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HIGHWAY GEOLOGY SYMPOSIUM

The University of Oklahoma

Norman, Oklahoma

April 22-23, 1971

CO-EDITORS

Rosemary Kellner and William D. Rose

Oklahoma Geological Survey

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Oklahoma Geological Survey

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PREFACE

On behalf of the National Steering Committee and the sponsors of the 22nd Annual Highway Geology Symposium, we express our appreciation to everyone who helped make this symposium a success.

The symposium was held in Norman at the Oklahoma Center for Continuing Education on The University of Oklahoma campus April 22-23, 1971. Cosponsored by the Oklahoma State Highway Department and the Oklahoma Geological Survey, the conference began with a field trip through the Arbuckle Mountains and Ardmore area of southern Oklahoma. The annual banquet--featuring Newell J. Trask of the Branch of Astrogeologic Studies of the U.S. Geological Survey, Menlo Park, California--and the technical session rounded out the program.

Special thanks are due Kenneth S. Johnson, Oklahoma Geological Survey, and Willard McCasland, Oklahoma Highway Department, for organizing the field trip. Robert Fay, L. R. Wilson, Gerald J. Petzel, Curtis Hayes, and Stuart Ronald assisted with the trip. A copy of the symposium's official field-trip guidebook, <u>Highway Geology in the Arbuckle Mountains and Ardmore Area</u>, <u>Southern Oklahoma</u>, can be obtained through the Oklahoma Geological Survey for a distribution charge of 50 cents.

The technical sessions consisted of the presentation of papers by engineers and engineering geologists representing universities, consulting firms, and state and federal agencies. Covering a wide range of subjects and dealing with problems encountered not only in the United States but also in Africa and South America, these papers are here presented in their entirety--with one exception: only the abstract of Arthur Cleaves' "The Camino Marginal, Peru" is given, because the paper has been presented as evidence in a court case and is therefore unavailable at this time.

We particularly want to thank Rosemary Kellner and Bill Rose for the pains they took in editing these manuscripts.

It is our hope that the papers will provide useful information and intellectual stimulation for our readers.

Mitchell D. Smith, Physical Science Engineer
Oklahoma Department of Highways

Charles J. Mankin, Director
Oklahoma Geological Survey

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WELCOMING ADDRESS FOR THE
22nd ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Chester Brooks, Director
Oklahoma Department of Highways
Oklahoma City, Oklahoma

I appreciate the opportunity to squeeze in on your program, for I would not want to miss the chance to welcome you to the Sooner State and to call attention to the importance of your contribution to highway progress and the significance of this meeting. We are sincerely grateful and indeed quite honored that you chose to come to Oklahoma for this most important 22nd Annual Highway Geology Symposium. It is our hope that you will not only benefit professionally but also enjoy yourselves. We trust that you will look back on your visit to our State with fondness and will want to return. It would be an affront to us if there were something we could do to enrich your time with us and you did not call upon us. We invite you to linger after the close of the symposium and get to know us better. Certainly, we would be pleased to have you visit our highway department.

There was a time in our road-building history when if you asked a man what he needed in order to build a good road his answer might well have been "a strong mule, a good fresno, and a direction to go." If that were all it took today, our jobs might be a lot better--but our highways wouldn't. No one knows better than you gentlemen the complexity of road building today. Our highways are the product of a specialized team that includes the engineer, the planner, the agronomist, the administrator, the environmentalist, and others.

But no role is more significant than that of the highway geologist. If you do not give us the best that is in you, there is little opportunity for lasting success. It is you who must make peace with Mother Earth for our transgressions against her. We depend upon you to teach us about her basic nature, her temperament and her moods. We will never again build our highways with just a mule and a fresno, and the science of highway building will become more complex, not more simplified. We need your talents, your energies, and your knowledge more than we needed them yesterday but less than we will need them tomorrow. The highway geologist is indispensable in the scheme of things to come.

It is well to look back over our shoulder to see how far we have come and what good works we have accomplished. It is truly nourishing to measure the distance between that mule of yesterday and the technical nature of today's 22nd Annual Highway Geology Symposium. It is also mandatory that we look forward to the challenges of tomorrow. The highway team can expect the critical eye of the user to scrutinize products and methods even more closely than in the past. We are entering a new era of public concern, a more vocal public opinion, and an intensified public involvement in our transportation progress.

Public demands for higher quality must be met by greater knowledge and better ideas as well as harder work. Our job will become more complex and will require of each of us who make up the road-building team increased understanding and appreciation for the talents and contributions of others.

We must put aside our differences and our jealousies; the task ahead of us has no room for interservice rivalries. We will build together or we will probably not build at all. We must provide energetic and dynamic leadership in transportation development.

I wish you a successful and fruitful meeting.

GEOLOGY OF OKLAHOMA--A SUMMARY

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Oklahoma Geological Survey
Norman, Oklahoma

Abstract.--Oklahoma's geology ranges from folded and faulted mountain systems in the south to horizontal and gently dipping strata forming broad rolling plains and cuestas in the central and northern areas. Each of the major geologic provinces is distinctive in terms of its suite of rocks, its geologic history, and its present geomorphology. Sedimentary rocks are at the surface in almost all parts of the State (shale, sandstone, and limestone being most abundant), but igneous rocks and mildly metamorphosed sediments make up a part of each mountain system.

Oklahoma is a region of complex geology where a mobile belt of Paleozoic geosynclines and uplifts on the south abuts against the margin of the North American craton to the north. The State contains many classic areas where fundamental concepts of sedimentation, stratigraphy, structural geology, and historical geology have been formulated through the years. In the southern Oklahoma mountain belts a great variety of igneous and sedimentary rock units rarely seen elsewhere in the Midcontinent area are exposed.

Major tectonic elements of Oklahoma (fig. 1) include (1) the cratonic and relatively stable central and northern areas, including the Ozark uplift, (2) the Ouachita geosyncline and associated Arkoma basin in the southeast, and (3) the Southern Oklahoma geosyncline, comprising the Anadarko, Ardmore, Marietta, and Hollis basins as well as the Arbuckle and Wichita Mountains uplifts. The three principal fold belts, the Ouachitas, Arbuckles, and Wichitas, all originated from a series of Pennsylvanian orogenies (about 300 million years ago) in the two Paleozoic geosynclines.

Most of the outcropping rocks in Oklahoma are of sedimentary origin, and most of these sediments are of Paleozoic age (fig. 2, map). The thickness of Paleozoic sediments ranges from 2,000 to 10,000 feet in cratonic shelf areas of the north and from 30,000 to 40,000 feet in deep basins of the south (fig. 2, cross section). Sedimentary rocks overlie a basement of Precambrian to Middle Cambrian igneous rocks and mildly metamorphosed sediments. Limestone and dolomite make up most of the Upper Cambrian to Lower Mississippian strata and attest the early and middle Paleozoic crustal stability in most of Oklahoma prior to the Pennsylvanian episodes of mountain building. Thick units of shale and sandstone predominate in the Upper Mississippian and Pennsylvanian sequence. Permian sediments are characterized by red shale and sandstone with interbedded gypsum and salt. Mesozoic and Cenozoic deposits are mostly thin units of conglomerate, sandstone, and shale. The accompanying general stratigraphic succession (fig. 3) shows the commonly used formation and group names where feasible, and standard series names where nomenclature is profuse.

Each of the major geologic provinces (fig. 1) in the State is distinctive in terms of its suite of rocks, its geologic and tectonic history, and its present geomorphology. The types and severity of highway-engineering problems resulting from this distinctive geology in various Oklahoma provinces is discussed by Curtis J. Hayes elsewhere in this symposium volume.

Oklahoma's three mountain regions have been the subject of much study. The Ouachita Mountains in the southeast make up an arcuate fold belt that consists mostly of Mississippian and Early Pennsylvanian sandstones and shales (Stanley, Jackfork, Johns Valley, and Atoka Formations), locally about 30,000 feet thick, deposited in a geosyncline or great trough. In the destruction of the geosyncline during Pennsylvanian time these strata were folded into broad anticlines and synclines and were thrust northward along a series of major thrust faults. Resistant units of steeply dipping sandstone form long, sinuous mountain ridges and hogbacks that tower 1,000 to 1,500 feet above adjacent shale valleys.

The Arbuckle Mountains in south-central Oklahoma make up an area of low to moderate hills (fig. 4) containing 15,000 feet of folded and faulted sediments ranging in age from Cambrian to Pennsylvanian (Ham, 1969). About 80 percent of these sedimentary rocks are limestones and dolomites and the remainder are shales and sandstones. Rocks in this part of the Southern Oklahoma geosyncline were thrust upward and were folded and faulted during several mountain-building episodes in the Pennsylvanian Period. The sedimentary

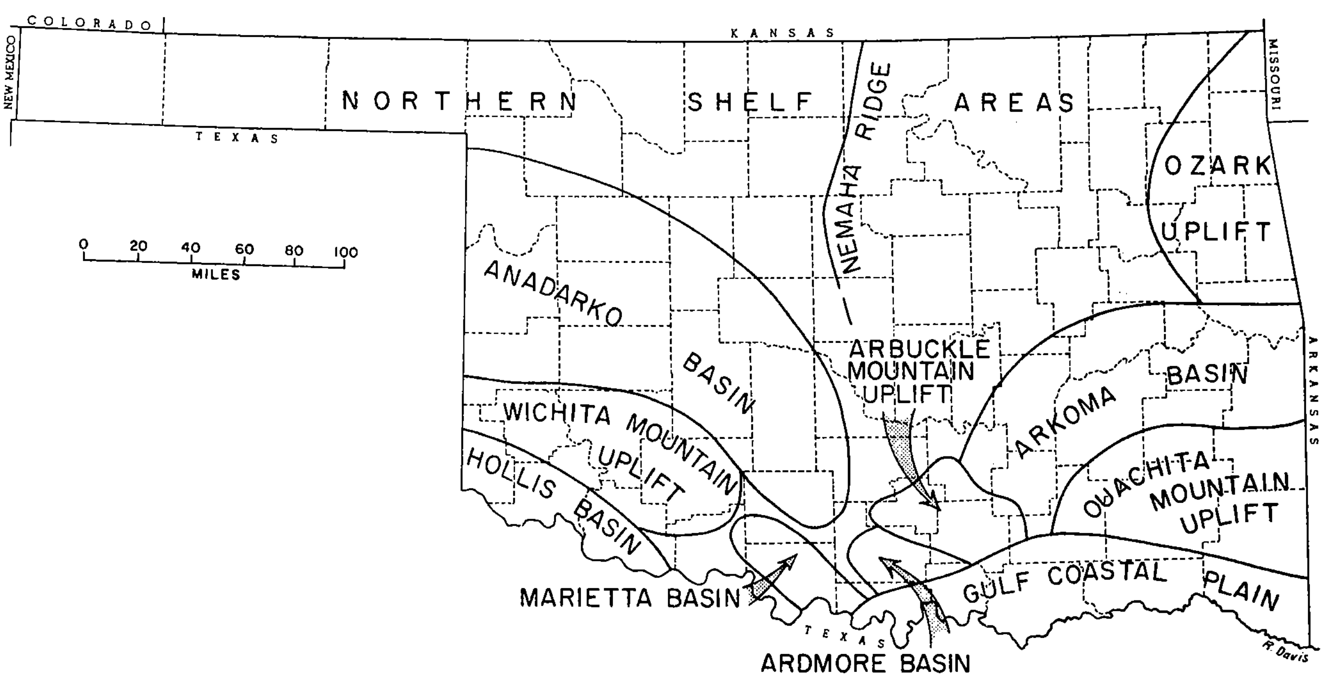

Figure 1. Major geologic provinces of Oklahoma.

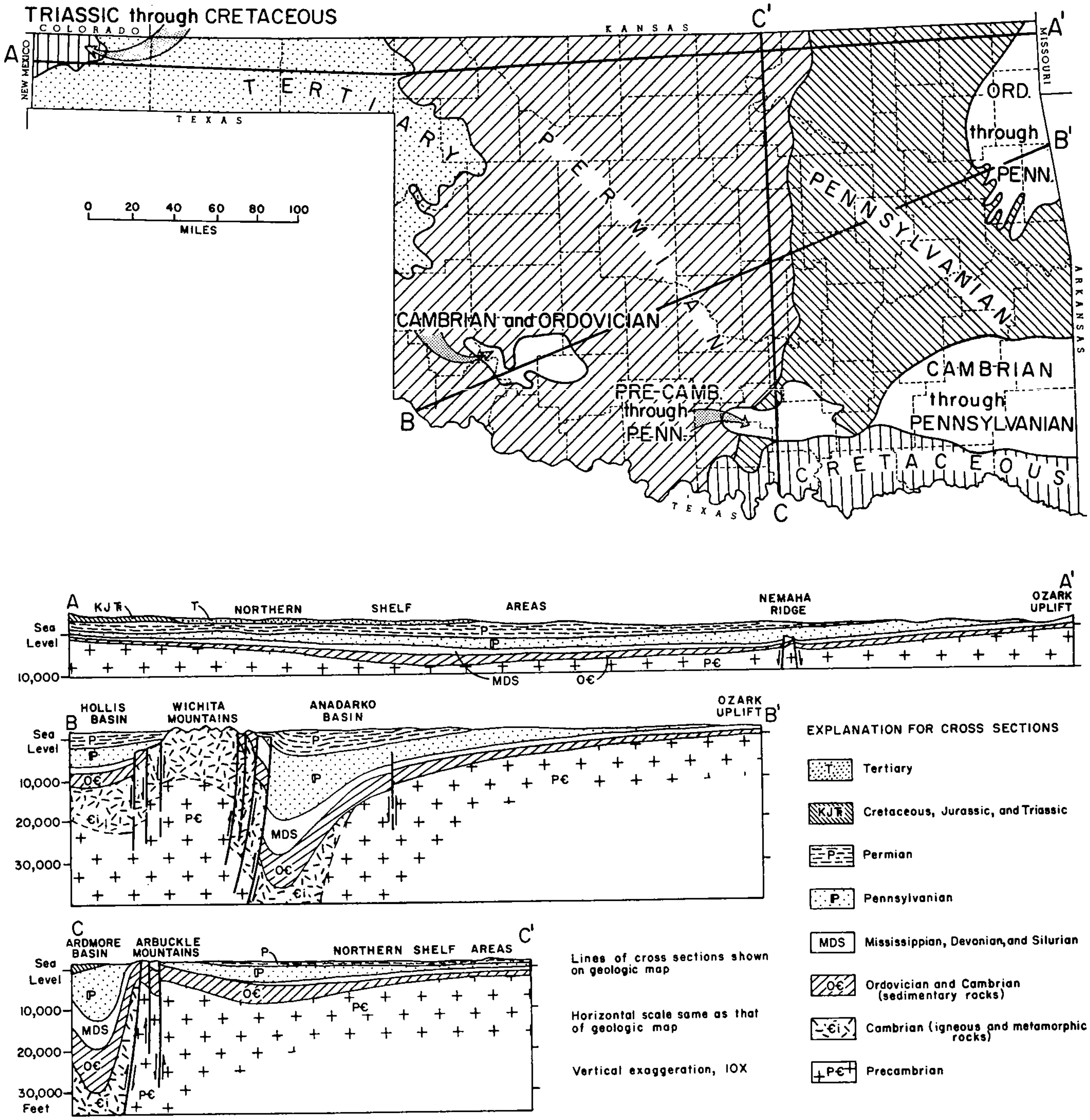

Figure 2. Generalized geologic map and cross sections of Oklahoma.

SYSTEM	CENTRAL AND NORTHERN OKLAHOMA		SOUTHERN OKLAHOMA GEOSYNCLINE (South-central and southwest Oklahoma)	OUACHITA GEOSYNCLINE (Southeast Oklahoma)
	Western Half	Eastern Half		
Quaternary	Alluvial and terrace deposits of gravel, sand, silt, and clay.			
Tertiary	Ogallala			
Cretaceous	Upper Lower			Gulf } (Gulf Coastal Comanche } Plain)
Jurassic	Morrison-Exeter			
Triassic	Dockum Group			
Permian	Ochoa(?), Guadalupe, Leonard, Wolfcamp		Ochoa(?), Guadalupe, Leonard, Wolfcamp	
Pennsylvanian	Virgil Missouri Des Moines Atoka Morrow		Virgil Missouri Des Moines Atoka Morrow	Des Moines (Arkoma basin) Atoka Johns Valley-Jackfork
Mississippian	Chester Meramec Osage Kinderhook Woodford Sh.		Chester Meramec Osage Kinderhook Woodford Sh.	Stanley Group Arkansas Novaculite
Devonian and Silurian	Hunton Group		Hunton Group	Missouri Mtn. Sh. Blaylock Ss.
U. and M. Ordovician	Sylvan Sh. Fernvale-Viola Ls. Simpson Group		Sylvan Sh. Fernvale-Viola Ls. Simpson Group	Polk Creek Sh. Bigfork Chert Womble Sh.
L. Ordovician and U. Cambrian	Arbuckle Group Honey Creek Fm. Reagan Ss.		Arbuckle Group Honey Creek Fm. Reagan Ss.	Blakely, Mazarn, Crystal Mtn., Collier, Lukfata ?
M. and L. Cambrian			Rhyolite, granite, gabbro, metasediments	?
Precambrian	Granite, rhyolite		Granite	?

Figure 3. General stratigraphic succession in major tectonic regions of Oklahoma. Principal regional unconformities shown by horizontal wavy lines; absence of stratigraphic record shown by vertical wavy lines. (After Miser, 1954; Ham, 1961; Jordan, 1967; and R. O. Fay, oral comm.)

cover was eroded from the underlying Precambrian granites in a 150-square-mile area in the southeastern part of the Arbuckle Mountains, making this the largest exposure of Precambrian rocks in the State.

In the Wichita Mountains of southwestern Oklahoma, granite, rhyolite, and gabbro are the dominant rocks (fig. 5). These igneous rocks are of Middle and possibly Early Cambrian age and are flanked by scattered outcrops of Cambrian and Ordovician limestones and dolomites like those of the Arbuckle Mountains. The Wichita fault blocks were thrust upward and slightly northward during several Pennsylvanian uplifts, at which time the cover of pre-Pennsylvanian sediments was eroded. The igneous rocks now form mountains 500 to 1,000 feet high, rising above a surrounding plain of Permian red beds.

Outcropping Paleozoic rocks outside the mountain regions are essentially horizontal, with dips of 10 to 50 feet per mile being most common. They typically form gently rolling hills and plains, although the thick shale units form broad, flat plains and valleys. In places, resistant sandstones and limestones cap cuestas and hills 100 to 500 feet high. Badlands, sinkholes, and caves are common in the gypsum-hill regions of western Oklahoma, and deeply dissected cavernous limestones and cherts are typical of the Ozark uplift. Surface rocks dip westward across northeastern and central Oklahoma and dip toward the axes of sedimentary basins in other parts of the State.

Outcropping rocks in the Anadarko, Hollis, and Marietta basins are flat-lying red Permian shales and sandstones. Few of the rock units are hard, and the topography is mostly gently rolling hills and broad flat plains. In some areas, thick beds of soft sandstone have been deeply dissected into steep walled canyons. Resistant gypsum layers in the Anadarko basin and western Hollis basin locally cap high escarpments (fig. 6).

The Ardmore basin is a lowland of folded Mississippian and Pennsylvanian shales and sandstones between the Arbuckle Mountains and the Gulf Coastal Plain. The general structure is synclinal, but a number of large anticlines are present within the basin. Dips of outcropping strata are steep, particularly along the margins of the basin, with angles of 45° to 90° being common. Outcropping dark gray shales are from a hundred to several thousand feet thick; they are separated by resistant sandstones, limestones, and conglomerates (10 to 15 feet thick) that form conspicuous subparallel ridges as much as a hundred feet above adjacent lowlands.

Outcrops in the Arkoma basin are Pennsylvanian shales and sandstones. Strata are gently folded into a series of anticlines and synclines. Resistant sandstones cap broad hills and mountains rising 300 to 2,000 feet above wide hilly plains and valleys.

The Ozark uplift is a deeply dissected plateau formed in gently dipping limestones and cherts of Mississippian age. Caves, solution cavities, and other karst features are more prevalent here than in any other part of Oklahoma.

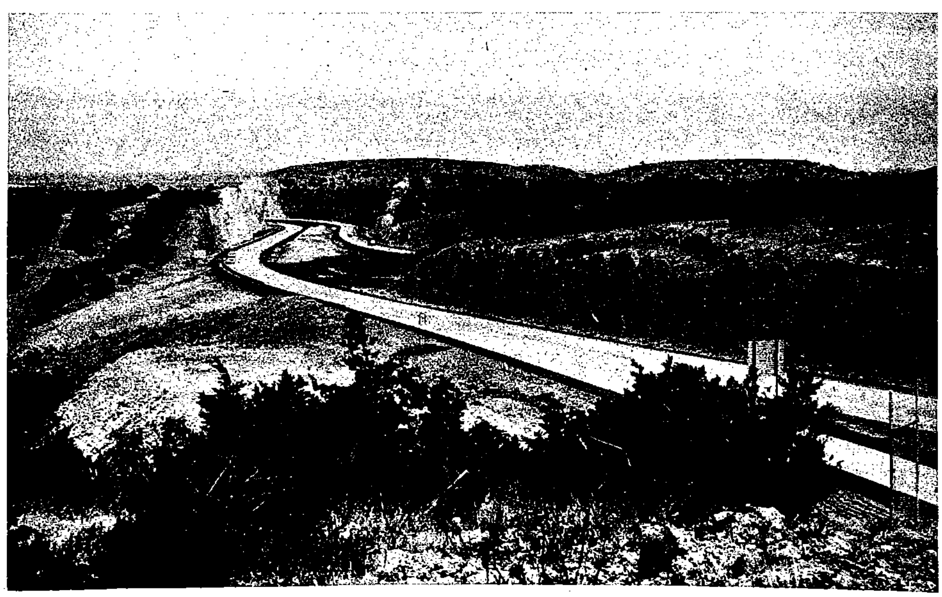

Figure 4. View of Arbuckle Mountains along Interstate 35. Steeply dipping limestone and dolomite make up most of the mountains. Half of a gentle synclinal fold is visible in the pre-split face in right center.

In most of the northern shelf areas, Pennsylvanian and Permian shales and sandstones dip westward 10 to 50 feet per mile. Most of the region comprises gently rolling hills and broad, flat plains. Resistant beds are more common in the Pennsylvanian strata, so relief is somewhat greater in the east where long cuestas overlook broad shale plains. The central part of the province is also variously known as the Prairie Plains homocline or the Central Redbed Plains.

Cretaceous strata of the Gulf Coastal Plain in the southeast are generally loose sands, gravels, limestones, and clays that dip gently southward toward the Gulf of Mexico. The sediments are only slightly dissected by streams and commonly form gently rolling hills and plains.

Tertiary outcrops in the west (fig. 2, map) are loose sands, gravels, and clays deposited by ancient streams and rivers draining the Rocky Mountains. They mantle part of the northern shelf area and Anadarko basin and constitute a featureless flat upland surface which is part of the High Plains.

The preceding summary of the geology of Oklahoma is expanded from the field-trip guidebook prepared for the 22nd Annual Highway Geology Symposium (Johnson and McCasland, 1971). Other recent articles by Jordan (1967) and Ham and Wilson (1967) contain more complete syntheses of Oklahoma's geologic history, and the reader is referred to them for additional information.

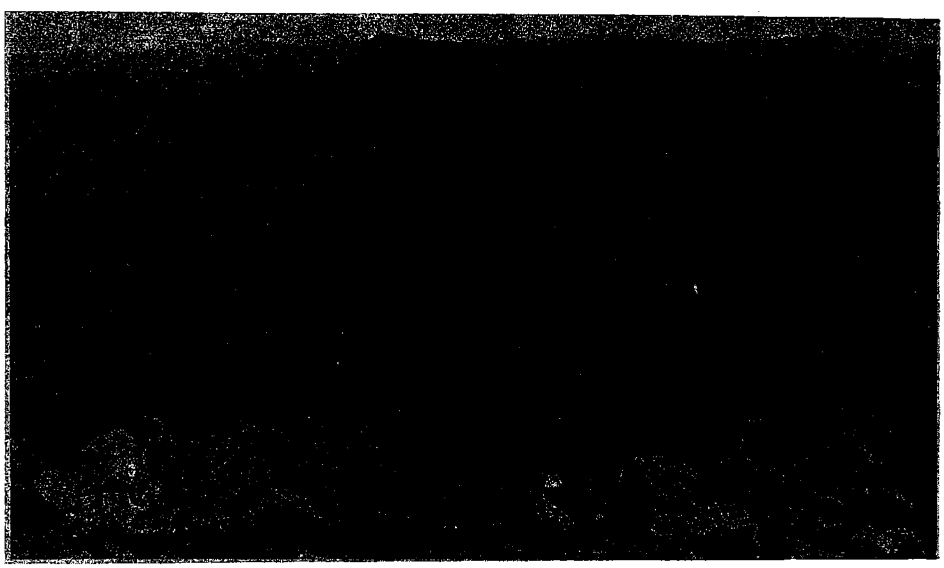

Figure 5. Cambrian granites form most of the rugged topography in the Wichita Mountains.

Figure 6. Horizontal beds of white gypsum capping high escarpments along Cimarron River in western Oklahoma.

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ENGINEERING CLASSIFICATION OF HIGHWAY-GEOLOGY PROBLEMS IN OKLAHOMA

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Abstract.--The vegetation, climate, geology, and soils of Oklahoma differ significantly from other states. These differences produce a set of highway-engineering-geology problems specific to Oklahoma. These problems can be classified into types such as landslides, sinkholes, and others. Geology and soil maps of many kinds, ranging from general to detailed, can be used to delineate their extent, but to be effective a problem-classification scheme must be devised according to the demands of the user. The highway geologist should assume responsibility for developing the classification into a comprehensible format for the user.

