS EFFECT ON SOIL PROPERTIES

The magnitude of the Zeta Potential is of considerable importance to the soils engineer since it is the determining factor in the type of structure the clay particles will have. From Figure 3 it is seen that the higher the potential, the greater the repulsion forces between particles and the more dispersed the system of clay particles will tend to be. When the energy on the particles is high and they are forced to approach each other, they will tend to orient themselves in a minimum energy position as shown in Figure 4. When the Zeta Potential is low the particles can approach each other much closer and due to the different surface characteristics the flocculated structure, edge, to face orientation can occur. Enumerated below each of the proposed structural arrangements in Figure 4 is a list of the relative physical properties that can be associated with the structures shown and the energy associated with them. Immediately it is evident that modifications in soil properties can be made if the Zeta Potential can be adjusted.

MODIFICATION OF THE ZETA POTENTIAL

In Figure 2 the equation for the Zeta Potential is given as $Z = \frac{4\pi Qd}{AK}$ where Q = charge quantity (e.s.u.), d = diffuse layer thickness, and K is the dialectric constant of the diffuse layer, and A is the surface area. The values of A, K, and Q are difficult to change in sufficient magnitude to materially change the Zeta Potential while the value of d, the thickness of the diffuse layer may be manipulated to a considerable degree.

From the equation it is evident that an increase in d increases the Zeta Potential and the repulsion forces, and a decrease in d decreases the same factors. Changes in the d value will occur if: (1) the concentration of electrolyte in the pore water is increased, and the ions of like charge repel each other, thereby packing the ions in the diffuse layer and decreasing d. Inversely lowering the electrolyte concentration of the pore water will allow the d distance to increase, (2) Ions of higher valence may be substituted for lower valence ions in the diffuse layer. The higher valence ions can satisfy the required electrical balance with fewer numbers and in less space than the highly hydrated lower valence cations, thus decreasing the diffuse layer thickness.

CHANGE IN SOIL PROPERTIES

From the above it is evident that small additions of the proper chemical can have rather large effects on the Zeta Potential and the properties of the clay soil. Every day usage is made of this knowledge in evaluating soil test data and in performing soil tests. When we want to disperse a soil for a grain size analysis we automatically reach for the box of calgon (sodium hexametaphosphate) or the jar of sodium silicate. These additives increase the Zeta Potential by replacing diffuse layer ions with the large highly hydrated sodium ions, thereby increasing the d distance and the anion added can complex with other cations in the electrolyte to decrease the concentration repulsion forces, thus allowing further d distance expansion. If it is desirable to decrease the plasticity of a clay soil decrease in zeta may be accomplished by flocculants. The Clarksville soil in this area of the Ozarks has a plasticity index of around 13 to 16 percent which can be reduced to zero by the addition of hydrated lime. The shear characteristics are changed materially by the same additive.

In natural soil systems, the in-place properties may change upon leaching and electrolyte concentration changes, due to changes in repulsion or attraction. Some of the stable slopes of many years may consistently lose strength due to change in electrolyte concentration, and suddenly fail. Some of the quick clay flow phenomena of Canada and Norway may be from the same causes.

If changes in the potential can be caused by natural leaching action then the permanence of stabilizing soils by Zeta Potential changes might be questioned. Fortunately, however, chemical additives can perform other stabilizing functions in a soil besides the Zeta Potential changes.

TABLE I

Effect of Chemical Additives as Stabilizing Agents

(1) Change the energy characteristics of the fine grained soils.

(2) React with water or gas to form insoluble precipitate binders.

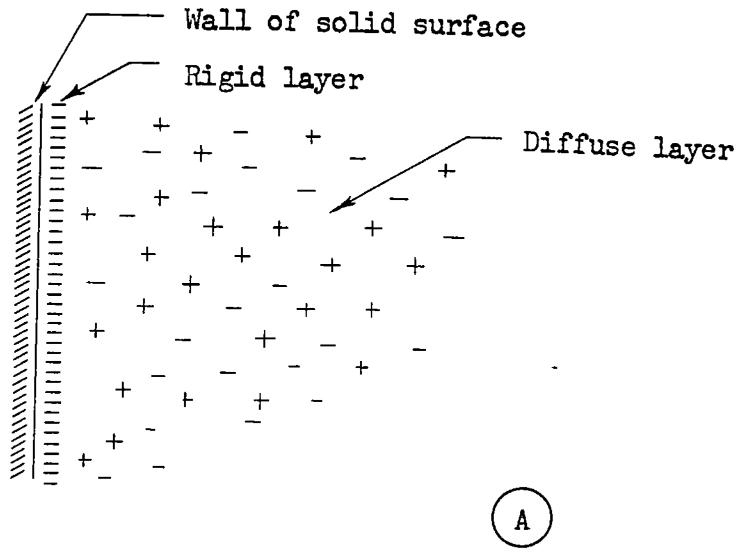
(3) React with naturally occurring soil constituents to form insoluble binders.