The 1970 New York Times Encyclopedic Almanac indicates that Oklahoma's total area is 69,919 square miles, 464 miles at the longest point and 230 miles at the widest. The total mileage of surfaced roads is 75,000, with about 12,000 miles of highways. The highest point in the state, 4,973 feet, is atop a lava flow at the western edge of the Panhandle. The lowest point, 300 feet, is in the extreme southeast corner of the state where the Red River enters Arkansas.

Climate changes are dramatic, especially considering that Oklahoma is landlocked. Rainfall ranges from an average of 15 inches in Kenton at the extreme western edge of the Panhandle to over 55 inches at the Kiamichi Mountain Observation Tower at the crest of Kiamichi Mountain in southeastern Oklahoma.

As far as vegetation is concerned, it ranges from piñon trees on Black Mesa, at the extreme western tip of the Panhandle, to bald cypress along the streams of southeastern Oklahoma. Several counties contain commercial grades of loblolly pine and short leaf pine. The most abundant species are post oak and blackjack oak. Approximately 10,300,000 acres, 24 percent of the state, is forested. Most of the forested area occurs in the eastern half of the State; the remaining portion is composed predominantly of tall, medium and short prairie grasses.

Oklahoma can be subdivided structurally into seven geologic provinces, each of which encloses smaller but, nevertheless, significant areas of differing geologic structure. Beginning in the far western part of the state and progressing eastward, the seven large provinces are listed as follows: the High Plains, Anadarko-Hollis-Marietta Basins, Prairie Plains Homocline, McAlester Basin, Gulf Coastal Plain, Ouachita Mountains, and the Ozark Uplift.

GENERAL GEOLOGY

The rock strata in these seven large areas can be described in a generalized manner as follows.

The High Plains area is composed principally of almost flat-lying weakly consolidated to unconsolidated, limy (calcareous) sandstones and conglomerates, with some soft, chalky, sandy limestones present locally. The High Plains are essentially Rocky Mountain outwash. Steep slopes are found only adjacent to large streams.

The Anadarko-Hollis-Marietta Basins region is composed principally of red-bed shale, with significant thicknesses of fairly soft, fine-grained sandstones. Locally, very thick (up to 100 feet) gypsum beds occur. Thin (one through five feet) dolomite beds occur. Dips are mostly gentle (usually less than 15 feet per mile), although a narrow band (about five miles wide) at the southern edge of the Anadarko basin shows dips of up to 250 feet per mile. Extensive granular terrace deposits occur adjacent to major streams.

The area referred to as the Prairie Plains Homocline consists principally of shales that are red bed on the western side, becoming grayish to the east. It contains thick sandstones which are fairly soft in the west but become harder to the east. Several thick (five through fifty feet) limestones are present in the northeastern portion of this province, producing pronounced eastern-facing escarpments.

The Ozark Uplift province contains hard, cherty limestones and cherts. The dips of these hard strata are gentle and trend westward.

The McAlester Basin is principally grayish shales and hard, thick (10-15 feet) sandstones. Strata in this area are gently folded with many local anticlines and synclines. Dips are generally less than 15 degrees. Many faults are evident locally.

The Gulf Coastal Plain area consists predominantly of gray and brownish clayey shales with some thick (50 feet), soft limestones and thick, weakly consolidated sandstones. Strata are nearly flat-lying with very gentle (about 15 feet per mile) southward dips prevalent.

The Ouachita Mountains contain many shales with nearly equal amounts of very thick, hard sandstones. This is a very rugged, mountainous area, highly faulted and with strata at steep dips. Pronounced ridges trend generally southwest to northeast.

The Wichita Mountain province comprises a small area located in the southern portion of the Anadarko-Hollis-Marietta Basins province. Most of the area is comprised of mountainous outcrops of granite with some rhyolite and anorthosite.

Another small province, the Arbuckle Mountains, occurs in the south-central portion of the state. It is primarily composed of steeply dipping rocks, largely carbonates with some granite.

The core area and the Potato Hills, both located in the Ouachita Mountain province, are small rugged areas composed of hard strata of mostly cherty limestones and limestones.

The Sierra Grande-Las Animas arch occurs in the extreme northwestern end of the Panhandle. The rocks are mostly red shales including several thick (10-50 feet), moderately hard sandstones. The eroded, flat-lying strata form small buttes in the area.

ENGINEERING-GEOLOGY PROBLEMS

The following are some of Oklahoma's highway-engineering-geology problems--listed by frequency of occurrence rather than by degree of engineering hazard or cost: (1) seepage, (2) landslides and slumps (slope stability), (3) rippability, (4) expansive soils, (5) rock excavation, (6) materials location, (7) erosion-sedimentation, (8) sinkholes and cave-ins, (9) corrosion, and (10) land use. Figures 1-4 illustrate several problems, which, of course, will be more prevalent and severe in certain parts of the State. For instance, landslides are almost restricted to the eastern third of our State. This is due mainly to higher rainfall (45+ inches), steep slopes, and thick sequences of shale and (or) colluvium that are present there. So, each of the geologic provinces will have its own specific set of engineering problems. The following tables and explanations show, in general terms, the engineering-geology problems associated with each of Oklahoma's seven large geologic provinces discussed in this paper.

High Plains

The High Plains produce relatively few problems (table 1). There are some minor surface-drainage problems in sinkhole areas and local swelling-soils and erosion-sedimentation problems. Some of the erosion is due to high winds. In this area, however, it is difficult to locate suitable coarse aggregate materials for paving-mixes or riprap.

Anadarko-Hollis-Marietta Basins Area

The Anadarko-Hollis-Marietta Basins also produce relatively few problems, although there are more than in the High Plains (table 2). Erosion-sedimentation is perhaps the major one. Locally, at certain times of the year, wind erosion is a problem. Severe sinkhole hazards occur locally in the thick gypsum areas, and gypsum beds over ten feet thick are usually nonrippable. Some seepage occurs in areas where thick gypsums or thick sandstones outcrop and where thick granular terraces overlie less permeable shales. There are occasional local swelling-soils problems, mostly confined to the southern

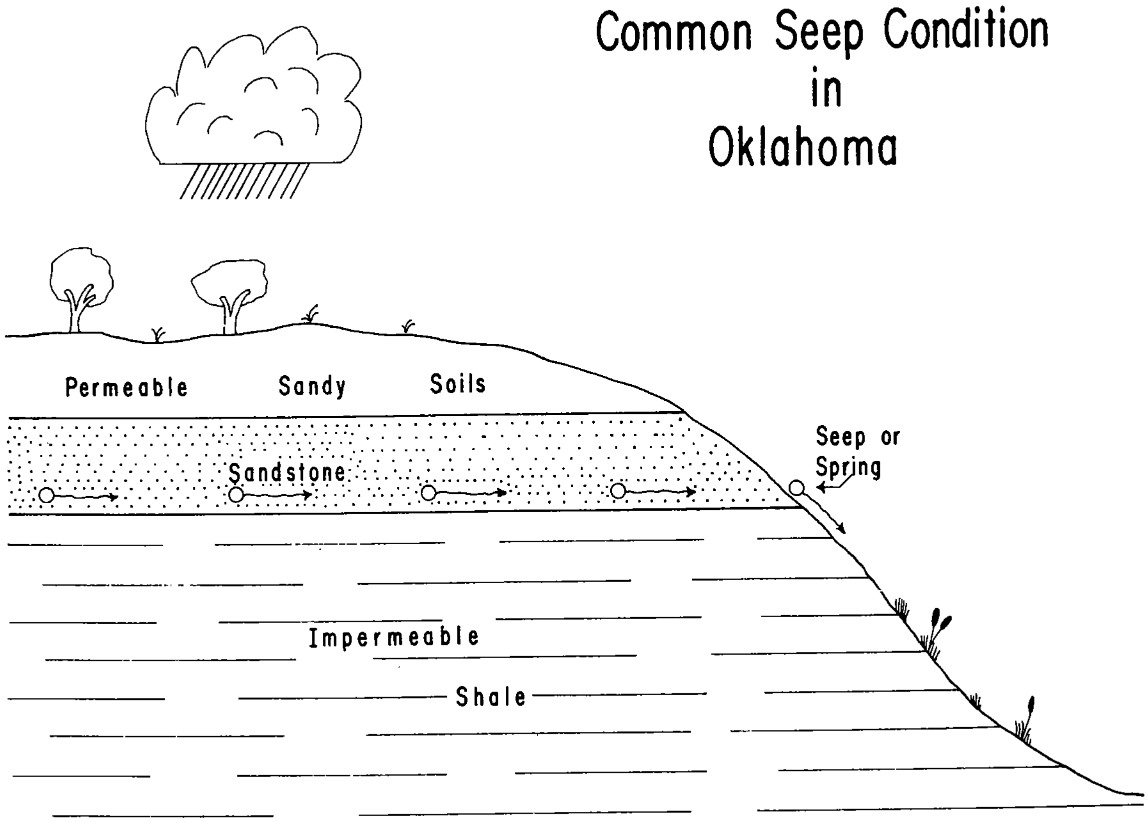

Figure 1. The most common type of seepage in Oklahoma is from sandstone overlying less permeable shale.

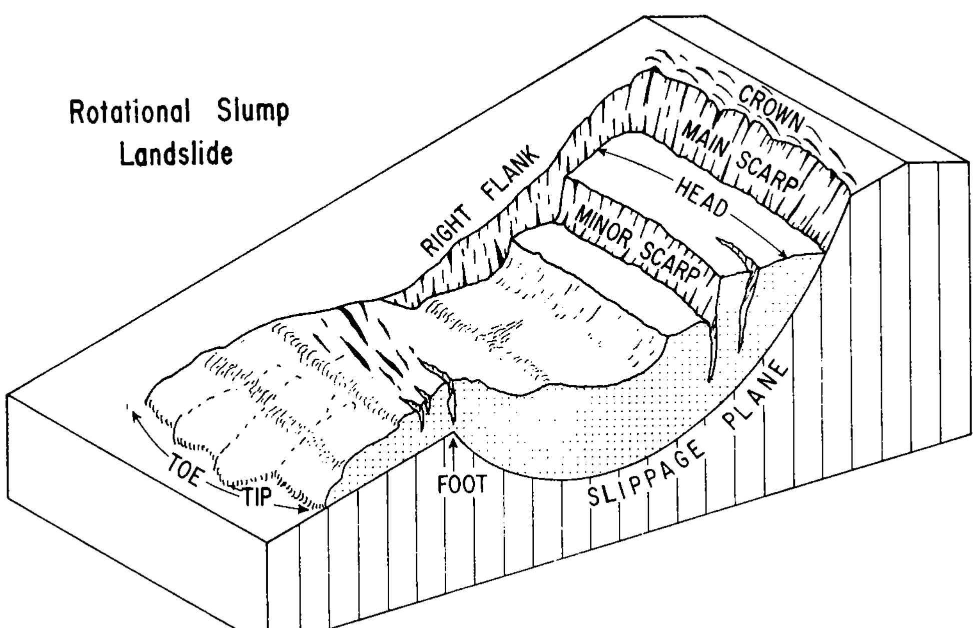

Figure 2. The most common type of landslide in Oklahoma is the rotational slump. (After Highway Research Board Spec. Rept. no. 29, 1958, plate 1-t).

portions of this area. A few isolated salt springs are corrosive. Coarse aggregates and riprap are difficult to obtain.

Prairie Plains Homocline Area

In the Prairie Plains Homocline, the problems become more frequent and pronounced (table 3). In the western portion of this province the problems are similar to those of the Anadarko-Hollis-Marietta Basins, but in the east seepage becomes a frequent problem, particularly on the east-west roads which must

Table 1. Engineering-Geology Problems Encountered in the High Plains

Problem	Severe	Moderate	Slight to none	General	Local
Seepage			X	X	
Slumps			X	X	
Rippability			X	X	
Expansive soils			X		X
Rock excavation			X	X	
Materials location	X			X	
Erosion		X		X	
Sinkholes			X		X
Corrosion			X	X	
Land use			X	X	

Table 2. Engineering-Geology Problems Encountered in the Anadarko-Hollis-Marietta Basins

Problem	Severe	Moderate	Slight to none	General	Local
Seepage			X	X	
Slump			X	X	
Rippability		X			X
Expansive soils			X	X	
Rock excavation		X			X
Materials location		X		X	
Erosion		X		X	
Sinkholes	X				X
Corrosion			X	X	
Land use			X	X	

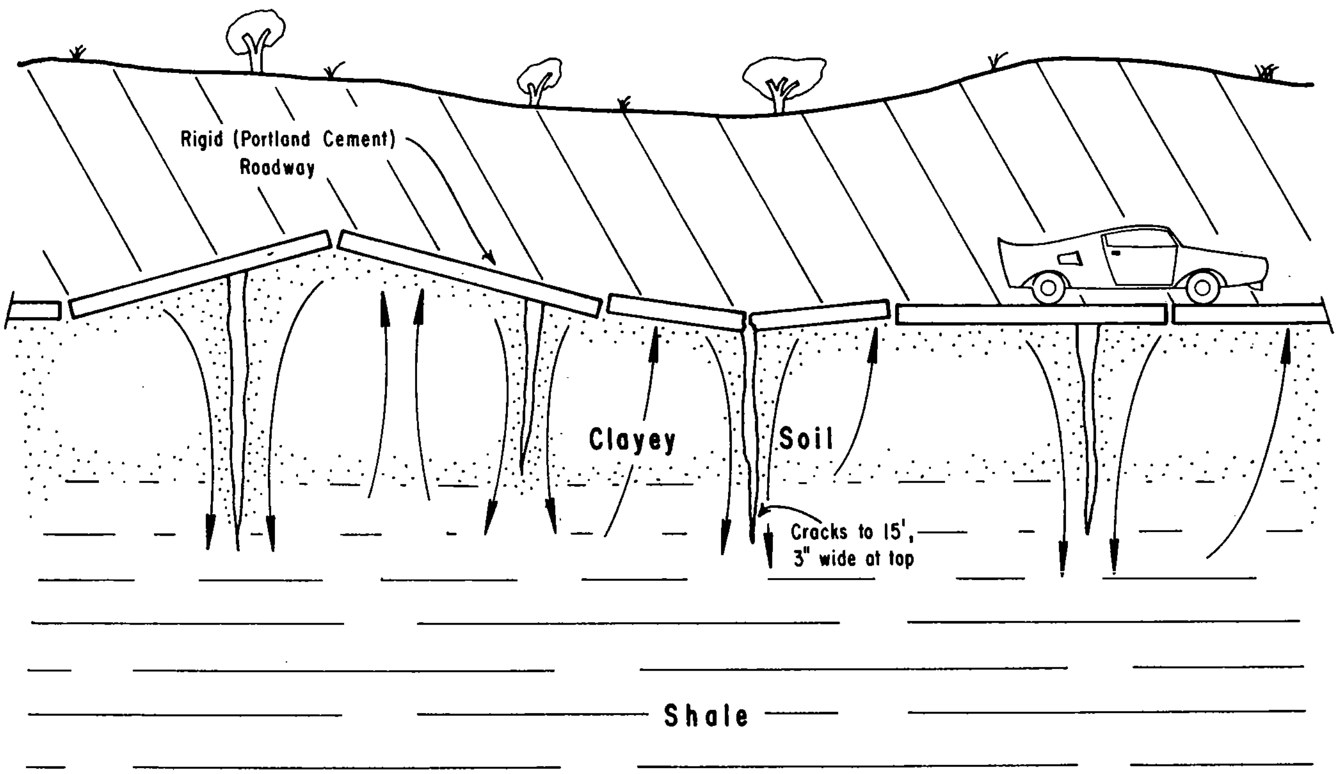

Figure 3. Expansive soils cause local problems throughout the State, but especially in eastern Oklahoma.

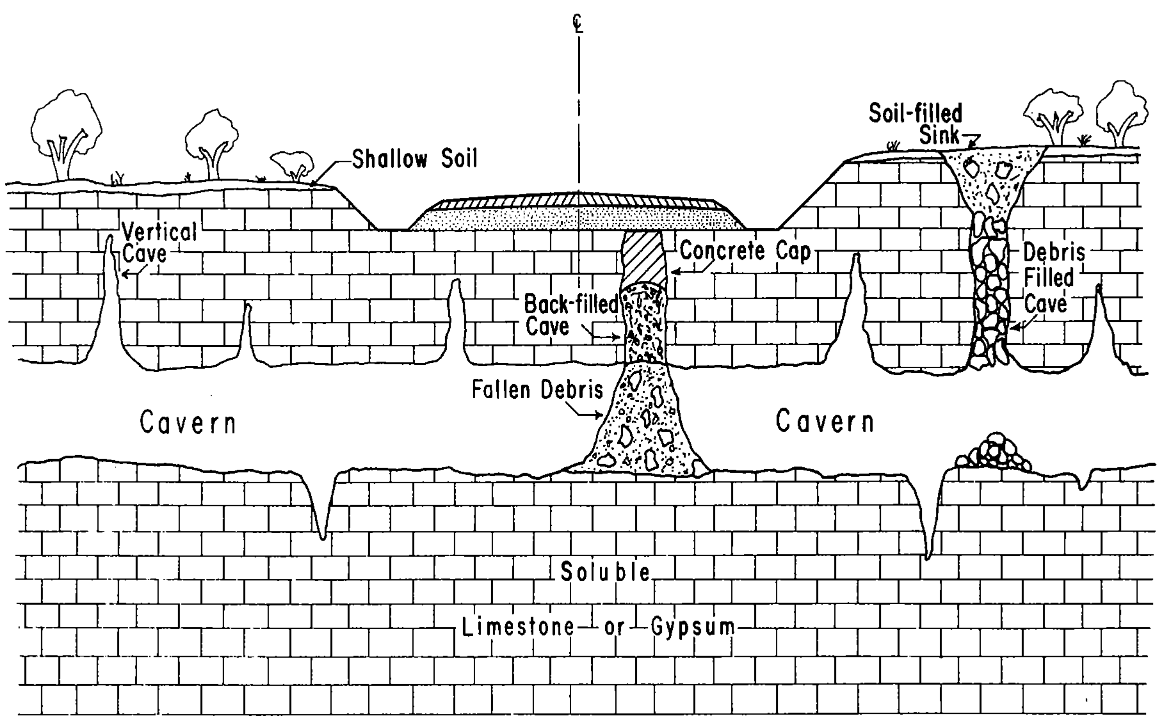

Figure 4. Sinkholes and cave-ins are local hazards in much of Oklahoma. This drawing shows a method of repairing a sink that occurred beneath a roadway.

cross the eastward-facing escarpments. Some slope instability is encountered in many of the deeper cuts. Land use must be planned around the cities of Tulsa and Oklahoma City to insure the proper placement of highways. The shale intervals between the nonrippable limestones and sandstones produce wide valleys underlain by clayey, moderately expansive soils.

Table 3. Engineering-Geology Problems Encountered in the Prairie Plains Homocline

Problem	Severe	Moderate	Slight to none	General	Local
Seepage		X		X	
Slumps		X			X
Rippability		X		X	
Expansive soils		X		X	
Rock excavation		X		X	
Material location			X	X	
Erosion		X		X	
Sinkholes			X	X	
Corrosion			X	X	
Land use		X			X

Ozark Uplift Area

In the Ozark Uplift, seepage is very common, but the most costly problem is rock excavation, as the entire area is underlain by thick sequences of hard, cherty, nonrippable limestones or other hard rocks (table 4). The hard rock also makes fine aggregates difficult to find. In one local area mine cave-ins present a distinct hazard; acid-seepage waters are corrosive to uncoated steel culverts in places; and unstable soil materials are present near the base of some of the larger hills.

Table 4. Engineering-Geology Problems Encountered in the Ozark Uplift

Problem	Severe	Moderate	Slight to none	General	Local
Seepage	X			X	
Slumps		X			X
Rippability	X			X	
Expansive soils			X	X	
Rock excavation	X			X	
Materials location		X		X	
Erosion			X	X	
Sinkholes	X				X
Corrosion	X			X	
Land use			X	X	

McAlester Basin Area

Thick shale sequences in the McAlester Basin produce large areas of moderately expansive soils. Seepage is again prevalent (table 5). Hard-rock excavation and nonrippable rocks pose difficulties. Slope stability is a problem on the sides of large hills, and abandoned coal mines produce sink (cave-in) problems.

Table 5. Engineering-Geology Problems Encountered in the McAlester Basin

Problem	Severe	Moderate	Slight to none	General	Local
Seepage	X			X	
Slumps	X				X
Rippability	X			X	
Erosion		X		X	
Sinkholes		X			X
Corrosion		X		X	
Land use			X	X	

Ouachita Mountains Area

The Ouachita Mountains province presents the most severe problems (table 6). It is an area of unstable slopes, many springs and seeps, very rugged and mountainous topography, and thick nonrippable rocks. Seepage waters are usually corrosive to steel culverts.

Table 6. Engineering-Geology Problems Encountered in the Ouachita Mountains

Problem	Severe	Moderate	Slight to none	General	Local
Seepage	X			X	
Slumps	X				X
Rippability	X			X	
Expansive soils		X		X	
Rock excavation	X			X	
Material location		X		X	
Erosion		X		X	
Sinkholes			X	X	
Land use			X	X	

Gulf Coastal Plain Area

Soil expansion almost always occurs on the shale formations in this province (table 7). Seepage, due to several soft unconsolidated sandstones overlying the clayey shale formations, is a slight problem. Erosion can be a local problem, as soft sandstones tend to erode on steeper slopes. In the Red River flood plain, surface drainage occasionally becomes a local problem.

Table 7. Engineering-Geology Problems Encountered in the Gulf Coastal Plain

Problem	Severe	Moderate	Slight to none	General	Local
Seepage		X		X	
Slumps		X		X	
Rippability			X	X	
Expansive soils	X			X	
Rock excavation			X	X	
Materials location			X	X	
Erosion		X			X
Sinkholes			X	X	
Corrosion			X	X	
Land use			X	X	

GROUPING PROBLEMS

Classification schemes are useless unless they convey to users new knowledge (or old knowledge presented in a new way) that will enable them to do their job quicker, better, at less cost, or all three. To do this, a classification must be comprehensible.

The preceding forms and explanations were examples of a method of relating highway-engineering-geology problems to mapped geologic units (provinces in this case). For general purposes, for instance in preparing cost estimates for future road-building programs, this classification of engineering problems might be adequate. Often, though, engineers require more detail in order to locate problem areas more accurately.

The results of an Oklahoma Highway Department research project have been released as Engineering Classification of Geologic Materials. This project utilized "geologic unit" maps to delineate problems such as seepage, slope stability, materials suitability (see figure 5). These maps were developed from Oklahoma Geological Survey bulletins and circulars as well

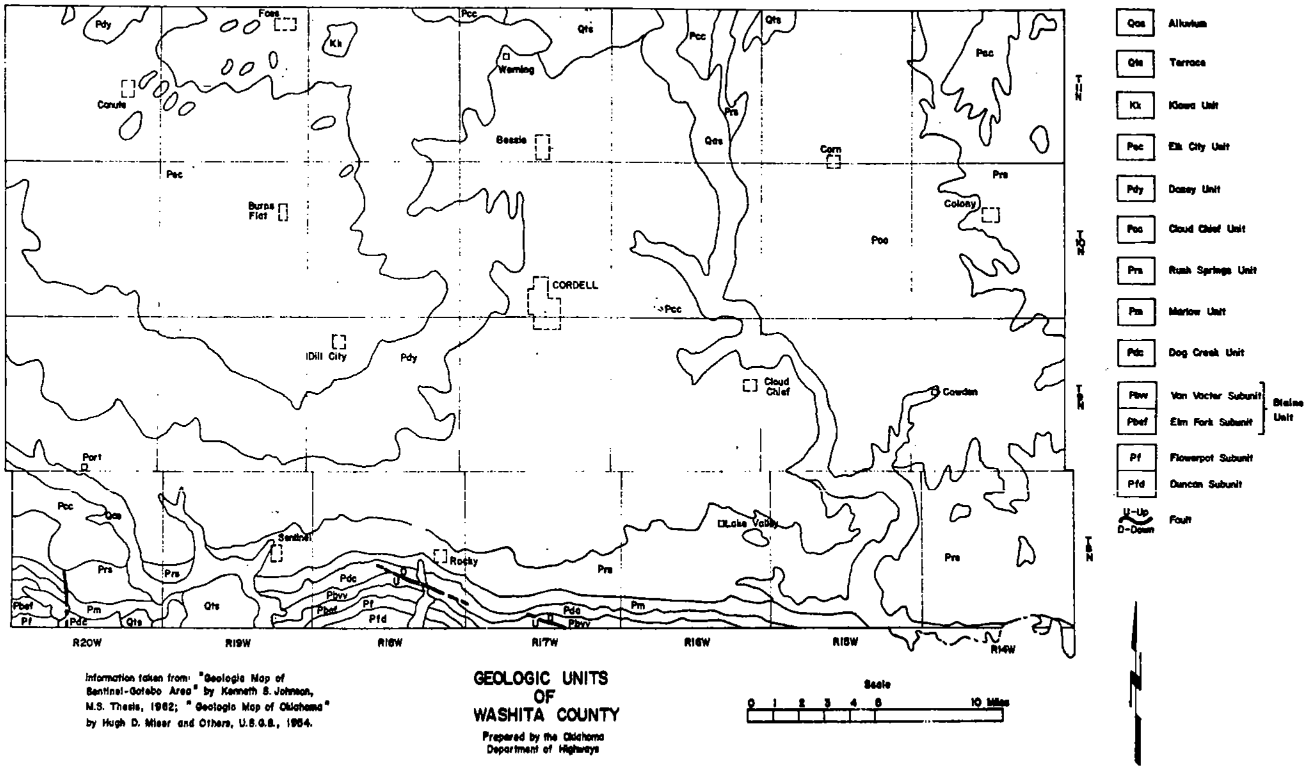

Figure 5. Geologic unit map of Washita County, Oklahoma, derived from published and unpublished geologic surface maps. Problems such as landslides and seepage can be associated with such units.

as M.S. and Ph.D. theses. Engineering geology problems were assigned for each mappable rock unit. The detail provided by this classification was satisfactory for many engineering needs, and the language of the classification scheme was such that engineers as well as laymen (contractors) could read and understand the work.