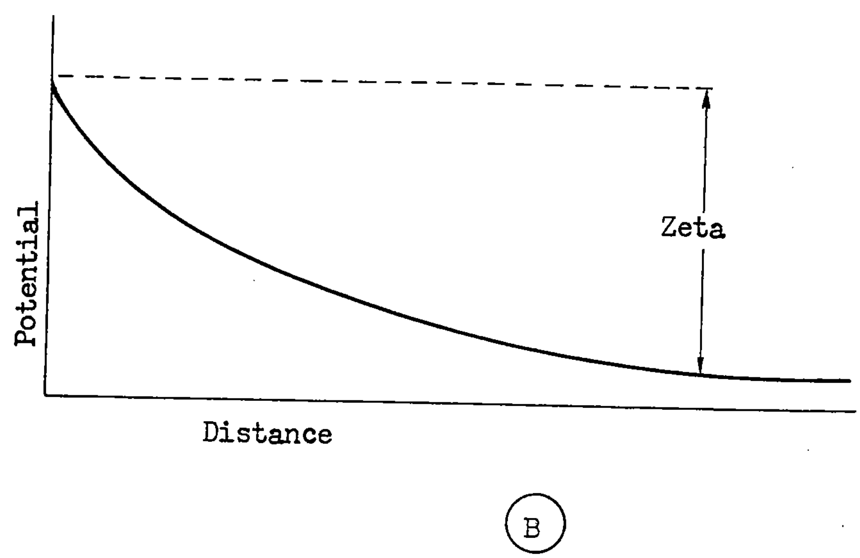
(4) React with a second additive to form insoluble binders.

(5) Waterproofing

There is, therefore, a possibility that changing the character of the soil through surface energy changes can be a permanent nature due to the reactions which may accompany it. An example of this is the reaction of lime with clay soils which will affect surface energy, may form insoluble binders by reacting with silica or aluminum ions or carbon dioxide present in the soil. If doubt still remains concerning the lasting nature of the surface energy changes, additional additives such as cement or asphalt may be used in combination with the chemical additive.

Understanding the mineralogical-colloid-surface energy concept gives an insight into problems and questions concerning soil properties which is invaluable to the soils engineer. The ability to see the cause of problems associated with clay soils is enhanced and the liklihood of finding a means of solving the problem is even more probable. Work in soil stabilization is continuing at a rapid pace with the testing and evaluation of many different additives that show promise of some success. There are chemicals, both organic and inorganic, which need to be evaluated and also various combinations of additives which together may perform quite well. Work is still necessary in all areas and with all soils but the fundamental approach seems to be in.