Classifying and categorizing engineers' geological problems should be performed according to their needs. If a set of problems exist that are geologically related and can be associated with certain soil areas or rock types, it seems probable that these problems can be located and classified. Hopefully, they can be grouped according to the type, severity, and frequency of the problem. In certain cases, a problem may be prevalent in a broad area, for instance corrosive seepage waters exist throughout the Ozark Uplift and Ouachita Mountains provinces. A map based on soil series, soil associations, or detailed geology could have been developed to locate the corrosive waters, but the engineers responsible for making decisions in regard to culvert corrosion wanted a broad, large-scale map instead of great detail. Therefore, a "Corrosion-Stress Map" was developed, based on geologic provinces (see figure 6).

CORRUGATED GALVANIZED STEEL PIPE

CORROSION STRESS MAP

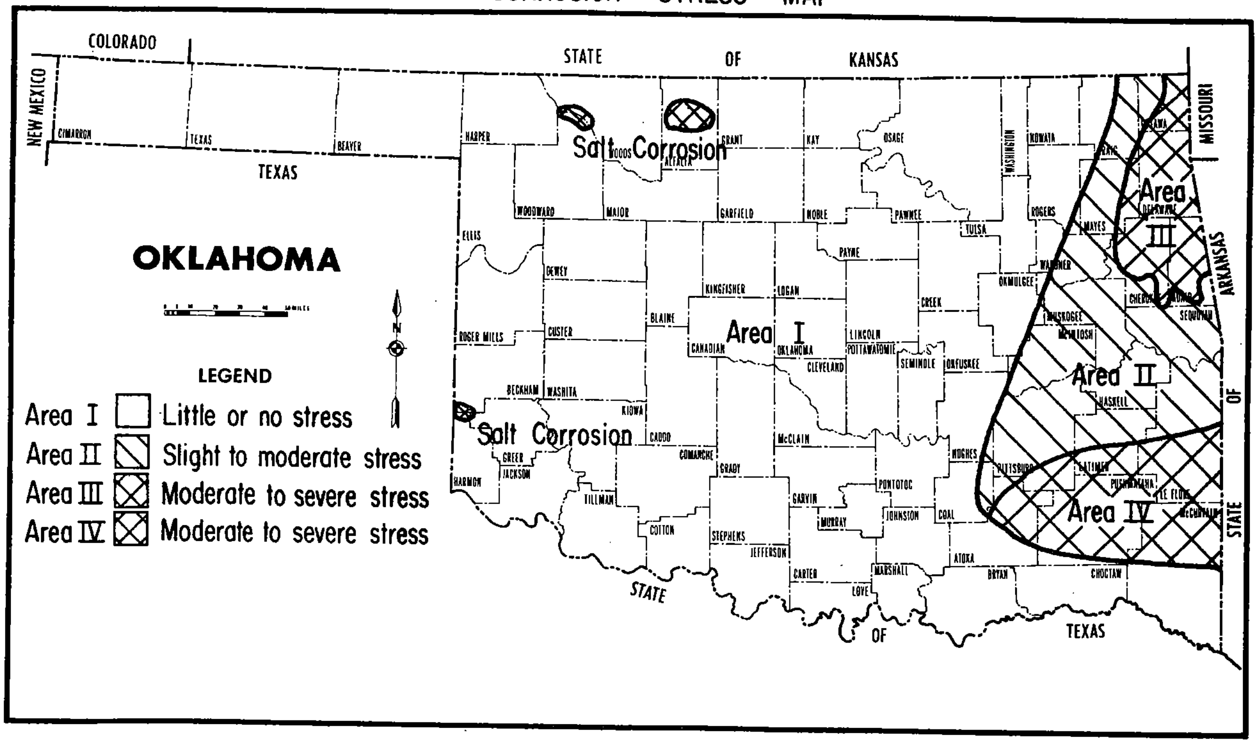

Figure 6. A corrosion-stress map of Oklahoma showing geologic provinces where uncoated steel pipe will corrode unless protective measures are taken. (After Hayes, 1971, figure 1).

While a geologically related corrosion problem can be associated with a geologic province, another factor should be used to determine when a landslide hazard exists. In Oklahoma, landslides can be satisfactorily associated with rock units (formations, members, and so forth). The Johns Valley unit, located in the southeastern part of the state, will almost always produce a landslide unless special designs are incorporated to offset the propensity of the Johns Valley to slump. In the case of this rock unit, delineation of the mapped unit also delineates the extent of the landslide problem and the engineering design for landslide prevention.

For an engineering-geology problem such as materials location, rock-unit map delineations such as "terrace deposits" are not specific or consistent enought to be used with confidence. Still another device is needed. This time a soil survey will fill the bill. Since by definition a soil series has nearly constant soil properties, a map (survey) showing where the soils occur also shows where materials occur, or at least where one should go to investigate (figs. 7, 8). For example, the Dougherty Soil Series, wherever mapped, always meets the requirement for Oklahoma's "Type I

Subbase" (15% to 45% passing the #200 sieve, liquid limit 30 or less, plasticity index 10 or less--Oklahoma Agriculture Experiment Station, 1969). The Dougherty soil is developed in certain sandy terrace deposits.

Meaningful land-use plans allow (and sometimes demand) the use of more than one kind of map for a comprehensive, sensible method of planning (Flawn, 1970, and Bartelli, 1966). For instance, in the area in and about Austin, Texas, it can be shown that certain geologic units, e.g. the Eagleford Formation, exhibits low to moderate slope stability, low infiltration capacity, and low bearing capacity. In and around Oklahoma City, Oklahoma, the Konawa Soil Series exhibits low shrink swellage, moderate to high permeability, and low plasticity. By obtaining earth-material characteristics derived from either of two aspects, geology and (or) soil science, and then relating these to engineering needs, very worthwhile plans can be made. The map units in these surveys show where the particular set of characteristics are located. Thus, with proper interpretations, a valid land-use plan can result--one that highway planners can use to good advantage.

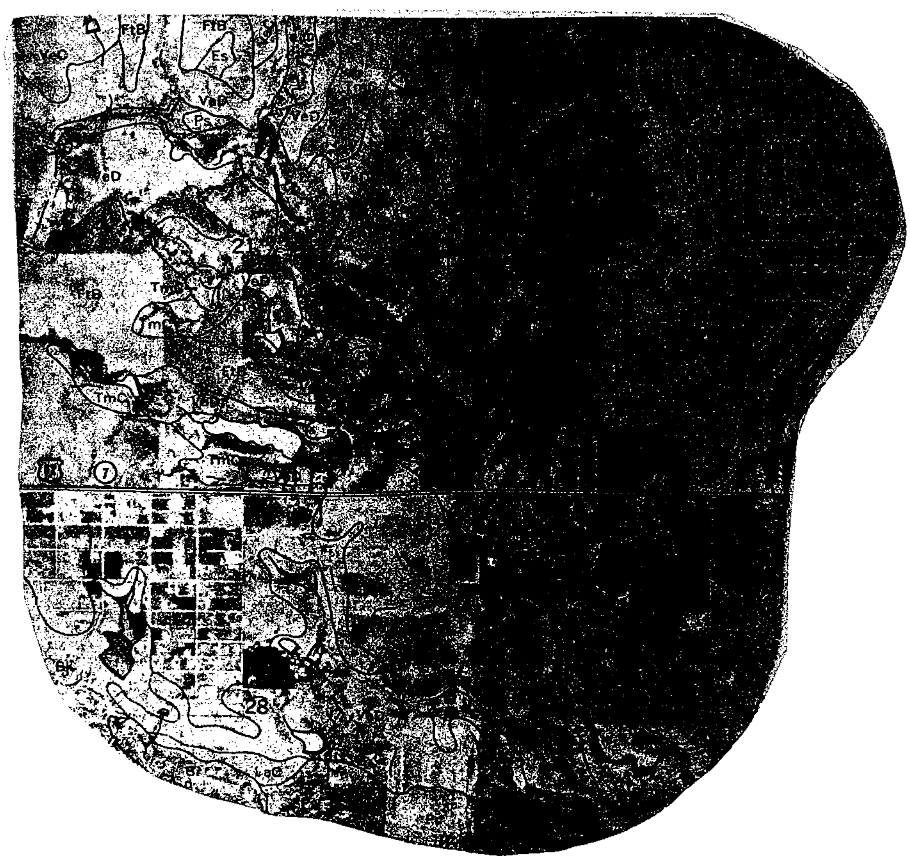

Figure 7. A portion of a soil survey of Comanche County, Oklahoma, showing soil mapping units which can be used for materials locations, land use plans, etc. (After Soil Survey of Comanche County, Oklahoma, 1967, Sheet 58).

Classification schemes must be derived with people in mind. This is not easy to do for at least two reasons. For one thing, the people who request the classification might not be the people who will routinely use it. But these requesters take it for granted that geologists will develop a classification system and explain it in terms anyone can understand. For example, as a result of a research project concerning erosion-control vegetation on backslopes, it was recommended that "love grass not be mowed." This sounds like a concrete, easily understood statement--except that the person responsible for mowing must know what love grass is. On the job, love grass was confused with other grasses and mowed down. What should have

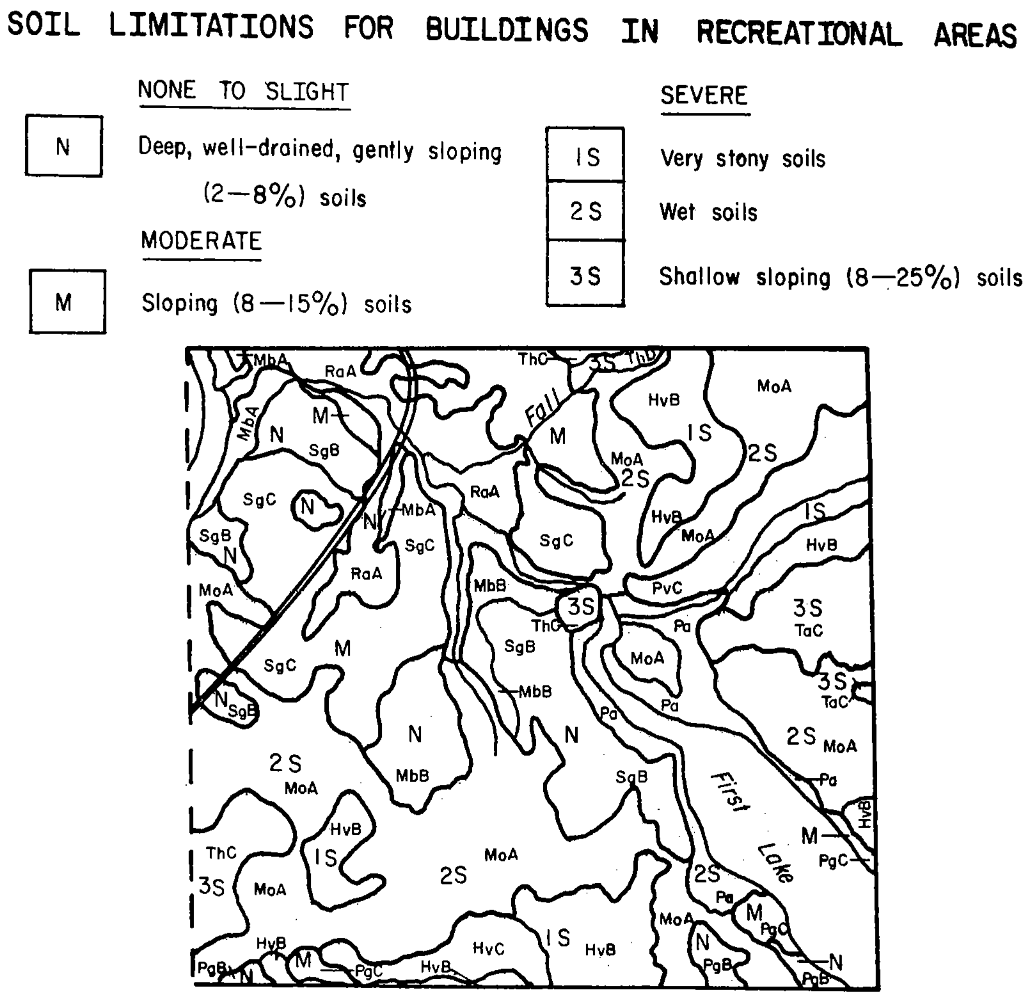

Figure 8. A land-use map based on a soil survey showing areas where buildings should or should not be built. (After Soil Surveys and Land Use Planning, *1966, pl. 15, p. 194).*

been said was, "Don't mow backslopes in this area." The classifier of vegetated backslopes did not gear his directions to the man who was going to use the information.

Secondly, the problem grouping must be done according to the engineer's needs, even if these needs are expressed inaccurately. Many times engineers devise their own geologic interpretations of problems. This can lead the classifier astray. For instance, an engineer may request something like, "Give me some information concerning those lousy seepage problems I have in these mountains over here. I wanna be able to tell when I'm goin' to have to put in underdrain." There is no doubt that he has a water problem, but it is really seepage? It would have been better for both the engineer and the geologist if the situation were presented as a "water problem" instead of "seepage." The geologist will review the pertinent elements concerned with highway water problems and determine how they can be grouped. For example, the classifier may arrive at the conclusion that certain soils have high water tables during the spring rainy season. Hence, a map with delineations showing where high water-table soils occur, along with their associated water table levels, might provide the answer to the engineer's problem.

Many of the geological engineering problems encountered by the highway planner, designer, builder, and maintainer can be grouped--either in a general or detailed manner, depending on the scope and limitation of the engineer's problem. The grouping can lead to better quality control of highway construction. The quality control comes from the ability to predict when, where, and with what intensity problems will occur, but the organization and language of the problems classification must be fitted to the final user in order for it to be effective.

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PREDICTABILITY OF SHALE BEHAVIOR

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Abstract.--Studies in the characterization of shales from a material viewpoint and by engineering behavior suggest that shale may be treated as a statistical macroscopic equivalent obeying the principles of continuum mechanics.

Shale disintegration is effected through the use of an ultrasonic device. This technique shows the potential to become a good indicator test for shale behavior. To further augment the significance of this test it is proposed that the analysis of shale behavior be based on a stochastic process and shale disintegration be treated as somewhat analogous to brittle fracture.

INTRODUCTION

Shales are sedimentary rocks containing large amounts of silt and clay. Various methods of classification and identification of shales have been proposed, but none is universally accepted (Underwood, 1967). This, in a sense, implies that some difficulty is experienced in characterizing shales from the standpoint of composition or engineering behavior. As a possible explanation for this difficulty, two reasons are offered: (1) the laboratory techniques and measurements applied to shales yield values which are not sufficiently close to the true mean values; (2) the effect of environmental factors has not been adequately and completely assessed and therefore the field-induced changes are different from the ones predicted.

The Oklahoma Department of Highways has expressed the allotropic behavior of shales and the associated engineering problem in terms of "the change in character and physical properties of shales when these shales are disturbed during construction and placed where they are subjected to various weathering agents." The changes referred to in the foregoing statement relate to the augmentation of the amount passing the no. 200 U.S. standard sieve, the attendant increase in the plasticity index, and the decrease in the resistance of the material to environmental changes. These alterations result in slides on the backslopes of cut sections and in excessive deformation or settlement of the pavement.

Maintenance measures, like any treatment of symptoms, have proven expensive. This work investigates the reasons for shale behavior and attempts to devise a predictive test methodology.

ANALYTICAL CONSIDERATIONS

Shale performance, as observed in the field and presented in the preceding paragraph, may be analyzed in terms of yield and failure criteria outlined for soils (Scott and Ko, 1969). Basically, the boundary-value problem reverts to one which is characterized by the shearing resistance depicted in the Mohr-Coulomb expression consistent with the limit equation (equation 1):

$$\sigma_{max} = \sigma_{min} N^2 + 2cN,$$

where $N = \mathrm{Tan}\ (\frac{\pi}{4} + \phi/2)$, ϕ = angle of interparticle shearing friction, and c = cohesion. Because of the wide variety of possible failure surfaces and other associated complexities, simplifications have been introduced so that, in practice, the failure surface is _a priori_ assumed to be the piecewise linear surface expressed as (equation 2):

$$\tau = \sigma_n \tan \phi + c,$$

where τ = shear stress, and σ_n = normal stress.

Essentially, then, the solutions to the engineering problems of shales--in the present context--could possibly be found in the existing solutions of those problems in continuous media (Fung, 1969) which deal with irrecoverable deformations and the collapse of the medium. Inferred in this approach is the acceptability of applying 3 postulates of the continuous media to shales: (1) the boundary distances and load-application areas are very large, compared to individual and discrete particles; (2) while the intrinsic molecular forces are of high intensity, their range of influence is short and act in the neighborhood points where stresses are generated; (3) the body is treated as a _statistical macroscopic equivalent_.

It becomes necessary to point out that in the macroscopic treatment, shown in figure 1, the shearing stress on the slip surface is not the maximum shearing stress, R, but is R cosine ϕ (Dunker and Prager, 1952); the obverse phenomenon is that failure occurs at an average energy below the theoretical value of atomic-bond strength or at an energy level which is nonuniformly distributed, thereby creating points of stress concentrations. These observations assume additional importance because of the possible analogy to Griffith's theory of fracture (Tetelman and McEvily, 1967). Another important feature of the macroscopic approach is that when the external work on the body equals or exceeds the rate of internal energy dissipation, collapse occurs to the extent that plastic deformation is accompanied by an increase in volume.

Basis of Experimentation

Inasmuch as shales are composed of discrete particles of varying shapes and orientations, their shear-response equations include both the friction and the cohesion parameters. The general tendency, however, is for cohesion to be the predominant contributory factor to strength. Thus, it may be possible to formulate the mechanomaterial problem and translate it to a laboratory test wherein either the bond strength is directly or indirectly evaluated under the most critical conditions. It may be argued that such a test may not be a test of fundamental concepts but primarily one of quality control. Be that as it may, it should be emphasized that the basic requirement of the design process is to have an explicit representation of the resistance and durability of the intrinsic shearing forces to the externally applied energy, the latter being in the form of changes in load intensity and (or) environmental equilibrium.

The conspicuous disintegration of shale materials in essence is the change of a continuous material into a discontinuous one. This supports the opinion that the rupture of the cohesive bonds takes place in a thermodynamically irreversible manner and that new surfaces are formed; the indirect measurement of this overall change which the shale undergoes has been formalized by James C. Gamble of the University of Illinois Department of Civil Engineering (written comm.) into a slaking-durability test (Gamble, 1971). Another method which is currently being tested at the University of Oklahoma is that of the ultrasonic treatment. Geologists have used ultrasonics to breakdown "chunks" of shale to prepare them for mineralogical and granulometry analyses.

Description of Ultrasonic Equipment

Literature surveys indicate that the "probe" type of equipment is preferred by researchers, both in the United States (Savage, 1969) and in Europe (Fagnoul,

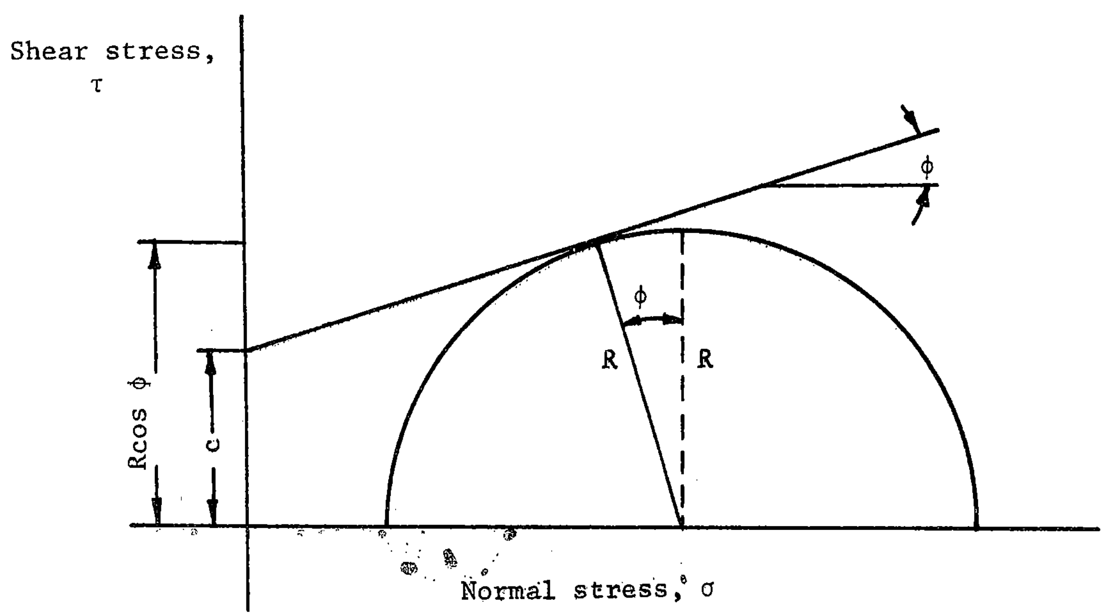

Figure 1. Mohr-Coulomb Hypothesis.

1966). Charles J. Mankin, director of the School of Geology and Geophysics at The University of Oklahoma is using probe equipment that essentially consists of inserting a probe into a beaker containing the shale test specimen plus water (oral comm., 1970). The ultrasonic action of the probe causes the shale to disintegrate.

In the present study, the ultrasonic equipment is of a tank type as depicted in figure 2. The energy is produced by transducers which change the low-frequency electrial energy into high-frequency (20,000 cps or higher) mechanical sound waves.

The sound waves, in turn, pass through the medium of water in the tank and produce an alternating condition of negative and positive pressure at any one point in the tank, as depicted in figure 3. As the negative pressure, which is below the vapor pressure of water, passes a point, it causes thousands of very small vapor bubbles to form. A half cycle later, in 1/40,000 of a second (20 kc), a positive pressure condition is created wherein the wave energy causes the vapor bubbles to implode, burst inwardly. This is called cavitation but it should not be confused with the formation of air bubbles in degassing a liquid. The quantity of energy expended in any one implosion is extremely small, but due to the small area, it is calculated that pressures in the order of 10,000 psi and temperatures approximately 20,000 ^{o}F are developed and dissipated. Cavitation causes the bonds between individual particles making up the shale to break and, thus, separate.

Test Methods

Because the original purpose of this phase of investigation was to ascertain how much more breakdown occurs in shales through use of ultrasonics than by the conventional mortar and pestle method, grain-size analysis and plastic properties were set as tentative criteria.

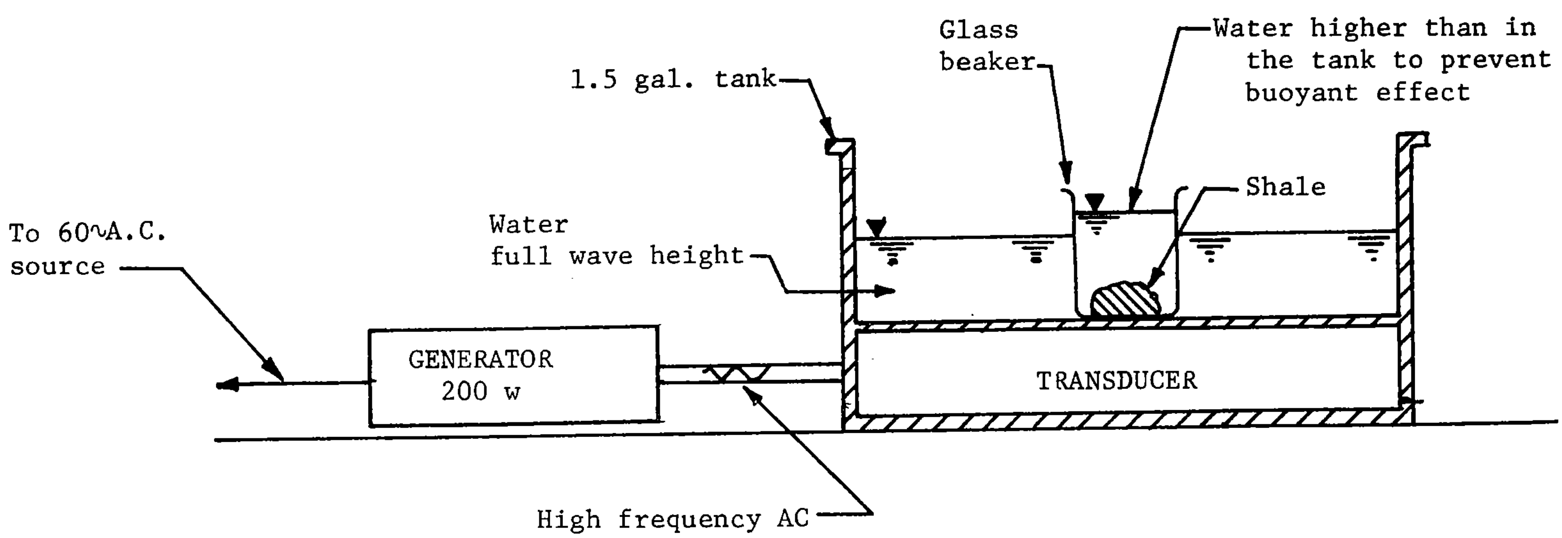

Figure 2. Schematic representation of the ultrasonic equipment.

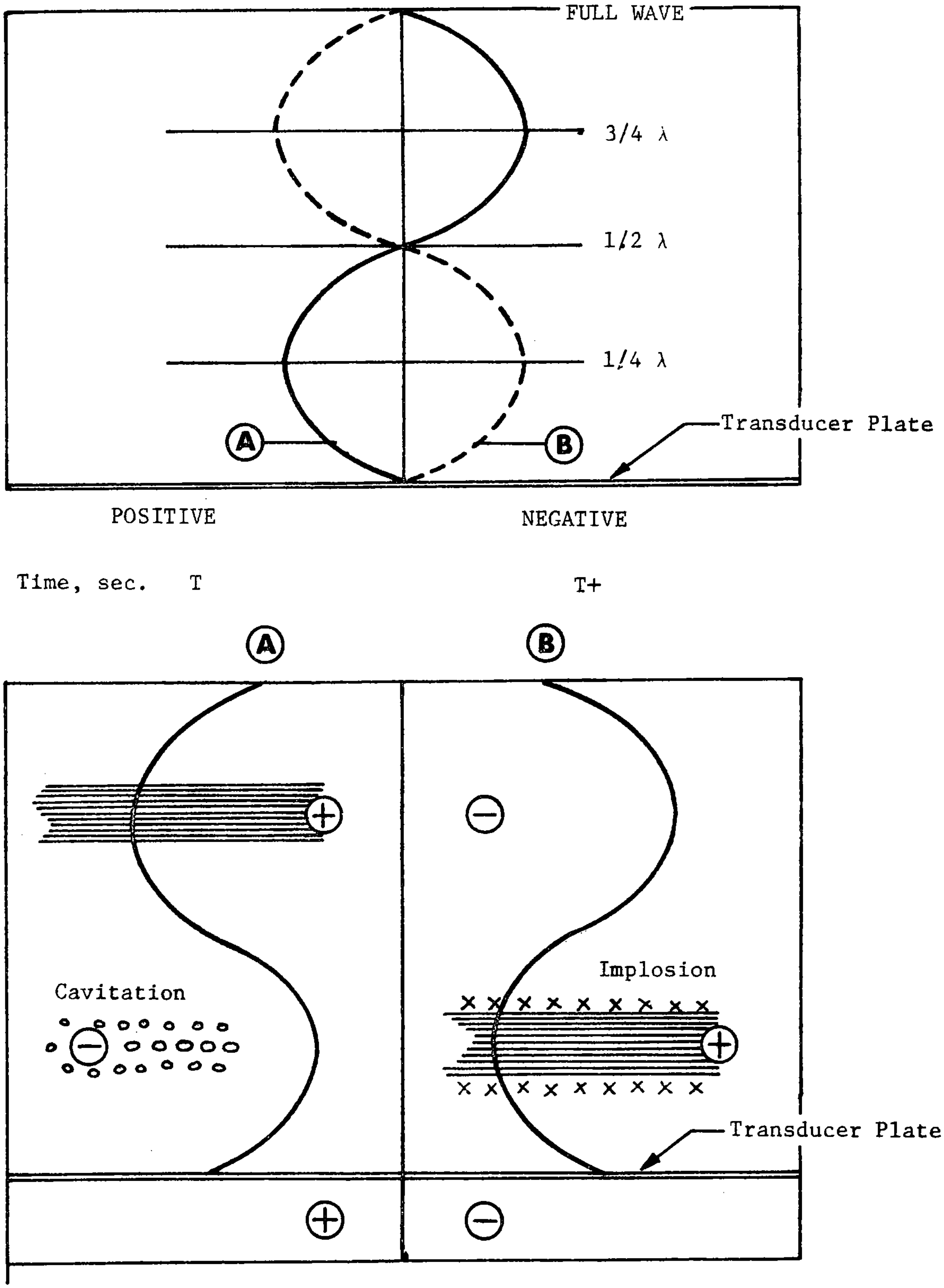

Figure 3. Standing-pressure wave.

Figure 4 is a flow chart depicting the methods and procedures used in this study to prepare samples for grain-size analysis and Atterberg-limit tests. An examination of the chart will indicate fundamental differences between the two methods of sample preparation.

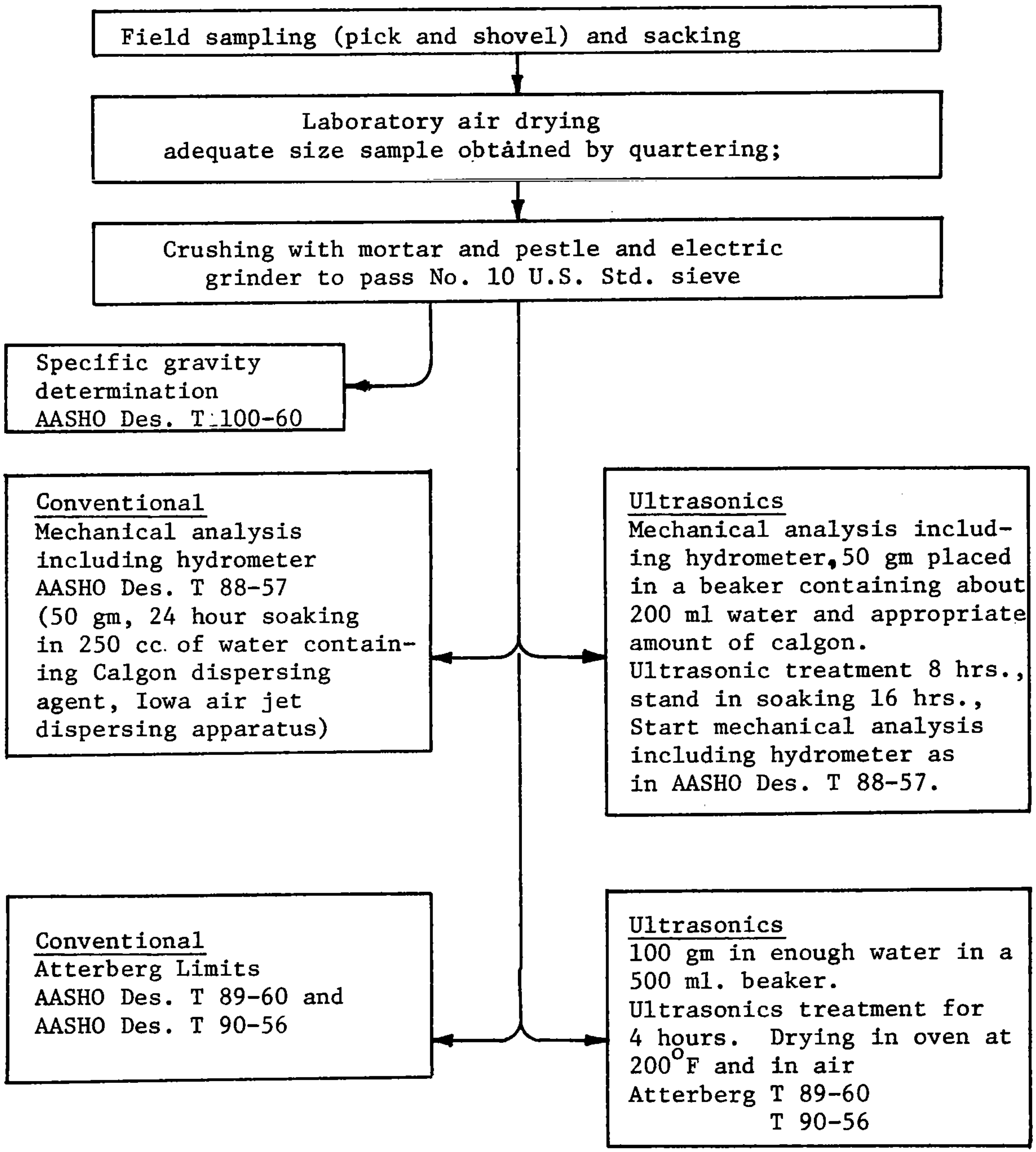

Figure 4. Flow chart for sample preparation.

Mechanical analysis.--By the conventional method the sample stands in water (fig. 4) and dispersing agent for 24 hours, but by the ultrasonic method the sample with the water and dispersing agent is subjected to 8 hours of ultrasonics and then it stands in water containing the dispersing agent for an additional 16 hours.

Atterberg limit tests.--By the conventional method, the procedure involves going from a dry to a wet condition. By the ultrasonics method, the sample of shale is put in a beaker with enough distilled water to prevent buoyancy effects (fig. 2). No dispersing agent is used. The sample is treated ultrasonically for four hours, during which it is frequently stirred. Then it is dried, first in an oven at 200°F until the mixture gets "soupy" and then in air. The liquid and plastic limits are determined as the sample loses water by evaporation.

Temperature measurements of the water bath and of the water in the beaker indicated values to 140°F. On the other hand, when temperatures neared 140°F, cold water was added to the tank. Admittedly, the temperature measured was the overall temperature and not the high temperature indicated in technical manuals (Westinghouse).

EXPERIMENTAL DATA AND DISCUSSION

Twenty-two Oklahoma shales are included in this general investigation. Presented herein are data from four shales used to demonstrate the effect of ultrasonic treatment. The pertinent properties of these shales are given in table 1, and their grain-size distribution curves are depicted in figures 5, 6, 7, and 8. The shales show increases in the 2-micron clay content and in the plasticity-index values following ultrasonic treatment. This may help explain the pavement failure experienced with shale no. 7 if it is theorized that the structural design of the pavement was based on the foundation material having 8 percent 2-micron clay content and a plasticity index of 2 while the environmental influences (traffic and weather) caused breakdown of the foundation material, thus increasing its 2-micron clay content and its plasticity index. These changes are manifestations of a weaker foundation material than that presumed during the design process. Ultrasonic treatment indicates that the 2-micron clay content increased to 20 percent and the plasticity index to 7. It would, of course, be conjecture to advance the opinion that precisely this same quantitative change had taken place in the field. Nevertheless, pavement failures tend to indicate that the shale underwent changes approaching test results. Similar reasons could hold true for the slope failure experienced with shale no. 23.

On the other hand, shale no. 8 contains small amounts of clay, and the variations introduced by ultrasonic treatment were not of the level that could affect the design; therefore failure has not been observed.

Field experience with shale no. 19 is extremely limited, and therefore the opportunity for observation and deduction has not yet arisen. However, based on data given in table 1, it may be estimated that this shale has a potential for trouble.

Table I. Properties of Shales

Sample			Nature of Trouble	Clay Minerals	2 micron clay, %		Liquid Limit, %		Plasticity Index	
No.	Location	Geologic Unit			C*	U*	C*	U*	C*	U*
7	Tulsa	Labette; dark-grey fissile	Pavement failure	Illite, chlorite	8	20	28	27	2	7
8	Mayes	Chattanooga; black, blocky-fissile	None	Illite, chlorite	2	14	22	37	4	8
19	Tillman	Hennessey; red, blocky	Not determined	Illite, chlorite	29	75	39	40	14	17
23	Coal	Boggy; grey, blocky	Slope failure	Illite, montmorillonite	34	49	37	37	12	18

*
C = Conventional
U = Ultrasonic

Since it appears that the extent of disintegration or the magnitude of shearing strength of shales depends on the geomorphological character and physicochemical properties of the particular shale tested, it follows that the amount of external energy required for shale disintegration is proportional to the degree of disintegration and might possibly be expressed in terms of "treatment or running time." Studies currently in process tend to support and quantify this conclusion, and figure 9 is presented as an example of the exploratory work in this area.

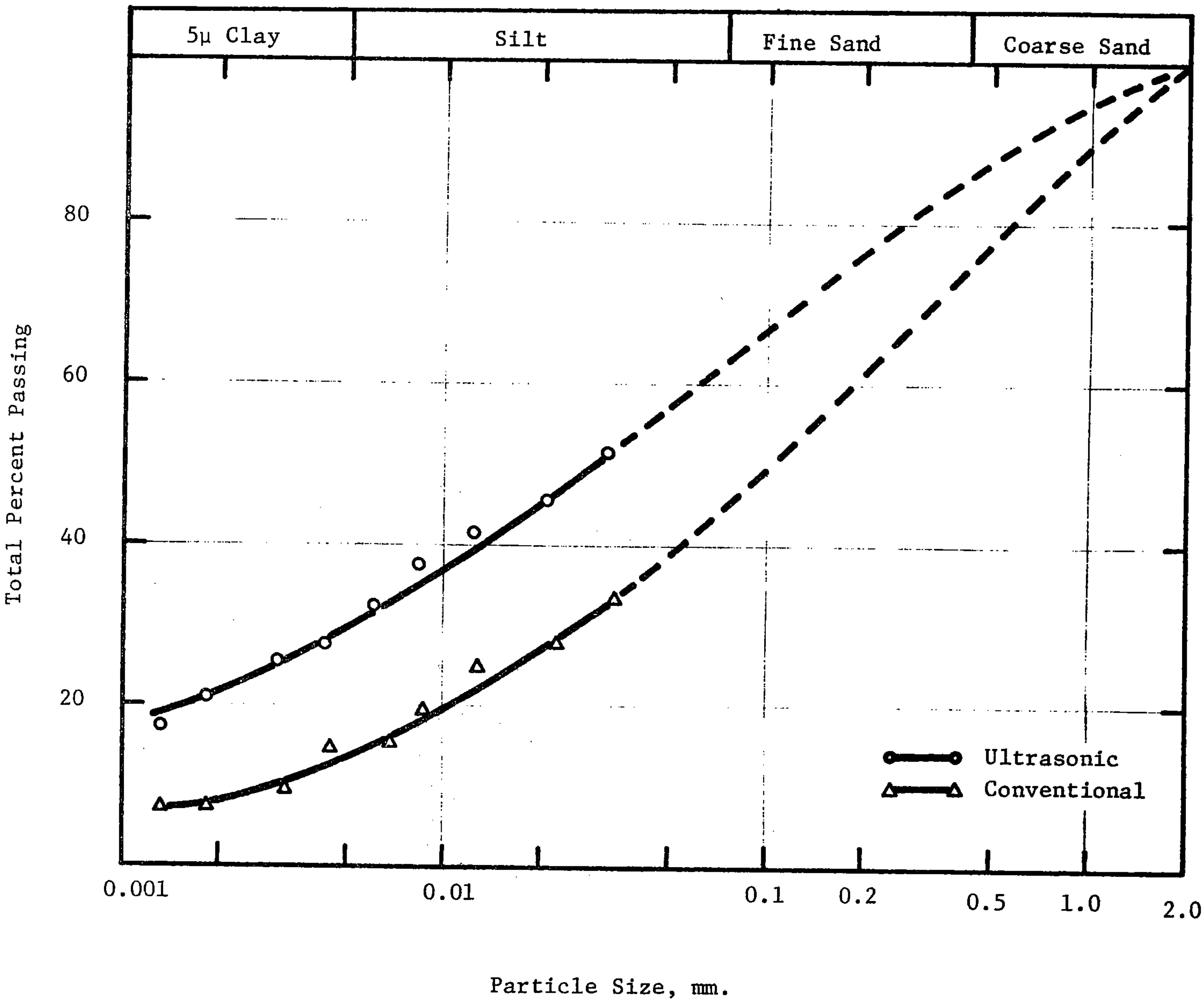

Figure 5. Grain-size distribution of shale no. 7.

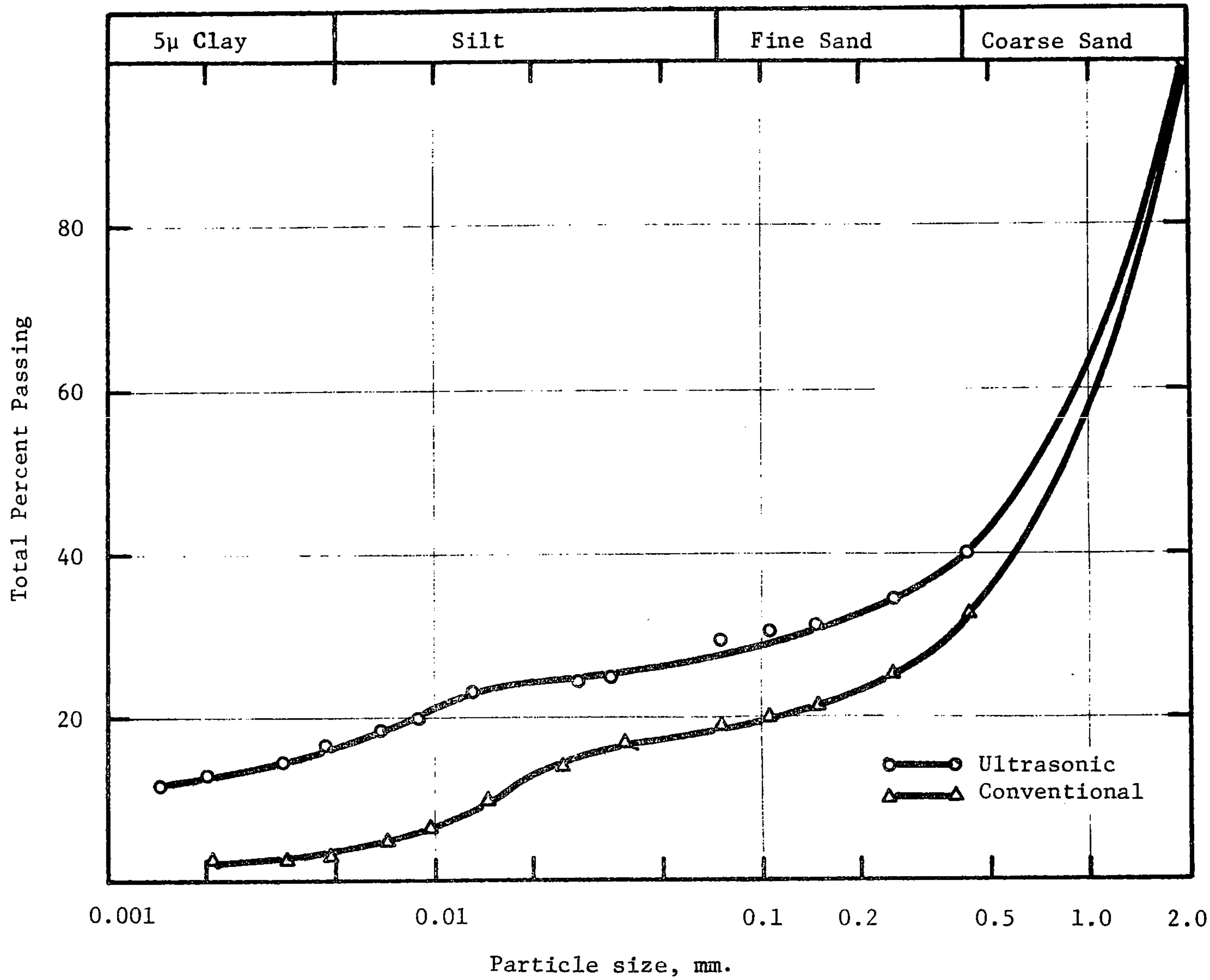

Figure 6. Grain-size distribution of shale no. 8.

STATISTICAL CONCEPTS

In measuring the shearing strength of a mass of shale, the principle of the statistical macroscopic equivalent is used. This suggests, at least, a random inhomogeneity at the microscopic level of material aggregation related to such characteristics of geometric position as voids, stacking faults, size, orientation, cohesive bonds. These deviations, in turn, give rise to variations in strength or durability properties.

Consequently, there seems to be a need to establish a probability model, which could be used to describe the phenomenon of shale disintegration in view of the random inhomogeneity. The analogy between shale disintegration and material fractures (Freudenthal, A.M.) appears to offer some merit, primarily because of the ability of the latter to explicitly treat the stochastic nature of the process. Therefore, it is tentatively proposed as an area of study which may provide a tool for predicting shale field performance.

Axiomatic to the attempt to accommodate disintegration and fracture in one domain is the acceptance that the probability of existence of weak spots of critical severity in a certain volume, V, of the shale is related to the

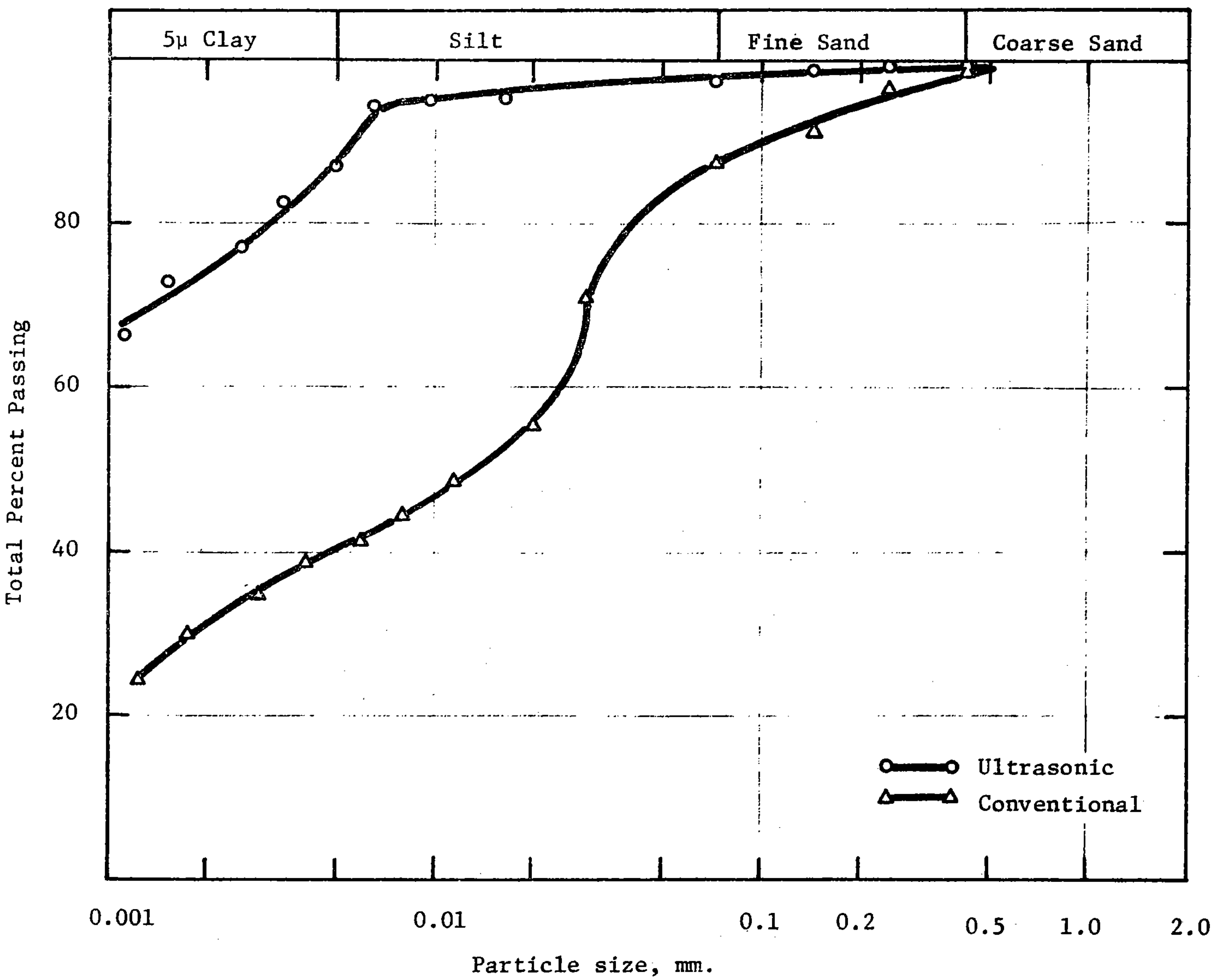

Figure 7. Grain-size distribution of shale no. 19.

magnitude of V. If P* (V) is the probability of nonoccurrence of the critical weak spot in volume V, and P* (V_1) that in volume V_1, then the probability of nonoccurrence in the volume ($V+V_1$) if given by (equation 3):

$$P^* (V+V_1) = P^* (V) \cdot P^* (V_1).$$

Differentiation with respect to V, gives (equation 4):

$$\frac{d}{dV} P^* (V+V_1) = P^* (V_1) \frac{d}{dv} P^* (V).$$

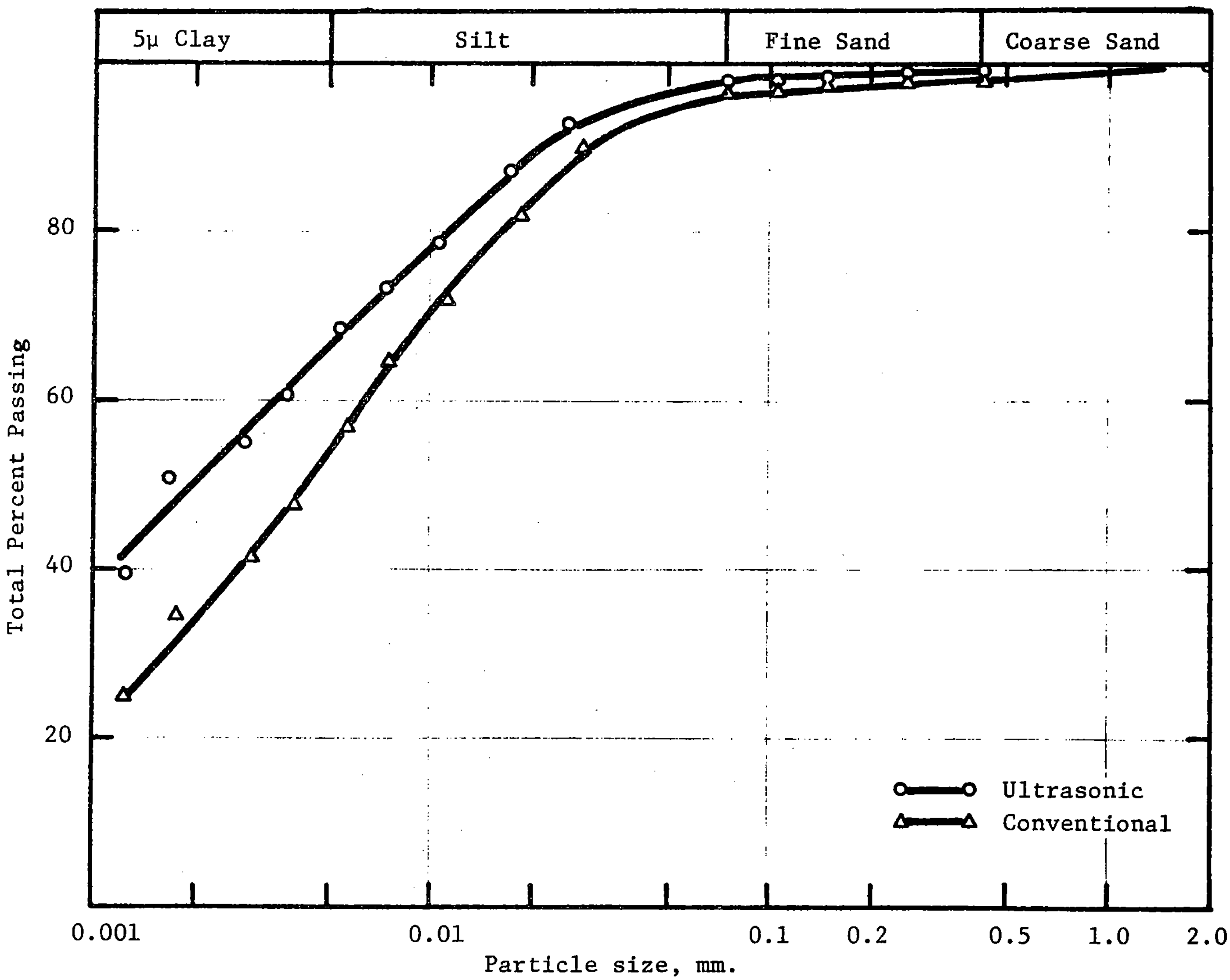

Figure 8. Grain-size distribution of shale no. 23.