Gouy-Chapman diffuse double layer

$$\text{Zeta} = \frac{4\pi Qd}{AK}$$

Figure 2

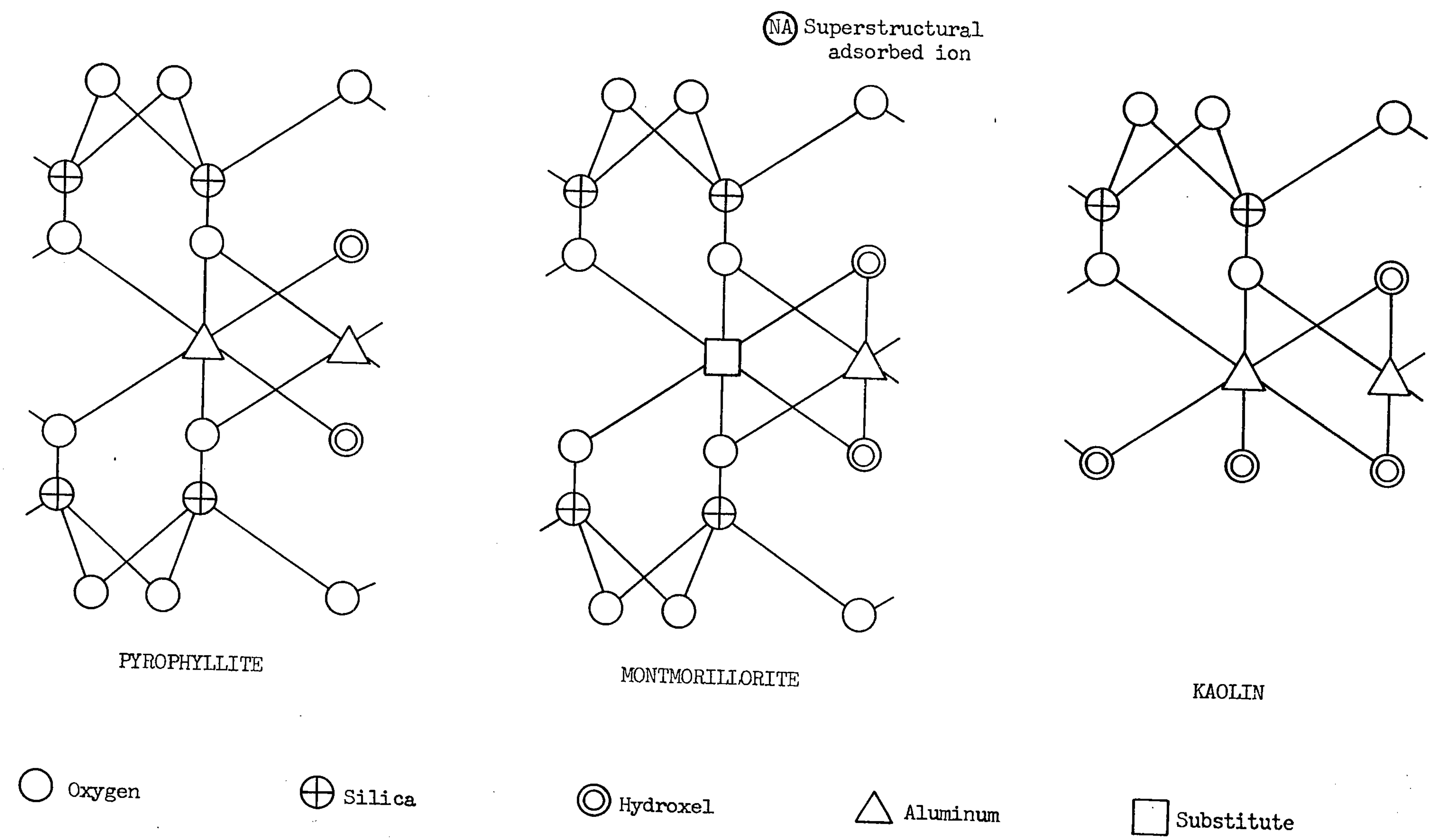
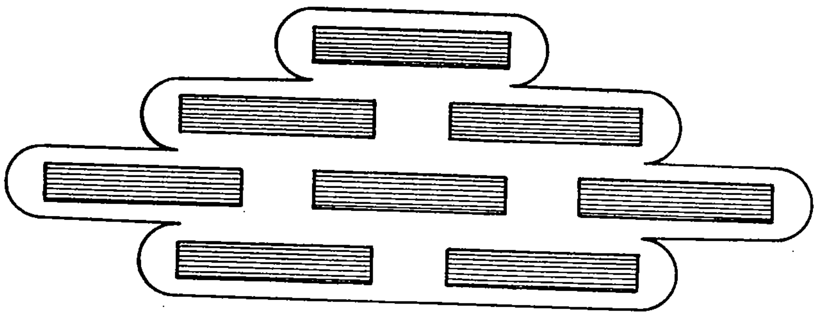
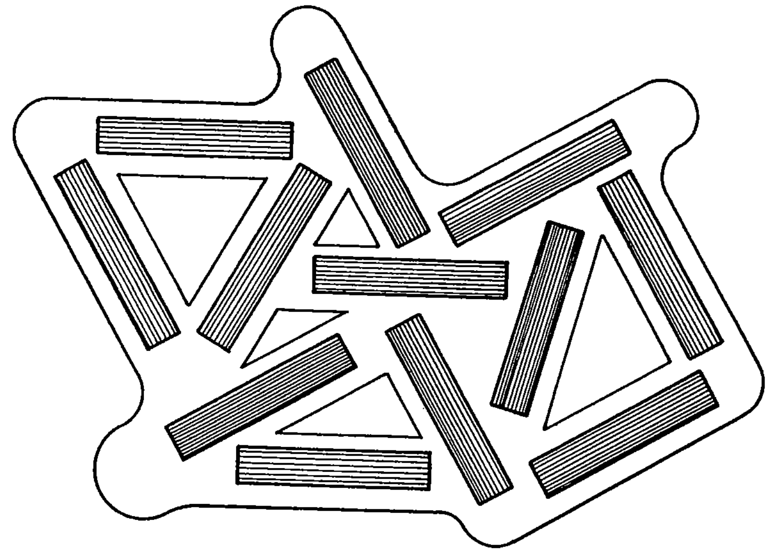