Dividing equation 4 by equation 3, the expression obtained is (equation 5):

$$\frac{d}{dV} \ln P^* (V+V_1) = \frac{d}{dV} \ln P^* (V) = \text{constant}.$$

Since $P^* (0) = 1$ and $P^* (\infty) = 0$, equation 5 becomes (equation 6):

$$P^* (V) = e^{-cV}.$$

Equation 6 implies that the smaller V is, the more rapid the decrease of P^* (V) becomes. On the other hand, if the occurrence of a single weak spot of critical severity in the volume V is assumed to produce failure of this volume, then the probability of failure is (equation 7):

$$P_F (V) = 1 - P^* (V) = 1 - e^{-cV}.$$

Thus, for the same concentration of weak spots, the probability of failure increases rapidly with increasing volume; conversely, to ensure the same probability of failure for different volumes, the mean concentration of weak spots must be drastically reduced with increasing volume V.

Following a similar mathematical method of approach it is possible to show that the probability of failure not only increases with increasing volume but is also a function of volume and some stress ration (Popovics, 1967).

CONCLUSIONS

In actuality, pavement and slope design are not deterministic but stochastic processes; therefore, it becomes imperative to evaluate the risk involved in selecting design values. Since one of the elements of good design is a low-risk level, the ability to predict performance may be enhanced in two ways. First, if some material characterization can be made to reflect quantified parameters which are close to the true values and can also be used as design values. Second, if some mechanistic model can be formulated or adopted to describe the engineering performance of shale. The proposal to accomplish the first by employing the response of shale to ultrasonic treatment, and the second by drawing analogies with Griffith's fracture mechanism seems promising and rational.

ACKNOWLEDGMENTS

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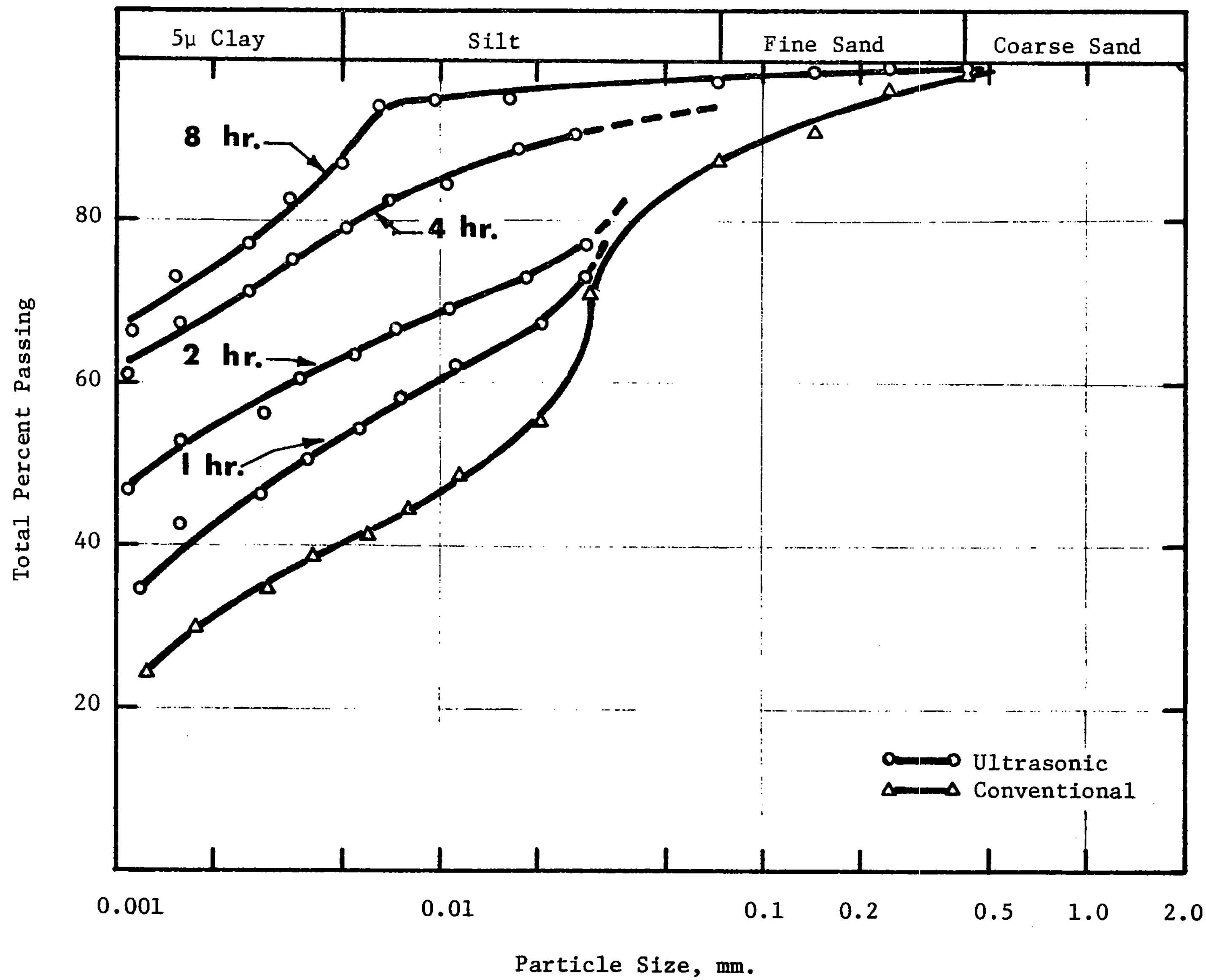

Figure 9. Effect of running time on grain-size distribution of shale no. 19.

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DURABILITY-PLASTICITY CLASSIFICATION OF SHALES AND INDURATED CLAY

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Abstract.--The wide variability in properties of materials labeled as shales, mudstones, and other argillaceous rocks requires a more detailed classification for engineering purposes. A classification based upon a slaking durability test and Atterberg limits is proposed. The slake durability index gives a relative measure of breakdown and weathering from drying and wetting effects. The plasticity index correlates generally with swelling pressures developed from shales and claystones submerged in water under a confined condition. The plasticity index is also an indicator of possible problems with low shear strength.

One might say that shales occupy a "gray area" between rock and soil. Shales as a group have a wide range of properties, some causing engineering problems while others do not. Shales range from soft bentonitic "clay shales," such as the Pierre and Bearpaw, which swell, slake, and stand in stable slopes with angles as low as 5° or 6°, to relatively hard shales that have been compacted and cemented almost to slate to hard siltstones with properties similar to sandstone to fine-grained cemented volcanic ash approaching volcanic tuffs.

Probably the best description of this whole group is: highly overconsolidated and fine-grained, composed predominantly of clay and (or) silt.

SHALE CLASSIFICATION

Much of the problem with shales for engineers and geologists lies in the lack of consistency in terminology. Often the same term is used for materials with quite different properties and behavior, or several different terms may be used for the same material.

Although the confusion may be perpetuated, for purposes of this discussion the following terms are used.

Shales or Mudrocks.--Used in a general sense, referring to the whole group of silty and clayey rocks.

Shale.--Used also in a specific sense, composed primarily of silt and clay, with fissility or a tendency to split along fairly close bedding planes. Modifiers for grain size and composition may be used, such as sandy, silty, clayey, carbonaceous, or calcareous.

Mudstone.--Composed primarily of clay and silt; massive; breaks in apparently random directions. Often with slickensides (or what the British call listric surfaces). Erosion of drill core is common because of softness, etc. The massive structure probably is a result of flocculated clays. Included in this type are the claystones, the indurated clays, the Pittsburgh and Clarksburg red beds of the Appalachian area, and many underclays or coal seatrocks. Where the material is definitely over 50 percent clay the terms claystone or overconsolidated clay might be used.

Siltstone.--Composed primarily of silt, massive; sometimes grades into fine sandstone; grittier and usually harder than mudstone.

"Clay Shale."--Overconsolidated montmorillonitic clay and silt, such as in the Cretaceous and Tertiary deposits of the Missouri River valley, e.g., Bearpaw, Pierre, and Fort Union Formations. This term is not very satisfactory because the deposits vary from fissile to massive, contain varying amounts of clay, often less than 50 percent, and vary considerably in hardness. It is suggested that the term be restricted, using montmorillonitic or high-plasticity clayey shale for fissile or thin-bedded materials, high plasticity mudstone or claystone for massive materials, or overconsolidated plastic clay or silt for the softer soil-like materials.

ENGINEERING PROBLEMS

Among the problems in the shales or mudrocks group are: low strength, high compressibility, swelling and heaving, slaking when exposed to air and water, and differential weathering. These problems are often compounded because of rapid changes in lithology both laterally and vertically in the argillaceous and associated rocks.

One striking example of differential weathering and rapid breakdown was near Charleston, West Virginia, where a thick Pennsylvanian sandstone overlies mudstone in roadcuts. Severe breakdown and erosion of the mudstone within a couple of years has undercut the sandstone, leaving large overhanging blocks of sandstone which eventually fall, depending upon strength, joint spacing, etc.

Philbrick (1959) reported that in 1942 in Aliquippa, Pennsylvania, 20 people were killed when a bus was crushed by a rock fall in conditions that were similar to these.

SLAKING DURABILITY

Wetting and drying effects as well as frost action and other weathering agents are responsible for the breakdown of the mudstones and shales. In order to measure the relative durability of these rocks to slaking from drying and wetting, several methods have been used by investigators. One of the more promising is a slaking machine and test devised by Franklin at Imperial College in London (Franklin and others, 1970).

The slaking machine is essentially a drum of 2-mm mesh screen 140 mm in diameter and 100 mm long, which is rotated about half submerged in a trough of water or other selected slaking fluid. For each test, 10 pieces of sample material 40 to 60 g each (about the size of walnuts) are oven-dried at 105°C to constant weight (usually 6 hours or more in these tests), placed in the drum, and weighed. The drum is placed in a trough, which is filled with water to 20 mm below the drum axis, and then rotated at 20 rpm for 10 minutes. After removal from the water, the remaining sample is oven-dried and weighed to find the percent of the original sample retained in the drum, which is termed the <u>slaking durability index</u> (I_D).

The 20 rpm rotation is slow enough to provide gentle rolling action without excessive abrasion. Chandra (1970) recommended preservation of cores or samples at natural moisture content until they are oven-dried for best reproducibility of results. He also recommended that water temperature be uniform for all tests (about 20°C or room temperature). The testing method is one of the "suggested methods" for rock testing issued by the Commission for Standardization of Laboratory and Field Tests of the International Society for Rock Mechanics. The apparatus is available commercially.[1]

The appearance of the pieces retained in the drum, as well as the passing material, should be noted briefly in the test records, as some samples may break down completely to pea-sized chunks or flakes but be retained on the 2 mm screen.

Results from recently completed tests in which 20 samples were run from 2 to 4 additional wetting and drying slaking cycles indicate that the percent retained after the second cycle of slaking is more sensitive to the breakdown of shales and other mudrocks than the value obtained from the first cycle. Rocks that show a durability of 95 to 100 percent retained in the first cycle show significant divergence of values with additional cycles. Beyond the second cycle the loss shows a nearly linear relationship with the number of additional cycles. Therefore, it is recommended that the original test be modified to include a second cycle of slaking. This requires running the remaining sample from the first cycle in the mesh drum in the water tank for another 10 minutes, then 4 to 6 hours of oven drying to obtain the dry weight remaining after the second slaking cycle. Where it is impractical to run the additional cycle, the approximate relationships as shown in figure 6 may be used. Other figures in this paper are plotted on the basis of the first cycle durability.

Slaking tests were run on over 120 different samples from about 38 different locations in the United States and other parts of the world (see table 1). Slaking durability is shown in figure 1. Volume was determined by mercury immersion method and weight on samples oven-dried at 105°C for 6 hours or more.

In figure 1, samples that are known to be montmorillonitic, tuffaceous, or highly carbonaceous are below average unit weight and are indicated by solid symbols. Most shales and siltstones fall into one broad band, with

[1] Seelec, Inc., Glen Ellyn, Illinois 60157.

Table 1. List of Samples and Test Results

TEST NO.	ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM	LOCATION	BORING NO.	DEPTH ft.	SLAKING DURABILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
2	SHALE	Grayish-green	Eau Claire Fm. Cambrian	Morris, Ill.	H-4	1785-1786	66.5	33.1	23.4	9.7	(2.3)	154		x
3	SHALE	Grayish-brown to grayish-green	" "	"	"	1782-1783	68.4	31.7	21.8	9.9	(2.6)	154		x
4	SHALE	Gray	" "	"	"	1800-1801	65.0	33.1	23.5	9.6	(2.3)	153		x
51	SHALE	Dark greenish-gray	" "	"	Drift	1820-1822	73.7	29.2	22.4	6.8	(5.2)	153		
32	SHALE (slickensides)	Grayish-red and grayish-green	Rome Fm. Cambrian	Watts Bar, Tenn.	P-65	98-99	30.8	18.8	19.6	N.P.	4.6	157		x
33	SHALE (slickensides)	Dark gray	" "	"	0-62	101-108	73.3	--	--	N.P.	2.6	160		
34	SHALE (slickensides)	Grayish-green and grayish-red	" "	"	P-65	104-105	88.1	--	--	N.P.	1.3	169		
13	SHALE	Greenish-gray	Upper Maquoketa Shale-Ordovician	Cook Co., Ill.	T-1	230	72.5	31.4	19.4	12.0	(0.7)	151		
14	SHALE	Brownish-gray	Lower " " "	"	"	417	93.5	30.2	20.3	9.9	(1.1)	150		
15	SHALE--silty	Dark brownish-gray	Upper " " "	White Co., Ill.	C-17	6189	99.3	--	--	--	(0.3)	168		
16	SHALE	Dark grayish-brown	Lower " " "	"	"	6344	99.5	--	--	--	(0.3)	169		
17	SHALE--silty	Light olive	Upper " " "	Livingston Co., Ill.	H-2	357	93.5	27.7	19.1	8.6*	(1.1)	167		
18	SHALE	Light gray	" " " "	"	"	367	51.5	31.6	23.6	8.0	(0.8)	150		
23	SHALE	Dark gray	Oswego Fm. Ordovician	Oswego, N.Y.	Tunnel	(?)	92.9	19.0	16.5	2.5	2.0	169		x
56	SHALE--sandy	Gray	" "	"	"	(?)	93.8	--	--	--	(0.7)	166		
94	SHALE--silty	Gray	Rose Hill Fm. Silurian	Bluefield, Va.	Tunnel	250±	56.2	25.1	18.9	6.2*	(2.9)	146		
95	SHALE	Light greenish-gray	Tuscarora Fm. Silurian	"	"	400±	63.4	47.6	30.3	17.3*	(8.8)	136		
1	SHALE	Brownish-black	New Albany Shale Devonian	New Albany, Ind.	B-21G	64-65	99.5	--	--	--	(0.4)	150		
24	SHALE	Greenish-gray	-- -- -- Mid-Devonian	Gilboa, N.Y.	Tunnel	(?)	99.6	--	--	--	(0.3)	170		
25	SHALE	Dark gray	-- -- -- "	"	"	(?)	99.5	17.8	14.4	3.4*	(0.3)	170		
55	SHALE	Dark gray	-- -- -- "	"	"	(?)	99.5	--	--	--	(0.9)	169		
59	SHALE--calcareous	Greenish-gray	Palestine Shale Mississippian	Cannelton, Ind.	Pier 2	15±	8.1	38.5	20.0	18.5	(10.1)	135		

Table 1 (Continued). List of Samples and Test Results

TEST NO.	ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM		LOCATION	BORING NO.	DEPTH ft.	SLAKING DURABILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
60	SHALE	Dark gray	Palestine Shale	Mississippian	Cannelton, Ind.	Pier 2	20±	5.7	43.8	23.7	20.1	(10.5)	133		
61	SHALE	Dark gray	"	"	"	"	"	3.6	45.2	24.8	20.4	(11.9)	132		
81	SHALE	Gray	-- -- --	Lower "	Kosmosdale, Ky.	(?)	108-122	85.7	39.4	23.7	15.7	(0.8)	156		x
82	SHALE	Gray	-- -- --	" "	"	(?)	108-122	90.8	35.4	23.1	12.3	(0.9)	155		
96	SILTSTONE--sandy	Grayish-red	Mauch Chunk Group	Mississippian	Davis, W. Va.	P-6	359-361	99.7	--	--	--	0.8	165		
97	SILTSTONE	Grayish-red	"	"	"	"	344-346	99.0	18.5	16.2	2.3	1.6	167		
98	SILTSTONE--sandy	Grayish-red	"	"	"	"	200-202	99.2	--	--	--	0.9	166		
8	SHALE--carbonaceous	Black	Allegheny Fm. (?)	Pennsylvanian	Dungannon, Ohio	Mine	30±	99.6	--	--	--	(2.7)	145		
52	SHALE--carbonaceous	Black	"	"	"	"	"	99.2	22.1	22.5	N.P.*	(2.4)	130		
53	SHALE--sandy	Gray to dark gray	"	"	"	"	"	99.0	--	--	--	(1.6)	160		
50	SHALE--clayey	Gray	Bond Fm.	Pennsylvanian	Fithian, Ill.		Outcrop	71.6	39.8	26.6	13.2	(6.2)	149		x
73	SHALE--clayey	Grayish-brown	Vamoosa Fm.	Pennsylvanian	Pawnee Co., Okla.	#10	Cut-slope	18.7	44.7	22.2	22.5	12.0	123		x
75	SHALE--clayey	Dark grayish-brown	Springer-Gottard Fm.	Pennsylvanian	Carter Co., Okla.	#22	Cut-slope	3.4	64.4	29.5	34.9	21.2	111		x
83	SHALE--silty	Gray	Bond Fm.	Pennsylvanian	La Salle, Ill.		Cut-slope	83.4	36.6	21.5	15.1	(1.2)	143		x
84	SHALE	Gray and maroon	"	"	"		Outcrop	51.7	40.3	20.3	20.0	(1.3)	139		x
85	MUDSTONE	Gray	"	"	"		"	57.5	41.3	19.8	21.5	(1.2)	138		x
77	SILTSTONE--clayey	Light gray	Caseyville Fm.	Pennsylvanian	Rock Island, Ill.	32A	27-28	66.3	(30.2	19.4	10.8)	7.1	145	6-62-32	x
68	SILTSTONE--sandy and clayey	Light gray	"	"	"	"	31-33	68.5	(30.8	19.1	11.7)	8.8	145		
72	SHALE--silty	Gray	"	"	"	"	40-41	86.7	(33.7	23.3	10.4)	7.7	146		
80	SHALE--coaly	Gray	"	"	"	"	54	73.6	(37.0	23.2	13.8)	9.6	134		
112	SHALE--clayey	Gray	"	"	"	33	28	35.0	(36.5	20.5	16.0)	11.2	133	6-63-31	x

Table 1 (Continued). List of Samples and Test Results

TEST NO.	ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM		LOCATION	BORING NO.	DEPTH ft.	SLAKING DURA-BILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
111	SHALE	Gray	Caseyville Fm.	Pennsylvanian	Rock Island, Ill.	33	30-31	83.0	(33.9	21.4	12.5)	7.9	147		
66	SHALE	Gray	"	"	"	34A	24-25	75.5	(40.8	24.3	16.5)	10.2	142	3-58-39	
115	SHALE	Gray	"	"	"	"	27-28	70.6	(37.8	22.9	14.9)	9.0	148	3-58-39	x
64	SHALE	Gray	"	"	"	"	34-36	74.1	(33.0	21.2	11.8)	8.9	150		
69	SHALE	Gray	"	"	"	"	36-38	81.5	33.6	21.8	11.8	8.9	149		
78	SHALE--silty	Gray	"	"	"	"	39-40	85.6	(31.9	20.8	11.1)	8.3	148		
113	SHALE	Gray	"	"	"	35	55	76.3	(35.0	20.8	14.8)	8.4	145	4-73-23	x
89	SHALE (nodules)	Dark gray	"	"	"	35A	31-32	(67.2)	(46.9	27.6	19.3)	13.2	142		
91	SHALE (nodules)	Dark gray	"	"	"	"	32-33	(35.6)	(47.5	27.5	20.0)	14.2	141		
87	SHALE	Dark gray	"	"	"	"	41-42	54.2	(47.7	28.8	18.9)	12.4	139	20-37-43	x
88	SHALE	Dark gray	"	"	"	"	42	46.8	(47.7	28.8	18.9)	12.4	142	20-37-43	x
65	SHALE	Dark gray	"	"	"	"	45-47	38.4	(46.4	28.4	18.0)	13.5	141		
86	SHALE	Gray	"	"	"	"	48-49	70.5	(38.1	22.8	15.3)	10.0	139	4-68-28	
90	MUDSTONE (underclay)	Light gray	"	"	"	36A	37-38	9.0	38.5	25.6	12.9	(9.3)	141		
29	SHALE	Black	"	"	"	"	41-42	37.6	38.5	26.3	12.2	11.5	135		
92	SHALE	Black	"	"	"	"	48-49	64.0	(41.1	25.5	15.6)	11.2	142	15-38-47	x
93	SHALE	Black	"	"	"	"	52-53	63.7	(40.7	24.5	16.2)	10.3	142	21-38-41	x
71	SHALE--clayey	Dark gray	"	"	"	37A	34-35	64.6	(41.5	21.8	19.7)	10.5	144	1-49-50	x
114	SILTSTONE--clayey	Light gray	"	"	"	"	40-41	67.2	(37.3	21.6	15.7)	8.6	152	10-62-28	x
79	SHALE	Dark gray to black	"	"	"	"	54-55	61.3	(43.5	25.1	18.4)	11.4	142	13-47-40	x
99	MUDSTONE--calcareous	Gray	Conemaugh Group	Pennsylvanian	Monroeville, Pa.		Outcrop	2.5	25.4	19.2	6.2	6.3	143		
100	MUDSTONE	Grayish-red and rusty-yellow	"	"	Pittsburgh, Pa.	B-9	14	0.4	32.0	21.5	10.5	10.0	129		
101	MUDSTONE	Grayish-red and gray mottled	"	"	"	"	52	0.1	30.0	20.5	9.5	6.6	151		

Table 1 (Continued). List of Samples and Test Results

TEST NO.	ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM		LOCATION	BORING NO.	DEPTH ft.	SLAKING DURABILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
102	MUDSTONE	Light gray, olive and grayish-red	Conemaugh Group	Pennsylvanian	Pittsburgh, Pa.	B-14	14	0.3	35.7	22.2	13.5	11.0	144		
38	MUDSTONE (nodules)	Mottled red and gray	Conemaugh Group(?)	Pennsylvanian	Wellston, Ohio	T-143	75-77	21.2	33.3	20.3	13.0	6.5	150		x
121	SHALE--silty and sandy	Greenish-gray	Allegheny Fm. (?)	"	"	"	183-184	96.0	35.5	19.7	15.8*	3.3	159		
126	MUDSTONE--shaly	Dark gray	"	"	"	"	195-196	19.1	48.7	22.2	26.2	7.3	151		
122	SHALE--silty	Gray	"	"	"	"	234-235	98.4	--	--	--	2.5	162		
123	SHALE	Dark gray to black	"	"	"	"	259-260	95.9	25.1	19.4	5.7*	3.4	159		
48	MUDSTONE (under-clay)	Dark gray to black	"	"	"	"	286-287	23.0	39.9	21.6	18.3	6.2	147		x
119	SILTSTONE--clayey	Mottled olive, red and gray	"	"	"	"	289-290	94.8	19.1	15.0	4.1*	2.7	160		
28	SHALE	Dark greenish-gray	Conemaugh Group(?)	"	"	T-196	109-112	95.0	31.2	21.7	9.5*	3.7	159		
124	SHALE--silty and sandy	Greenish-gray	"	"	"	T-233	41-42	97.5	--	--	--	3.1	164		
42	MUDSTONE (nodules)	Gray and red	"	"	"	"	53-54	40.3	46.3	22.3	24.0	9.1	141		x
49	SHALE--silty and sandy	Grayish-green	"	"	"	"	76-77	98.5	--	--	--	3.1	163		
105	SHALE	Greenish-gray	"	"	"	"	87-88	94.4	36.2	23.1	13.1	3.6	156		x
41	MUDSTONE	Greenish-gray	"	"	"	"	95-96	26.3	39.2	19.8	19.4	6.7	156		
36	SHALE--silty	Greenish-gray	Allegheny Fm. (?)	"	"	T-304	68-69	98.8	--	--	--	2.2	165		
117	SILTSTONE--sandy	Gray	"	"	"	"	143-144	70.3	24.6	13.2	11.4	(0.9)	160		
46	SHALE	Grayish-green	Conemaugh Group(?)	"	"	T-347	61-62	89.2	32.0	22.4	9.6	4.3	158		
39	MUDSTONE	Greenish-gray	"	"	"	"	74-75	6.6	35.2	19.4	15.8	8.5	143		

Table 1 (Continued). List of Samples and Test Results

TEST NO.	ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM		LOCATION	BORING NO.	DEPTH ft.	SLAKING DURABILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
125	SHALE--silty	Dark greenish-gray	Conemaugh Group(?)	Pennsylvanian	Wellston, Ohio	T-347	83-84	97.7	--	--	--	2.8	163		
118	SHALE--silty and sandy	Dark gray	Allegheny Fm.(?)	"	"	"	142-147	98.6	21.0	18.3	2.7*	1.8	159		
44	SHALE	Gray	"	"	"	"	220-221	98.0	--	--	--	2.4	162		
35	MUDSTONE (nodules)	Red and gray	"	"	"	"	224-225	71.4	26.7	19.3	7.4	3.9	162		
40	SHALE--clayey	Gray	"	"	"	"	266-267	85.3	29.5	22.3	7.2	3.7	160		
45	SHALE--silty	Grayish-red	Conemaugh Group(?)	"	"	T-492	72-73	96.0	28.9	20.5	8.4*	(2.2)	160		
43	MUDSTONE	Olive, gray and maroon mottled	"	"	"	"	168-169	26.7	30.6	19.8	10.8	3.9	155		
37	MUDSTONE (nodules)	Maroon and green mottled	"	"	"	"	190-191	79.5	35.8	21.0	14.8	5.1	153		
120	SHALE--silty	Gray	"	"	"	"	219-220	97.9	27.6	19.6	8.0*	2.6	160		
6	SILTSTONE--clayey	Grayish-red	Monongahela Group(?)	Pennsylvanian	St. Albans, W. Va.	H-403	17-19	79.5	33.5	25.5	8.0	(10.6)	155	21-65-14	
58	SILTSTONE--clayey	Grayish-red	"	"	"	H-406	32-34	94.6	25.4	20.7	4.7	(1.4)	165		
5	MUDSTONE	Red and gray	"	"	"	H-403	40-42	23.0	25.5	21.6	3.9	(6.7)	157	35-50-15	x
12	MUDSTONE (nodules)	Grayish-red	"	"	"	"	69-71	6.0	36.6	25.1	11.5	(4.7)	151		
9	MUDSTONE	Red and gray	"	"	"	H-404	13-16	4.9	47.1	25.5	21.6	(4.4)	153	4-62-34	
62	MUDSTONE (nodules)	Grayish-red	"	"	"	H-406	34-36	16.6	33.7	23.1	10.6	(2.3)	154		
11	MUDSTONE	Dark olive gray	"	"	"	H-402	42-44	0.2	31.8	24.4	7.4	(13.4)	138	61-24-15	
10	MUDSTONE (nodules)	Olive and dark gray mottled	"	"	"	H-403	63-66	1.8	35.0	27.2	7.8	(14.4)	135	59-30-11	x
110	MUDSTONE--shaly	Grayish-red and gray	"	"	"	H-404	36-40	30.1	31.4	22.4	9.0	(1.8)	149-162		x
31	SILTSTONE	Orangish-brown	Sundance Fm.	Triassic	Spearfish, S. Dak.	96-1	63-65	96.9	--	--	--	(1.1)	145		

Table 1 (Continued). List of Samples and Test Results

TEST NO.	ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM		LOCATION	BORING NO.	DEPTH ft.	SLAKING DURABILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
57	SHALE--silty	Grayish-red	-- -- --	Triassic	Hackensack, N.J.	DB-3	95-96	98.6	22.3	20.5	1.8*	(1.4)	160		
74	SHALE--clayey	Grayish-brown	Washita Fm.	Cretaceous	McCurtin Co., Okla.	#12	Cut-slope	4.4	71.0	30.7	40.3	32.2	91		x
7	SHALE--tuffaceous	Gray	Colorado Group(?)	Cretaceous	Elstow, Sask.	5-22	1563-1565	91.5	43.8	20.8	23.0	(2.4)	132		
47	SILTSTONE--tuffaceous	Gray	Dakota Sandstone	Cretaceous	Belle Fourche, S.Dak.	90-1	98-99	91.9	26.4	20.7	5.7	(0.7)	128		
106	SHALE	Dark gray	Mancos Shale	Cretaceous	Aspen, Colo.	TH-1	44-45	99.1	--	--	--	(2.1)	161		
107	SHALE	Dark gray	"	"	"	TH-2	50-51	98.5	21.9	16.0	5.9*	(1.9)	160		x
108	SHALE	Dark gray	"	"	"	TH-4A	69-70	99.2	--	--	--	(1.8)	161		
109	SHALE--silty	Dark gray	Colorado Group	Cretaceous	Ft. Benton, Mont.	DH-1	120-122	83.4	43.9	19.7	24.2	6.5	144		x
54	SHALE--clayey	Dark gray	Pierre Shale	Cretaceous	Chamberlain, S.Dak.	DH-1	260-262	Mud-ball	179.	32.	147.	25.2	102	∠2u=82	x
63	SHALE--silty	Dark gray	Bearpaw Shale	Cretaceous	Fergus Co., Mont.	DH-1	389-390	67.4	94.	19.	75.	10.5	130	∠2u=49	x
70	SHALE--clayey	Dark gray	Claggett Shale	Cretaceous	Fergus Co., Mont.	DH-1	494-496	Mud-ball	224.	32.	192.	14.8	115	∠2u=64	x
76	BENTONITIC SHALE	Light olive gray	Ft. Union Shale	Tertiary	T. R. Park, N. Dak.	DH-1	154-156	41.5	133.	22.	111.	23.4	106		x
27	BENTONITE	Light yellowish-gray	Plum Bentonite	Tertiary	Plum, Texas		Outcrop	(30±)	93.5	45.5	48.0	26.7	94		
19	SILTSTONE--tuffaceous	Greenish-gray	Nacimiento Fm.	Tertiary	Farmington, N. Mex.	113	98-99	98.0	--	--	--	(2.1)	150		
20	SHALE	Olive gray	"	"	"	"	103-104	61.2	40.3	28.2	12.1	(2.6)	151		x
21	SILTSTONE--tuffaceous	Maroon and olive gray	"	"	"	117	293-294	85.7	61.5	20.4	41.1	(3.7)	141		
22	SILTSTONE--tuffaceous	Olive gray	"	"	"		Outcrop	41.5	36.7	21.7	15.0	(4.1)	135		
103	SILTSTONE--clayey	Dark olive gray	-- -- --	Tertiary	Turkey	HW-6	(?)	89.8	41.4	27.4	14.0	11.7	129		x
104	MUDSTONE	Grayish-olive and bluish-gray	Cucaracha Shale	Tertiary	Panama Canal	WMS-1	180-182	0.0	54.9	37.3	17.6	15.2	117	∠2u=22	

Table 1 (Continued). List of Samples and Test Results

TEST NO.	* ROCK TYPE	ROCK COLOR	GEOLOGIC FORMATION AND SYSTEM		LOCATION	BORING NO.	DEPTH ft.	SLAKING DURABILITY INDEX I_D	LIQUID LIMIT L_w	PLASTIC LIMIT P_w	PLASTICITY INDEX P.I.	WATER CONTENT W%	DRY UNIT WEIGHT pcf	GRAIN SIZE % sand, silt, clay	CLAY MINERAL ANALYSIS
116	MUDSTONE	Olive and grayish-purple mottled	Cucaracha Shale	Tertiary	Panama Canal	WMS-1	29-31	0.2	44.3	33.5	10.8	15.8	120	∠2u=33	
30	VOLCANIC TUFF	Light yellowish-green	-- -- --	Tertiary(?)	N.T.S., Nevada		(?)	98.3	--	--	--	17.0	86		

NOTES:

1. Samples are arranged by geologic systems.
2. Under Rock Type, Mudstone includes materials often termed claystone, indurated clay or clay shale.
3. Slaking Durability Index, I_D, is % weight retained in 2 mm. mesh drum after one cycle of drying and wetting with rotation of drum for 10 minutes in de-ionized water.
4. Atterberg limits were run on material passing 2 mm. mesh in slaking test, except as noted below and have been subjected to two cycles of drying (at 105° C) and wetting.
5. * indicates Atterberg limits were run on crushed retained material in addition to the passing fraction from slaking tests.
6. ( ) Atterberg limits were obtained from other tests on some Rock Island, Ill., samples.
7. Water contents, in % of dry weight, are from samples prepared for slaking tests which had been wrapped or otherwise protected to preserve natural moisture content, but with varying degrees of effectiveness. Values in ( ) are from unprotected samples in which moisture content alteration was substantial, usually air-dried.
8. Grain size analyses are from samples adjacent to those for slaking tests.
9. x indicates clay mineral analysis was performed on adjacent sample.

durability increasing with increase in unit weight or compactness. The mudstones generally fill in another band with low durability. Two "clay-shale" samples formed mudballs during the slaking test and oozed through the mesh, giving an indeterminate durability.

Figure 2 contains a plot of water content versus slaking durability. Water contents shown are approximate natural water contents from wrapped samples prepared for the slaking tests by oven drying at 105°C for 6 hours or more. The samples were received from many different sources, and methods of moisture preservation varied considerably. Thus the values shown may be slightly low in some cases from drying in transport and sample preparation.

Generally the durability decreases with increase in natural water content. Again the mudstones appear in a separate band from the shales and siltstones.

PLASTICITY AND SLAKING DURABILITY

Mentioned before were other potential problems in the shales group such as swelling, heave, and slope stability. As a possible index, Atterberg limits were run on the material passing through the screen in the slaking test or on crushed retained samples when the durability was high. Most of the samples plotted slightly above and a few below the A-line on the plasticity chart in the low to medium plasticity range, with the montmorillonitic "clay shales" in the high-plasticity range above the A-line (fig. 3). These values should be used with some caution because of possible effects from two cycles of oven-drying and wetting. Two samples from the Panama Canal had significantly higher plastic limits under this treatment compared with normal air-drying techniques that resulted in rather low plasticity indices.

A proposed classification for argillaceous rocks based on plasticity index and slaking durability is shown in figure 4. The various rock types tested generally fall into the areas indicated, although there is some scatter.

Well-cemented siltstones and silty and sandy shales have a high durability and low plasticity generally. Shales with higher clay content are usually softer, have lower durability, and have medium plasticity. Durability decreases with weathering and also with sample defects such as slickensides and closely spaced joints. The mudstones are generally very low in durability, with varying degrees of plasticity. A small amount of active clay in tuffaceous siltstones and shales will give a medium to high plasticity, while they are well enough cemented to have fairly high durability. The montmorillonitic "clay shales" varied considerably in hardness and amount of clay present, with plasticity indices up to 200. The two most plastic samples formed the mud balls in the slaking drum. Durability of the clay shales probably depends on cementing, amount of silty and other materials, and probably the amount and type of the clay mineral montmorillonite in the sample.

Several swelling-pressure and swelling-strain tests were conducted on intact samples in consolidation cells submerged in deionized water, but no simple relationships were found between swelling test results and the plasticity index although the highest pressures and strains were developed in the

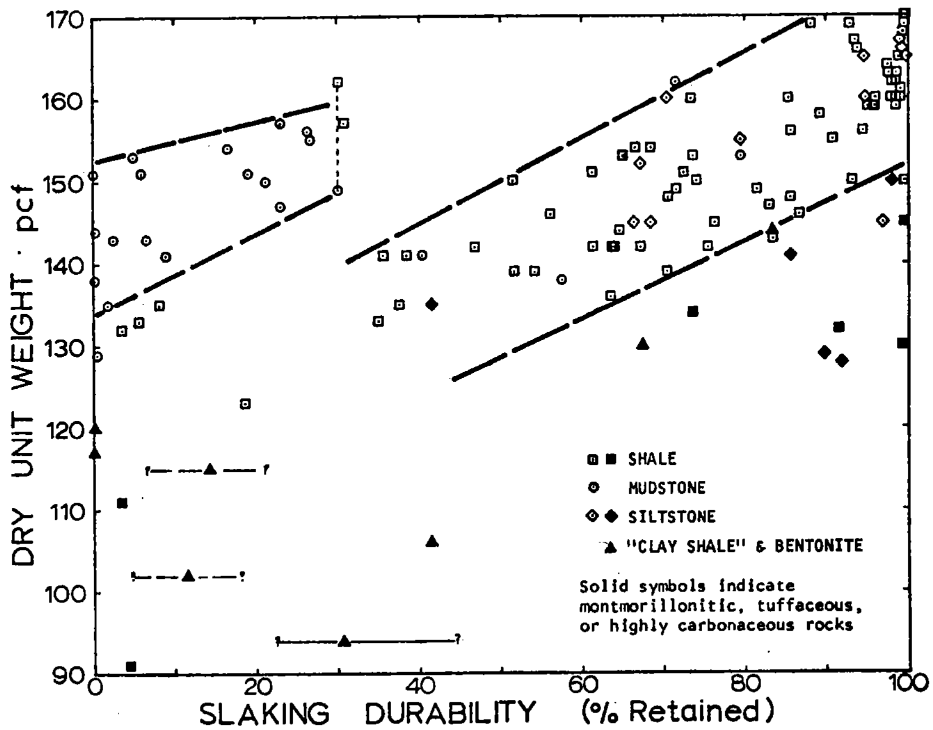

Figure 1. Dry unit weight vs. slaking durability.

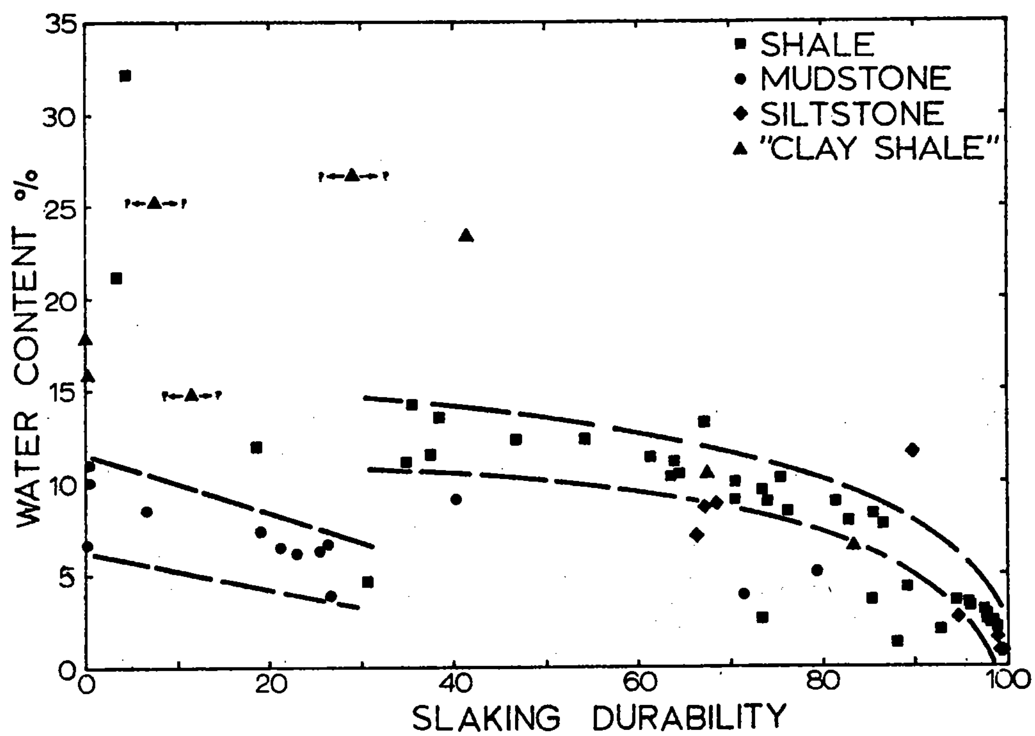

Figure 2. Water content vs. slaking durability.

most plastic "clay shales." It appears there are several factors controlling swelling in addition to the clay mineralogy and activity (which affect the plasticity index), such as cementing, degree of weathering or breakdown, depth of burial, and stress relief.

Slope stability problems are common in the high plasticity "clay shales" throughout the Missouri River basin. Slope failures and pavement heave have been observed in Oklahoma in partially weathered plastic shales. Numerous problems with slope failures have occurred in the Appalachian area in the red beds or mudstones. Both low durability and high plasticity appear to have a relation to slope stability. This is noteworthy because a high percentage of slope failures occur in the weathering zone.

CLAY MINERALOGY

Is there any relationship between these index properties and the mineralogy of the clays in the shales?

Clay-mineral analyses for about 40 of the samples were obtained from various sources. Although each investigator has his own method of analysis and reporting X-ray-diffraction results, a general pattern emerged from the information available, as shown in figure 5. The clay minerals are listed in general order of predominance for each area shown on the plot of plasticity index versus slaking durability. They appear to be arranged over the plot in relative order of clay activity. The plasticity index is greatly affected by the precent of montmorillonite present. The randomly interstratified clay minerals are primarily illite and vermiculite. The terms mixed layer and interlayered clay minerals and expandables are used by some investigators and include the randomly interstratified clay minerals and are used here as approximately equivalent.

Illite- and chlorite-rich shales are the most durable with low plasticity. These shales exhibit the best fissility along with those with carbonaceous matter and often showed good crystallization that may be a result of some diagenetic recrystallization in the clay minerals. None of the samples analyzed contained a high percentage of kaolinite.

CONCLUSION

The slaking durability and the Atterberg limits tests, especially when used together, have promise as index properties for shales and other argillaceous rocks. Both tests are fairly simple to perform without need for highly trained personnel. It is suggested that a classification based upon these index properties might be a useful way of looking at these geologic materials with such wide-ranging engineering properties and behavior. Also, unit weights and natural water contents promise to be useful index properties for shales, particularly for extension of limited testing information at one site or in a

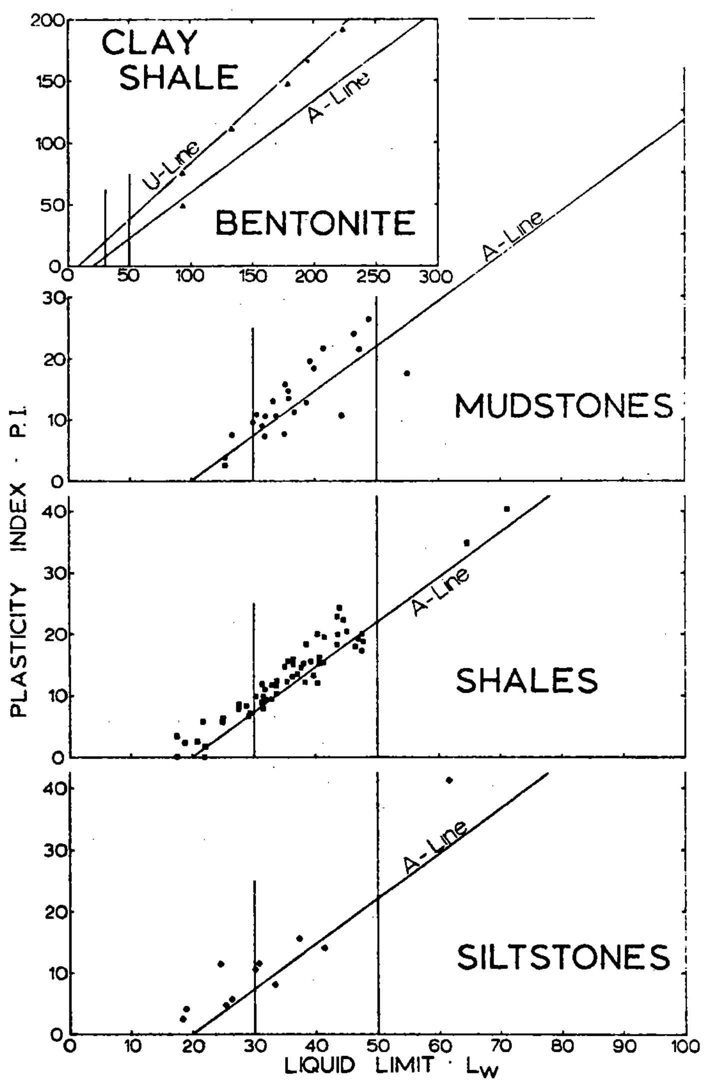

Figure 3. Plasticity charts.

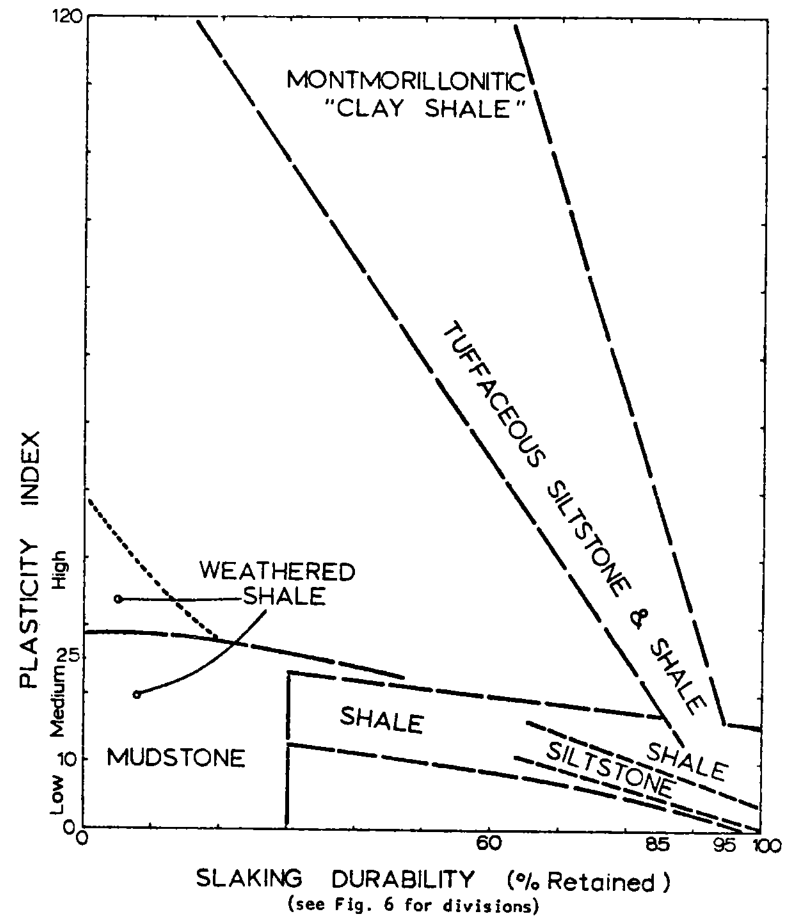

Figure 4. Plasticity index vs. slaking durability for rock types (generalized).

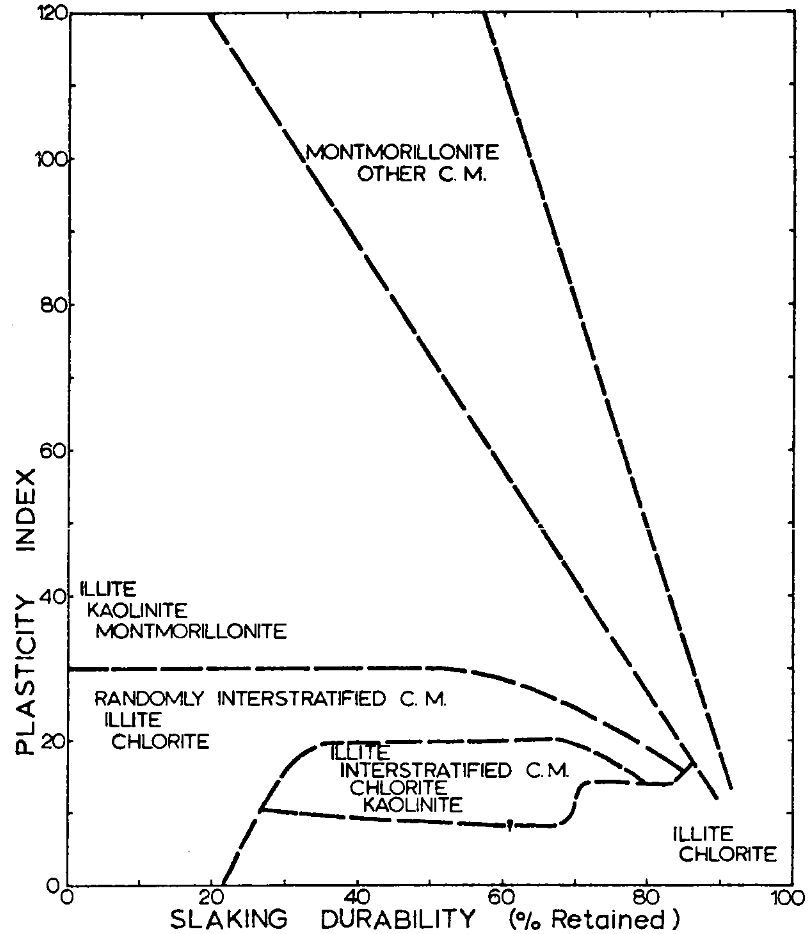

Figure 5. Prodominant clay minerals on plasticity index vs. slaking durability plot.