NA Superstructural adsorbed ion
PYROPHYLLITE
MONTMORILLORITE
KAOLIN
Oxygen
Silica
Hydroxel
Aluminum
Substitute

Figure 1



DISPERSED STRUCTURE



FLOCCULATED STRUCTURE

RELATIVE PROPERTIES OF THE STRUCTURES SHOWN

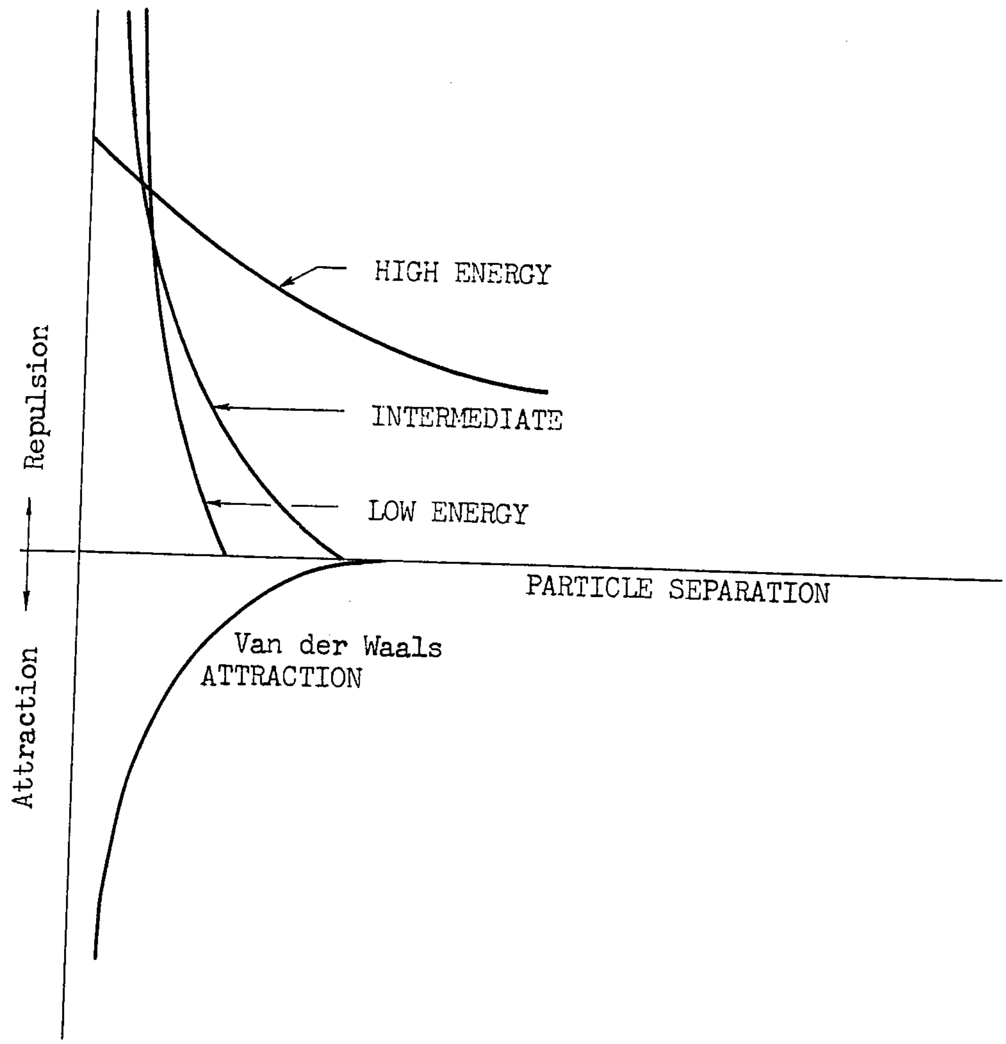
DISPERSED (HIGH ENERGY)

1. High Plastic Index
2. Low Shrinkage Limit
3. High Compacted Density
4. Low Angle of Friction
5. Low Permeability
6. Poor Granularity

FLOCCULATED (LOW ENERGY)

1. Low Plastic Index
2. High Shrinkage Limit
3. Low Compacted Density
4. High Angle of Friction
5. High Permeability
6. Good Friability

Figure 4



Repulsive and Attractive Energy as a Function of Particle Separation at High, Intermediate and Low Zeta Potential

Figure 3