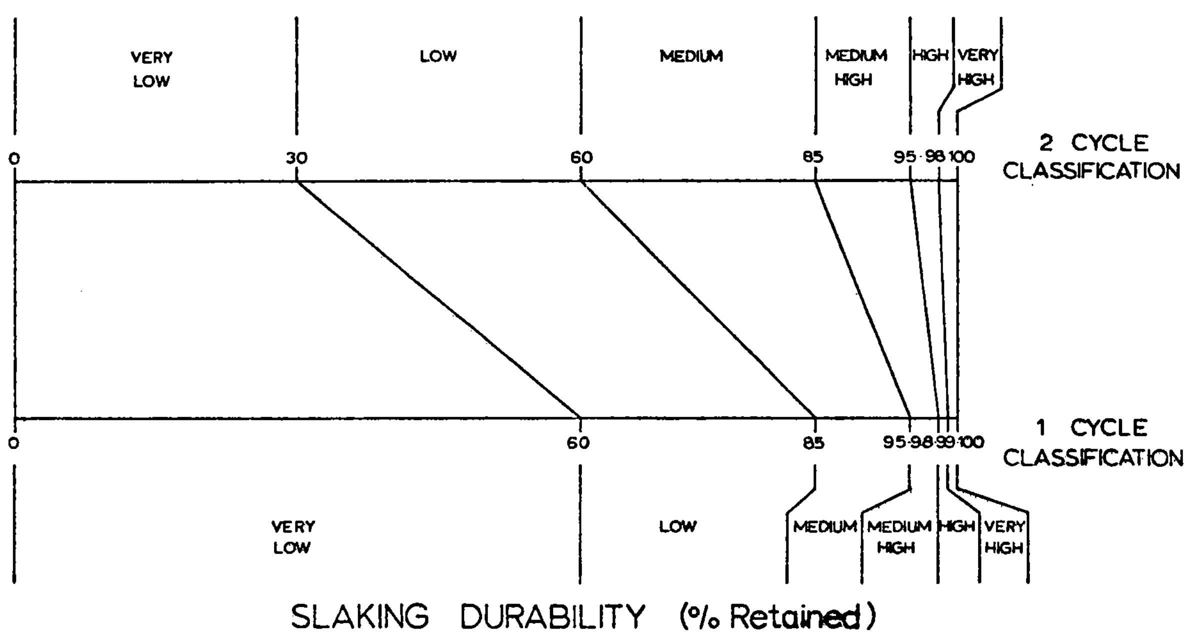

Figure 6. Slaking durability classification.

small area where the shales are similar (see Hendron and others, 1970; Gamble and others, 1970).

ACKNOWLEDGMENTS

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POST-CONSTRUCTION PERFORMANCE OF P-18 BRIDGE ABUTMENTS AND APPROACH FILLS, EASTERN OKLAHOMA

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Abstract.--The paper presents a case history of the post-construction performance of the Gaines Creek bridge abutments and their associated approach fills. It also describes the geologic, structural, and site features of this major lake crossing as well as the post-construction movements attributable to lake impoundment. Also discussed are the vertical and horizontal movements that resulted in significant damage to the abutment backwalls and the remedial measures necessary to repair the damage. Design or construction procedures to minimize the damage on future projects constructed in a similar environment are recommended.

INTRODUCTION

Eufaula Dam and Lake were authorized by the 1946 River and Harbor Act. The dam was Designed by the Tulsa District U.S. Army Corps of Engineers and was built between 1956 and 1964 under Corps supervision at a cost of some $122 million.

The dam is situated in eastern Oklahoma on the Canadian River about 27 miles above its confluence with the Arkansas River. The lake (fig. 1) extends up the Canadian River, into the valleys of the North Canadian River, Deep Fork River, and Gaines Creek, as well as minor tributaries, forming a very irregular shoreline in McIntosh, Haskell, Pittsburg, and Okmulgee Counties.

Construction of Pittsburg County road P-18 was one of the many major satellite programs undertaken as a part of the overall relocation plan for the Eufaula project. It was started in December 1960 and completed in August 1962 at a total cost of $1,700,000. Basically, the relocation consisted of the construction of 6.2 miles of 24-foot graveled roadway beginning southeast of Crowder and extending east across Gaines Creek and Eufaula Lake toward Blocker, as shown in figure 1.

The principal feature of the project was the construction of a 540-foot plate girder bridge spanning the Gaines Creek channel and its associated 65-foot-high approach fills. It is the post-construction performance of these fills and bridge abutments that provides the basis for this paper.

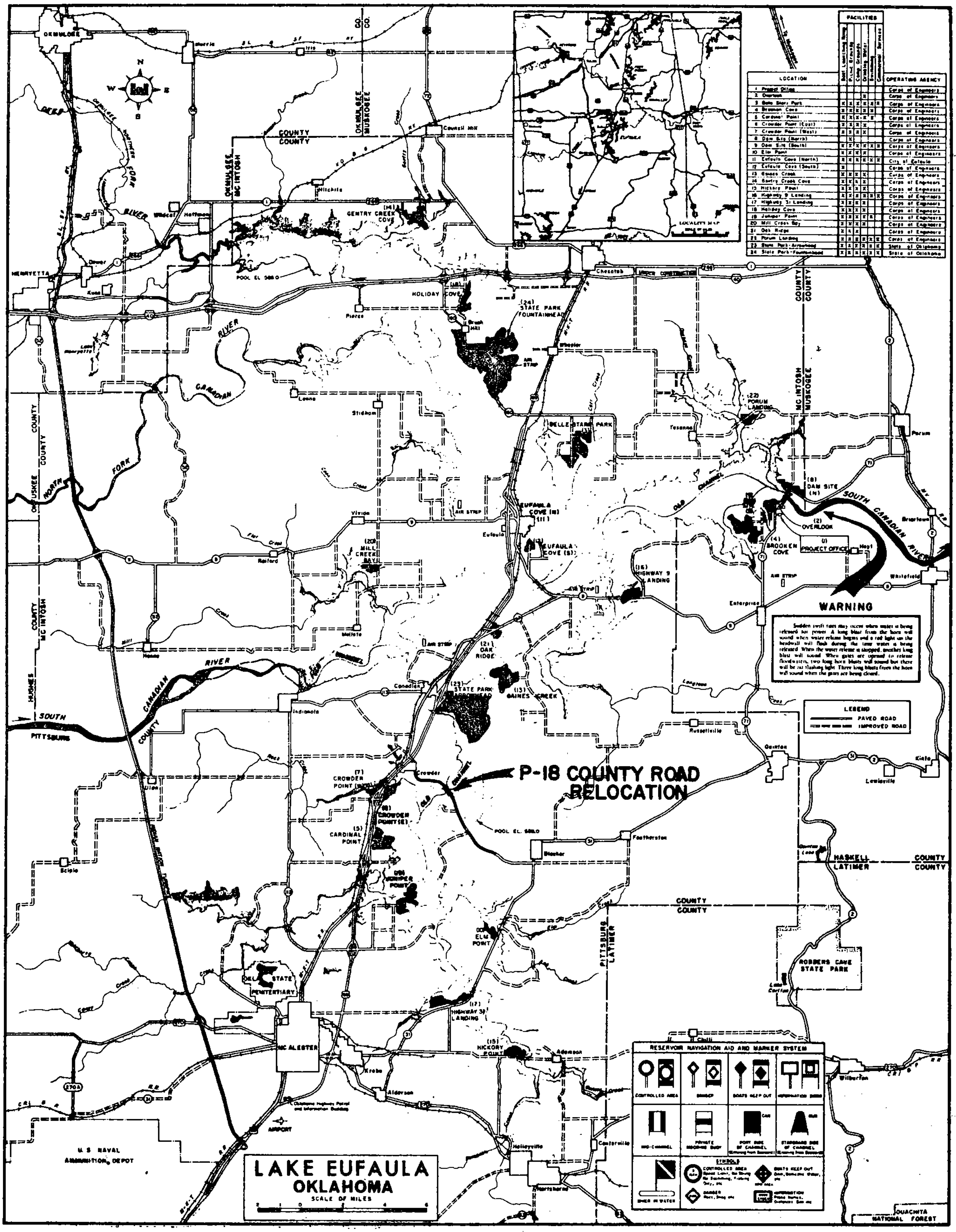


Figure 1. Vicinity map, P-18 county road relocation.

GEOLOGIC AND SUBSOIL CONDITIONS

The site topography is that of a broad, flat shale valley with moderately rugged hills and ridges capped with resistant sandstones.

The bedrock strata (fig. 2) are shales, sandstones, and siltstones of the Boggy Formation, with regional dips to the west. The sandstones vary from soft to hard, tan to gray, and from a few inches up to 40 feet in thickness of beds. The shales are mostly fissile, range from sandy to clay shale, are generally dark colored, and occur in zones with occasional thin stringers of sandstone and siltstone.

In the valley, the overlying flood-plain deposits (fig. 2) average about 40 feet in thickness and are composed principally of moderately plastic clays interspersed with permeable lenses of silty sands, gravels, and sandstone boulders. It is important to note that the preimpoundment water table existed at a depth of about 20 feet.

PROJECT FEATURES

The 65-foot-high approach or abutment fills were constructed of moderately plastic clays obtained from nearby flood-plain borrow areas. A typical section is shown in figure 2, and the as-built characteristics are summarized in Table 1.

Table 1. Results of Construction-Control Tests

	Abutment No. 1			Abutment No. 2		
Test	Max.	Min.	Avg.	Max.	Min.	Avg.
Procter, Standard AASHO						
Max. dry unit wt., pcf	116	90	103	113	84	98
Optimum water content, %	29	15	21	34	15	23
Field						
Dry unit wt., pcf	129	85	103	115	79	97
Water content, %	28	5	17	29	6	18

These data indicate that placement-moisture content averaged 4 to 5 percent below optimum for both abutment fills. Similarly the data indicate that both fills were compacted to relatively high densities, as the average was 99 to 100 percent of Standard AASHO.

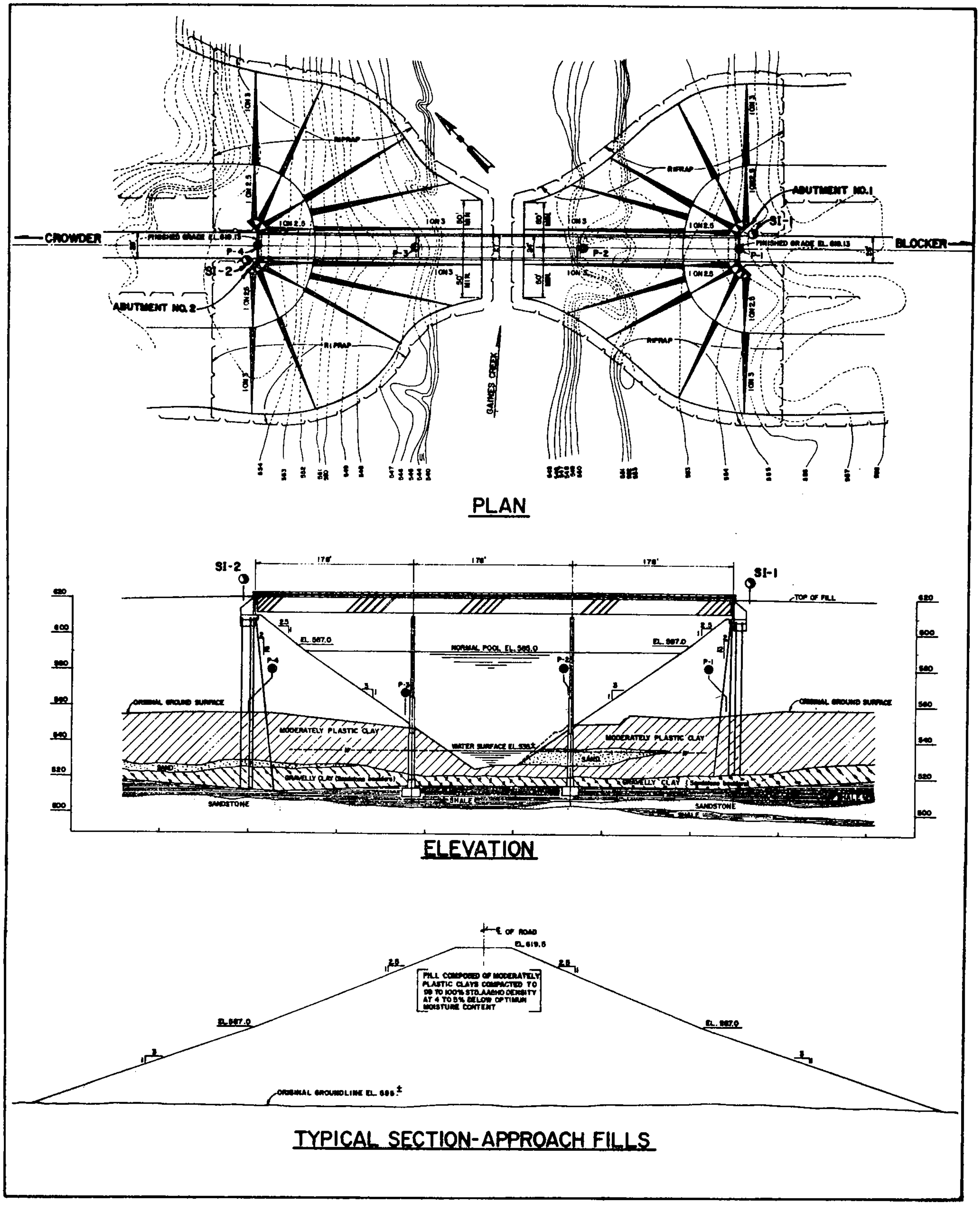


Figure 2. Plan and section, Gaines Creek bridge, P-18 relocation.

The bridge (fig. 2) consisted essentially of a 3-span, 540-foot-long, 27-foot-wide concrete deck constructed on a superstructure of four 7.5-foot-deep continuous plate girders. The 2 central piers were founded on firm, moderately hard shale of the Boggy Formation, while the abutments were supported by 13 H-beam piling (8 vertical and 5 battered) driven to rock for firm bearing in the overlying sediments. The as-built piling lengths for abutment no. 1 indicate that these piling refused short of firm rock. However, post-construction performance of both abutments has been quite similar, and this factor, though suspect, does not appear to have contributed to the overall problem.

POST-CONSTRUCTION HISTORY

Shortly after completion in August of 1962, the P-18 project was turned over to the Pittsburg County Commissioners. In the ensuing $5^{3}/4$ years there was no record of major problems until an inspection was requested by the County in March 1968. The inspection, made in April 1968, revealed the following:

1. The expansion gaps at each end of the structure were completely closed, and the plate girders as well as the concrete deck rested against the abutment backwalls, acting as struts. As shown by figures 3, 5, 6, considerable damage had occurred to each of the concrete backwalls. The position of the rocker shoes (fig. 3) and the unchanged elevation of the top of the backwall (same as as-built) indicate that both abutments had moved laterally toward the center of the structure without evidence of vertical displacement.

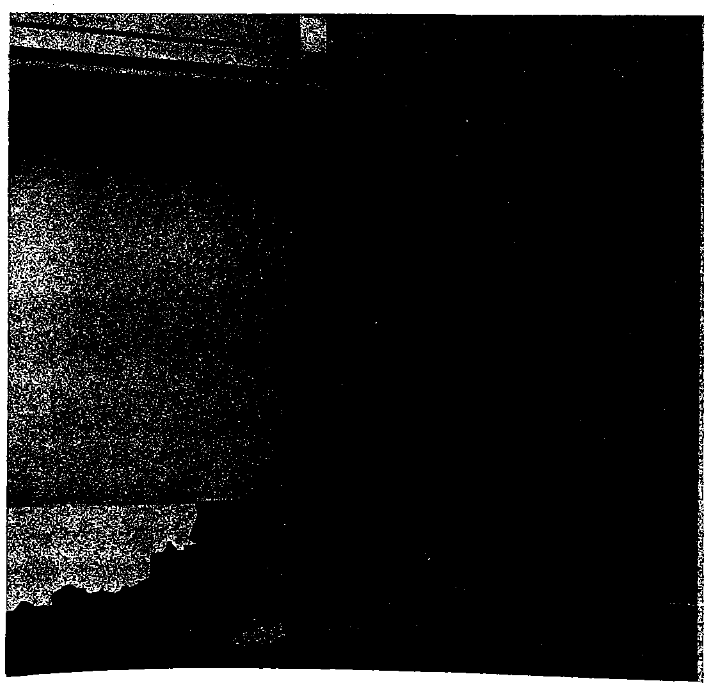


Figure 3. Plate girder restraining horizontal movement of abutment. Note position of rockers.

Figure 4. Resetting rocker shoes to vertical position.

2. The apparent subsidence of the approach fills was on the order of 5 inches at the top of the backwalls (fig. 6) and about 18 inches at the base of the backwalls (fig. 6). This difference of 13 inches is attributed to reshaping of the roadway by the county maintenance forces and a grading and surfacing program completed by the State Highway Department in the summer of 1967. Since there was no evidence of additional maintenance after completion of the surfacing project, it was concluded that some 13 inches of settlement had occurred prior to the summer of 1967 and 5 inches between this date and April 1968 (fig. 11).

Remedial repairs were effected in December 1968 and January 1969. These consisted of resetting the rocker shoes to a vertical position (fig. 4), cutting 3 inches off each end of the plate girders, removing and replacing the upper portion of the backwalls (figs. 7, 8), and patching the backwall concrete.

The structure was again inspected in July 1969, and at that time the backwalls had again moved horizontally (no vertical displacement) toward the center of the structure, all but closing the newly created expansion gaps. Fortunately, there was no damage, and repairs similar to those previously described were again made in October-December 1969.

An inspection of the structure in March 1971 showed that approximately 2 inches of opening remained to offset temperature expansion of the girders or future movements of the approach fills.

INSTRUMENTATION

Originally, it was theorized that the horizontal movements could be attributed either to normal spreading of the abutment fills (horizontal

A

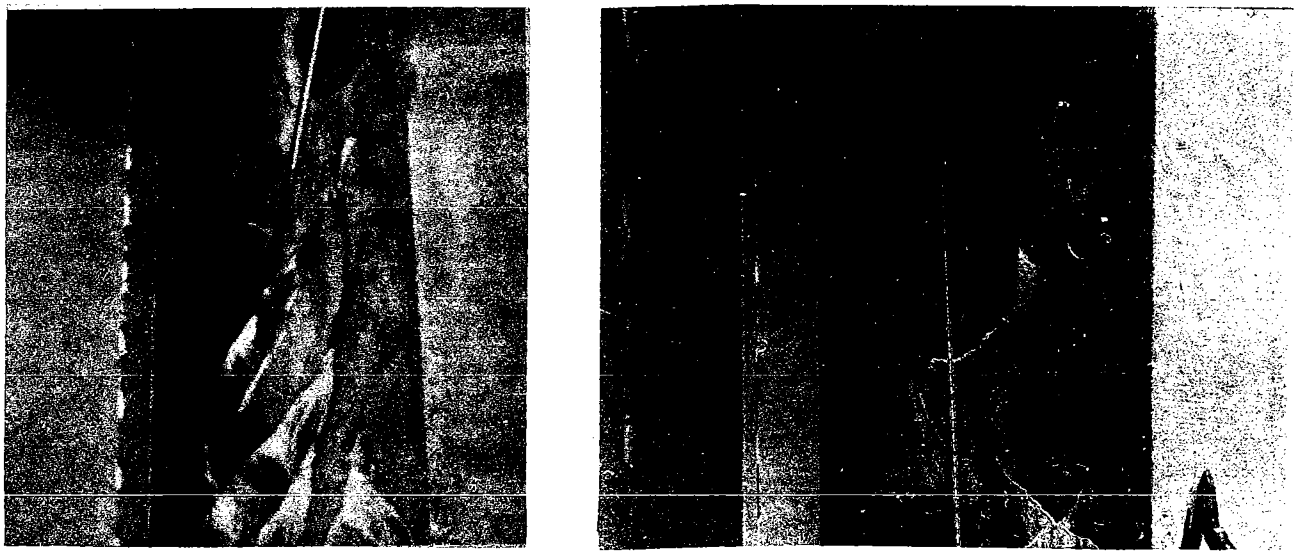


Figure 5. Damage to abutment backwalls. A is a photograph of abutment no. 1; B is abutment no. 2.

strains that generally accompany vertical subsidence) or to a creep type of deep-seated failure. The latter was a distinct possibility, since movement appeared to be toward the center of the bridge where the channel cut increases the slope heights from 65 to about 100 feet.

It soon became apparent that more precise information was required as to the nature of the movement, i.e., did a failure zone exist? where was it located? what was the rate of strain? etc. For this purpose, slope indicators SI-1 and SI-2 were installed at locations shown on figure 2 in August 1968. The data obtained from these instruments are summarized in figures 9 and 10. Basically, it indicates:

1. The rate of lateral movement has been gradual but fairly continuous since August 1968.

2. The direction of horizontal movement has been generally toward the center of the structure.

3. The magnitude of lateral movement has been much smaller than anticipated, as only 2.2 and 1.7 inches have been measured at the tops of SI-1 and 2, respectively.

4. The lateral movement decreases gradually from a maximum at the top of the casings to near zero at a depth of some 80 feet. Almost 100 percent of the continuing movements are occurring within the embankment fills.

5. Some 5.8 and 6.8 inches of settlement have been recorded since August 1968 at the tops of SI-1 and 2, respectively. It is important to note that basically all the continuing settlements are occurring at depths of 40 feet or less. This is a significant factor, since the 40-

foot depth roughly coincides with the water level of the reservoir and a corresponding change from partially saturated to saturated soils.

DISCUSSION

The limited field observations indicate that the movements are not confined to a single shear zone, thus eliminating the possibility of a localized shear failure, at least on a continuing basis. This suggests that the lateral movements are attributable to the horizontal strains that inevitably accompany consolidation. If this theory is valid, it raises the question of why major movements occurred several years after completion of the fills (fig. 11). This question can best be answered by comparing the embankment performance to a consolidation test sequence whereby a moderately

A

B

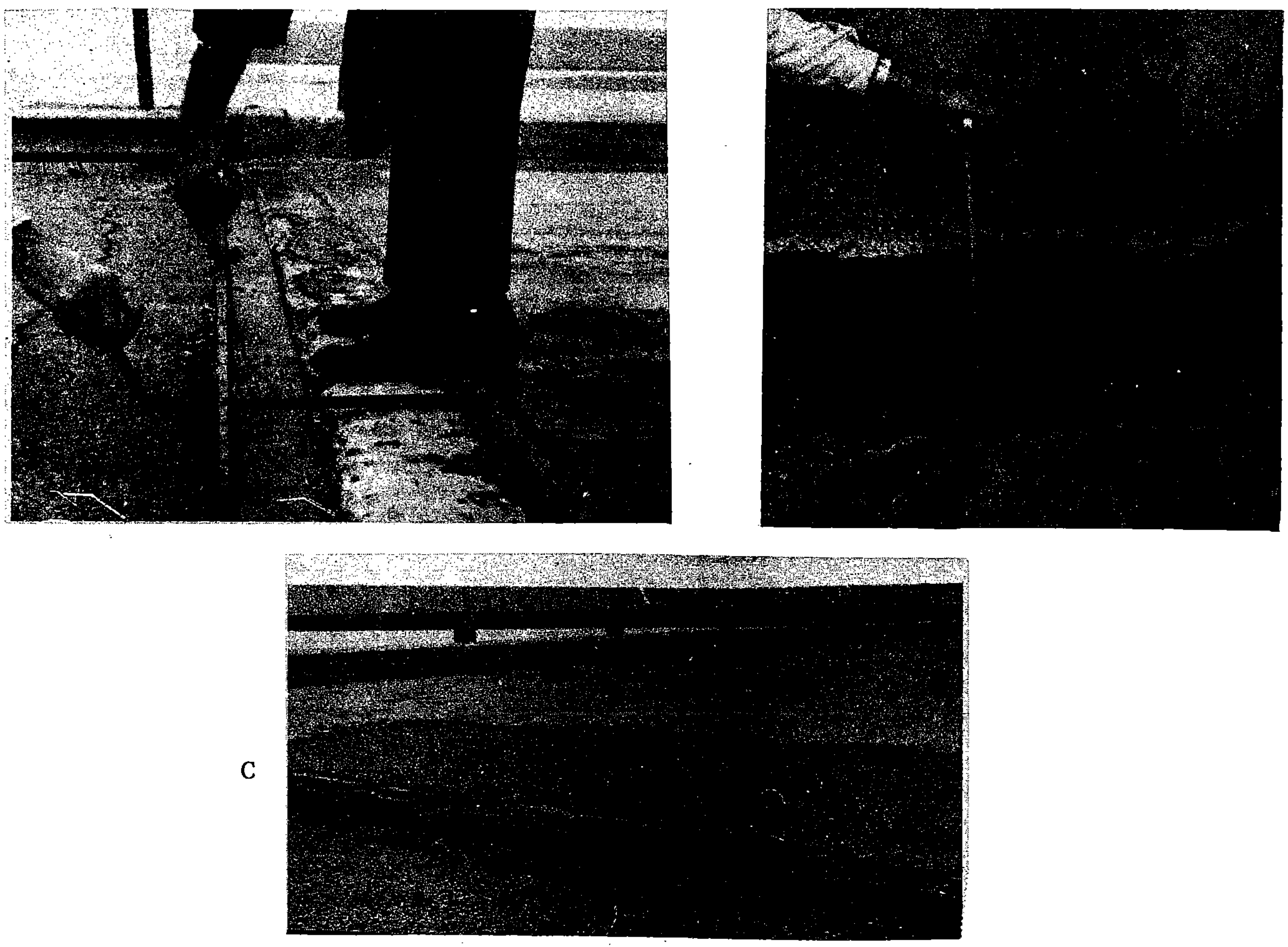


Figure 6. Evidence of settlement. A. Settlement of approach fill at abutment no. 1, about 5 inches. B. Settlement of approach fill at abutment no. 2, about 4 inches. C. Settlement at base of abutment seat, about 18 inches in April 1968. Note exposed piling.

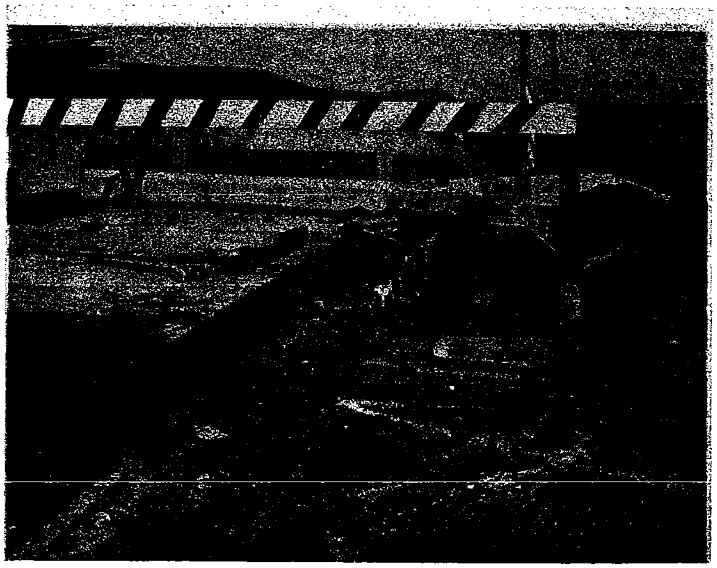


Figure 7. Removing top portion of backwalls to obtain clearance for concrete deck.

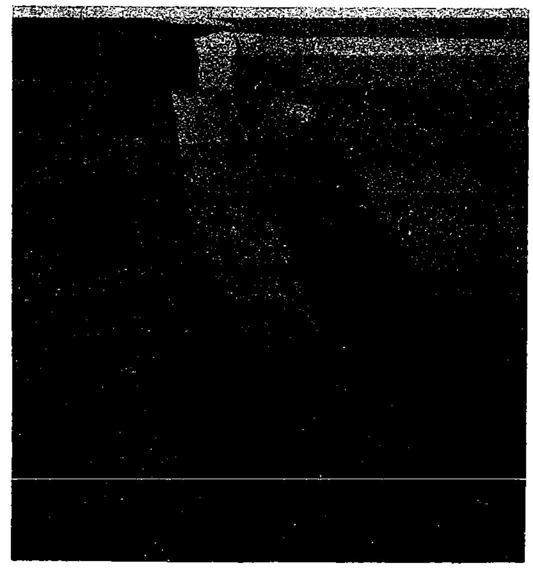


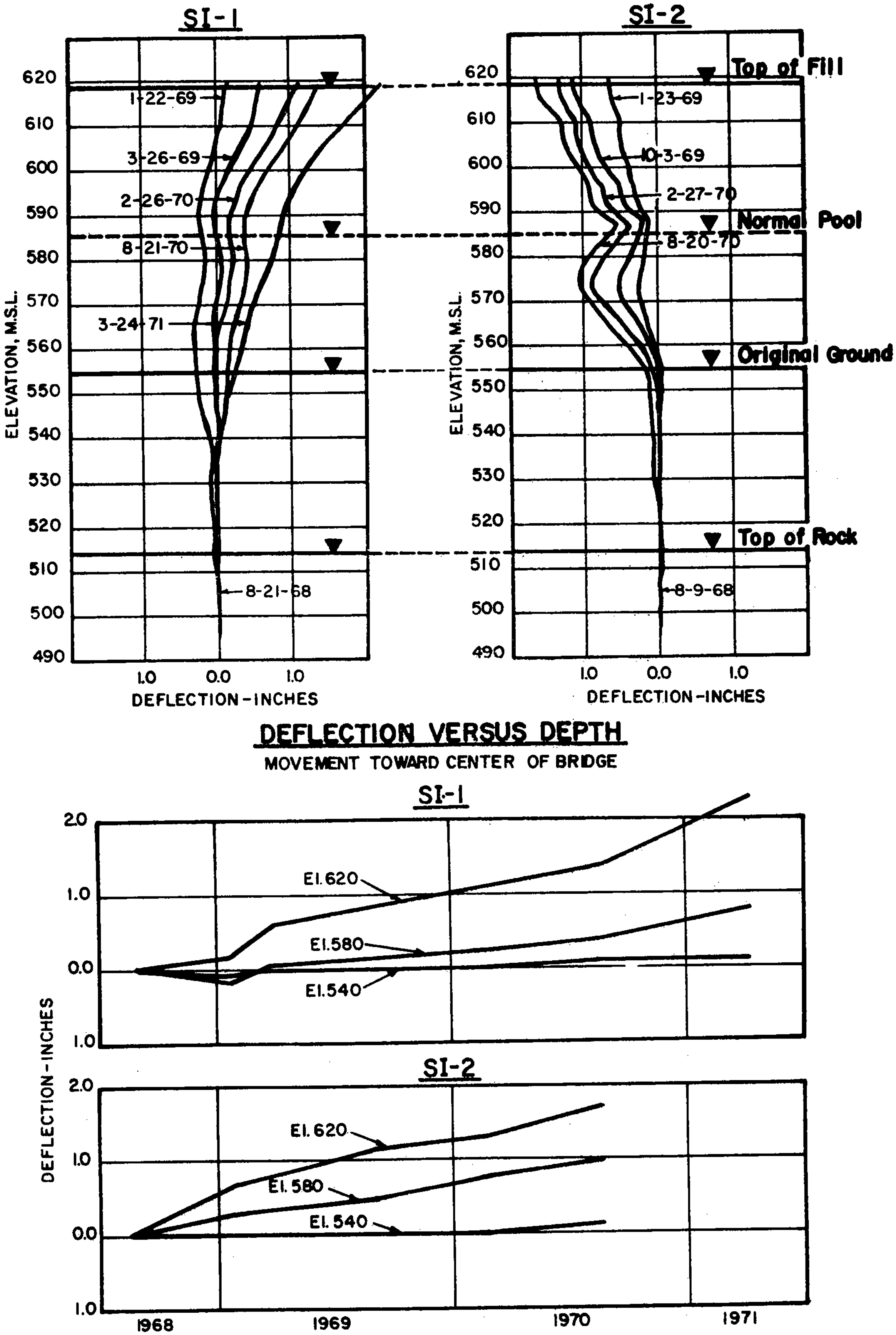
Figure 8. Completed backwall prior to resurfacing. Position of guardrail indicates settlement of fill had been approximately 18 inches.

plastic clay with a water content extremely dry relative to optimum is molded to a relatively high dry density (partially saturated soil mass) and then allowed to consolidate until equilibrium is reached under moderate to high confining pressures prior to saturation. Such a sample will generally undergo significant additional volume changes upon saturation, with no increase in the confining pressures. This phenomenon is often termed "delayed consolidation (owing to saturation)" or "saturation collapse" and is a result of strain softening or structural collapse of a partially saturated, randomly oriented soil structure as it becomes saturated. The event is discussed in detail in the literature.

The delayed-consolidation or saturation-collapse phenomenon can be related to the events which occurred at P-18 as follows:

1. The approach fills were constructed of moderately plastic clays compacted 4 to 5 percent dry of optimum. This placement condition normally results in an end-product mass with a relatively high void ratio and a low degree of saturation. Add approximately the top 20 feet of the foundation soils (assumed to be partially saturated, as the preimpoundment water table existed at about this depth) to this mass, and the result is some 85 feet of high-void ratio, partially saturated soils that could be susceptible to additional volume change if and when saturation occurs.

2. Reservoir impoundment began about 2½ years after completion of the fills. If an additional time factor is allowed for filling of the reservoir and subsequent imbibing of water by the plastic soil mass, one can reason that primary consolidation as a result of construction loading was essentially complete before introduction of the saturation water. This sequence (saturation



RATE OF HORIZONTAL MOVEMENT OF SLOPE INDICATOR CASING

Figure 9.

after consolidation as a result of construction loading) alters the consolidation equilibrium of the soil mass and provides the final conditions necessary to initiate the saturation-collapse phenomenon.

3. Subsidence would have been almost insignificant at first but would have increased in magnitude as the saturation zone progressed toward the center of the fills. This tends to account for the significant time factor between initiation of the action in 1964 (reservoir impoundment) and its reflection in major structural subsidence in 1967 and 1968. The 1967-68 period is inferred, since the county and state highway departments failed to note the structural damage when the roadway was surfaced in the summer of 1967.

4. The current measurements (figs. 9, 10) indicate that the movements are continuing but are confined basically to an elevation at or above the normal reservoir pool. Thus, it can be concluded that settlement because of saturation collapse is continuing, but at a decreasing rate. This is to be anticipated, as the saturation level changes with fluctuations of the reservoir.

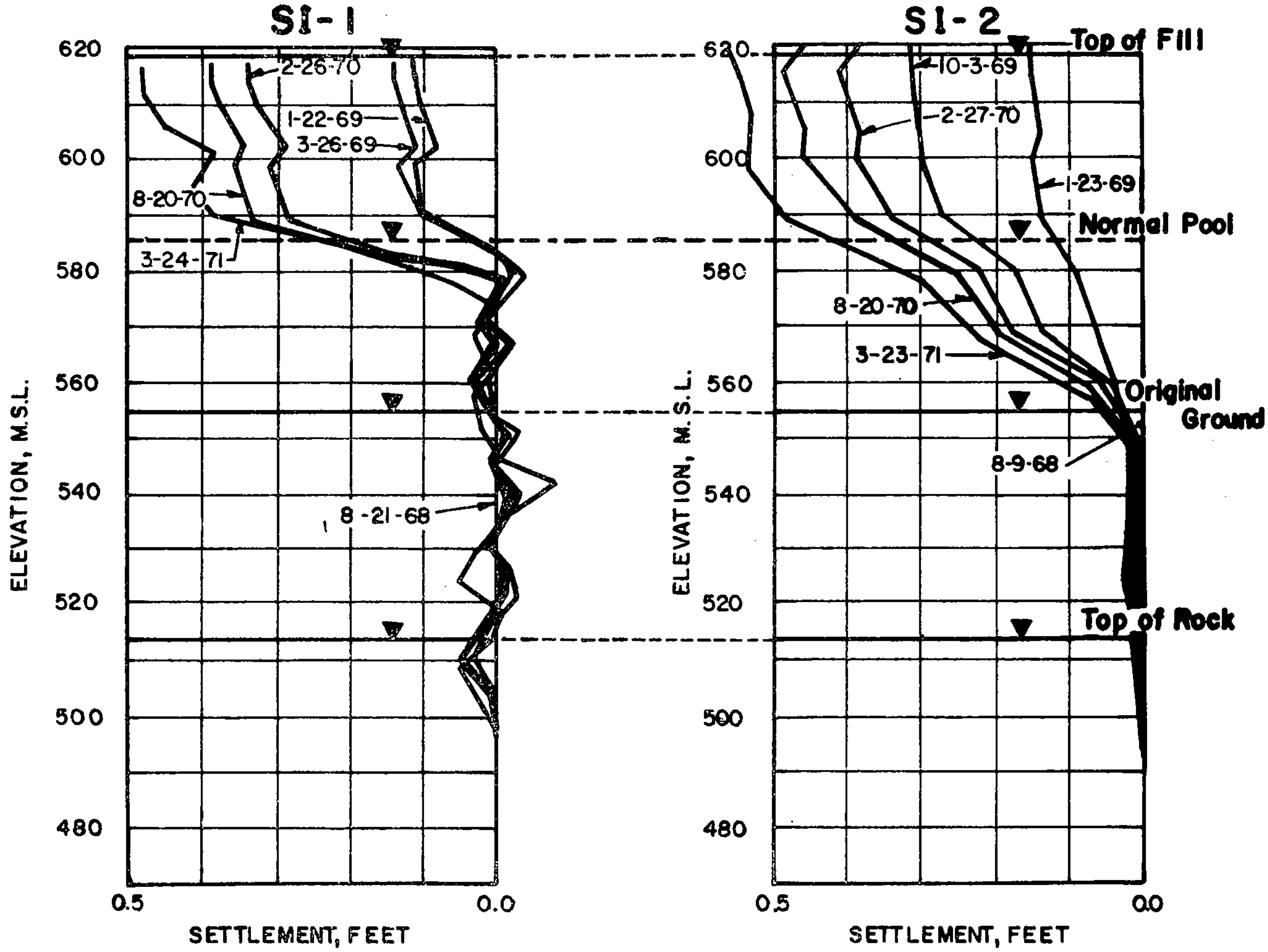
RECOMMENDATIONS

To minimize post-construction movement that may accompany saturation, it is recommended that one or more of the following alternatives be adopted during the design stage:

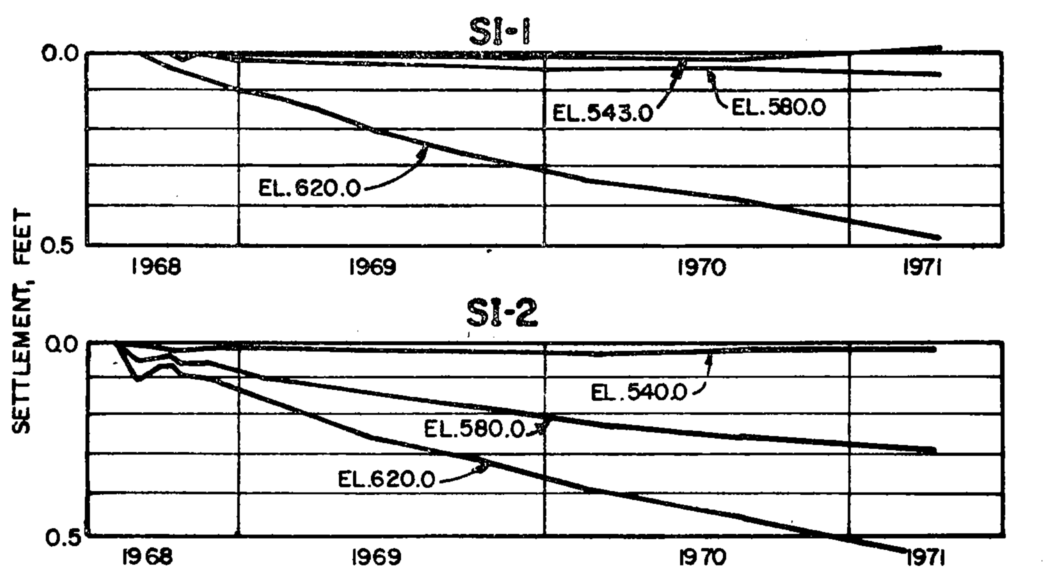
1. Specify minimum placement moisture as well as density control for the embankment materials. A simple series of tests similar to those developed for the soil will aid in determining the minimum acceptable placement moisture. Holtz (1948) described a suitable procedure. As a rule of thumb, a lower moisture limit of 3 percent dry of optimum (for individual tests) will generally provide satisfactory results.

2. Require periodic inspection of the structure both during and after reservoir impoundment so as to identify the problem prior to occurrence of significant damage. Cutoffs and regrading of the approach fills can be made as necessary.

3. Provide for lateral movement of the abutment with larger expansion openings between the deck and the backwalls and larger bridge seats and rocker shoes that can be repositioned with minimal effort.

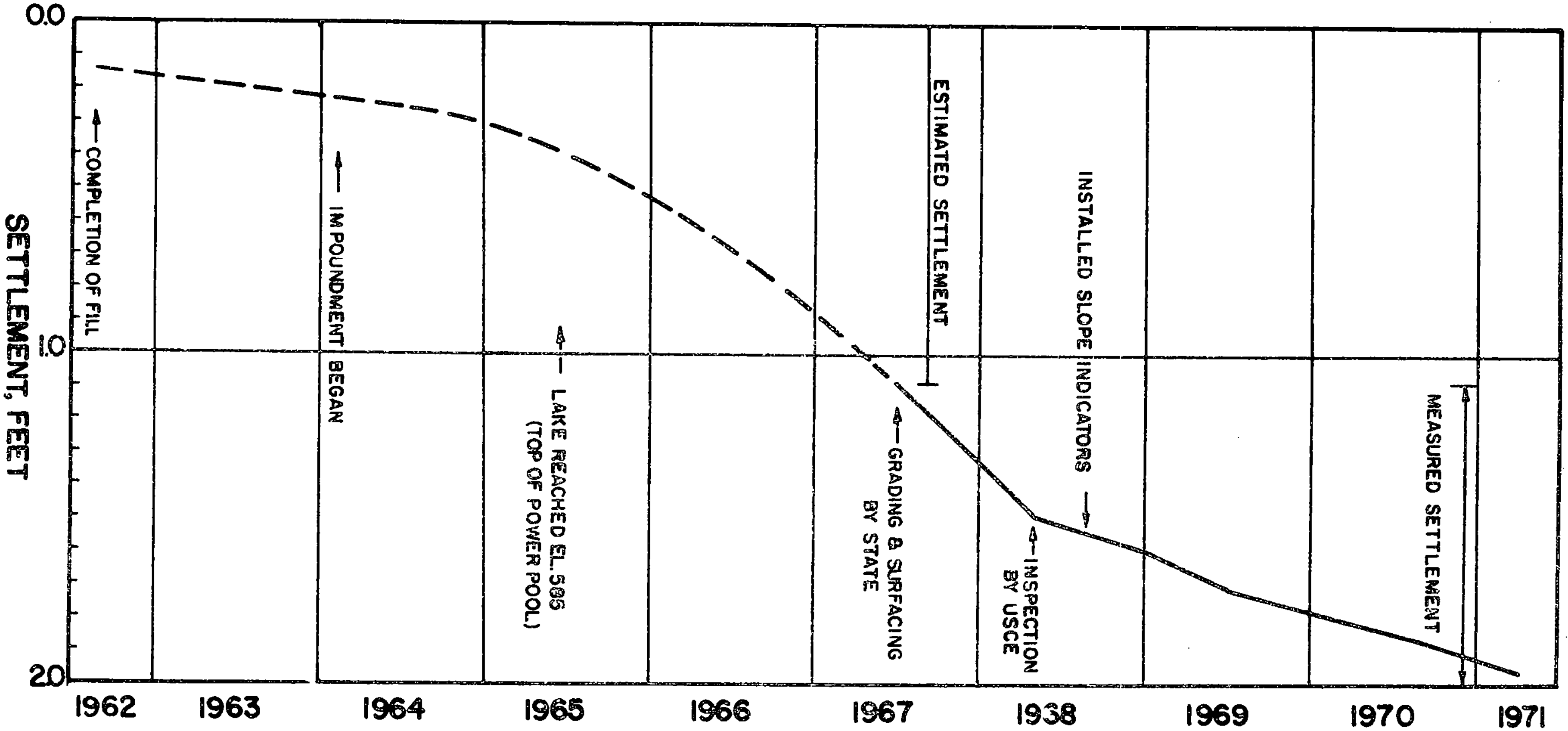


SETTLEMENT VERSUS DEPTH



RATE OF VERTICAL MOVEMENT OF SLOPE INDICATOR CASING

Figure 10.



POST CONSTRUCTION PERFORMANCE - SETTLEMENT OF APPROACH FILLS

Figure 11.

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CONTROL OF A SLIDE BY VERTICAL SAND DRAINS
U.S. ROUTE 220, ALLEGHANY COUNTY, VIRGINIA

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Abstract.--This paper reviews the construction of a portion of U.S. Route 220 between the town of Iron Gate and the city of Clifton Forge in Alleghany County, Virginia, in 1947-49. Some of the more interesting points of construction are covered, especially the control of a major slide by the use of vertical sand drains, one of two instances when these drains were utilized in such a manner.

The slide developed on the flank of an anticline in Rich Patch Mountain in a thick mantle of talus. The construction of a stout toe wall failed to check the slide, and after consultation with the U.S. Geological Survey use of sand drains was deemed feasible. Accordingly, 8 well-drill holes 8 inches in diameter were drilled 80 feet deep and backfilled with a well-graded concrete sand. The drains were then capped with a bituminous sealer to prevent surface water from entering. The drains were completed in the spring of 1949. By intercepting percolating ground water, these drains have effectively stopped the slide.

In 1946, the Virginia Department of Highways programmed the reconstruction of U.S. Route 220 between the town of Iron Gate and the city of Clifton Forge, Alleghany County, Virginia, a distance of approximately 2.5 miles. The work concerning the sand drains, which was done by convict labor, was begun in the summer of 1946 and completed in the fall of 1949. This paper is an account of the control of a slide which developed during construction, and it also relates the removal of a rock overhang above the highway. During this time the writer was in complete charge of the engineering geology.

The first paper on this method was given at the Highway Research Board's annual meeting in January 1955, and the method was reported in Highway Research Board Bulletin 29, Landslides and Engineering Practice.

The physiographic province in which this section of Route 220 lies is the Valley and Ridge province of Virginia. The topography is much dissected. The main stream is the Jackson River, which has cut through Rich Patch Mountain, forming Iron Gate Gorge. The project passed through this gorge for about three-fourths of its length. The gorge reveals a cross section of the mountain core, a classic combination of a perfect anticline and overturned folds. The rocks exposed are the Clinch Sandstone and the Tonoloway Limestone of Silurian age. The only fossil found by the writer was in the Clinch Sandstone; it was a very large worm by the exalted name of *Arthrophycus alleghaniensis*.

From the very beginning it was realized that the key to successful construction of this highway lay in proper drainage and the elimination, or at least the minimizing, of slides. To complicate matters, the section through the gorge was the area in which most of the rock was encountered. This rock had to be removed without (1) blocking the road, (2) blocking the James River Division of the Chesapeake and Ohio Railroad, or (3) damming the Jackson River.

On the north side of Rich Patch Mountain the road parallels the axis of the mountain and is 15 to 20 feet above the Jackson River. Now we can proceed to the construction problems.

In the gorge itself, two overhanging ledges of quartzite had to be removed, and utmost care was necessary to accomplish this without interfering with normal road or rail traffic. Very small charges of explosives were used, detonated by delayed-action caps having a 1/25-second lag between each individual detonation. During the 18 months during which these ledges were being removed only 2 minor incidents marred an otherwise perfect record: a telegraph wire was cut and a rail was knocked out of alignment. This is certainly a tribute to the highway maintenance personnel who supervised the construction.

Meanwhile, on the north slope of the mountain, trouble began to materialize. A number of small springs which emerged from the colluvial slope were effectively contained by pipes and spring boxes, but during the summer of 1948 a major slide began to move.

The then-existing Route 220 was perched some 75 feet above the new grade, and the slide began to break back into the pavement of the road. Water was observed emerging from the cut slope about halfway up the slope, eating away the support beneath the old road.

In an effort to stabilize this slope, a stout toe wall was constructed. Here, for once, Nature gave us a helping hand. The ledges of quartzite high on the mountainside had bedding and joint planes at right angles. Through the ages, these rectangular blocks had broken off and rolled down the mountain and were intermixed with the excavation for the wall. These blocks were utilized in the construction of the wall, which was about 3-4 feet in height and about 5 feet wide. This wall was extended the entire length of the north slope. As an added precaution against the slide's enlargement, excavation for the wall was carried forward only 5 feet and the wall built immediately behind it. The wall contained the slope well except in the area of the slide, where it was pushed about 18 inches out of line. Fine sand and clay continued to be brought down the slope by water, undermining the old road.

In September 1948, Mr. Robert A. Laurence, regional geologist for the U.S. Geological Survey (a former member of the Steering Committee of this organization), and the writer made a detailed survey of the north slope and found that a relatively thin but very impervious layer of clay was preventing the downward passage of water, causing it to break out on the slope and undercut the old road. After various methods of arresting the slide had been explored, vertical sand drains were employed. So far as we knew, this was a novel use of these drains, but it seemed logical that if the clay layer were punctured and a clear line of drainage established, the slide should stabilize.

Accordingly, the Highway Department contracted for the drilling of eight 8-inch-diameter holes 80 feet deep and spread on 100-foot centers. The 80-foot depth was decided upon in order to discharge the water below the bed of the Jackson River. The holes were drilled in the ditch line of old Route 220, installed with a 6-inch well casing, and filled with concrete sand. The casing was then withdrawn, and a bituminous cap was placed on the hole.

After the first hole was completed, it was found that the water did not drain as expected. Investigation showed that the hole had terminated in a well-cemented bed of sand and gravel of unknown depth and lateral extent. This difficulty was resolved by lowering five sticks of dynamite to the bottom of the hole and pulling the shot. This worked fine, as the impervious material was fractured, affording free drainage. The casing was then lowered to the bottom of the hole, filled with sand, and capped. This procedure was followed on the remaining holes.

The work on these drains was started in January 1949 and carried through a very wet spring, the last hole being completed in April. On this hole a pocket of gas, probably methane, was encountered. A spark from the drill bit ignited it with a loud WHOOSH! A tongue of flame shot high in the air, and the driller--accustomed to drilling for water rather than gas--took off. It took the rest of the day to catch him and persuade him to return to work.

Now that a span of 22 years has elapsed since completion of the work, and because there has been no appreciable movement of the slide, it may be concluded that these drains, plus the retaining wall, have effectively controlled the slide.

It is the opinion of this writer that similar utilization of drains in slide areas will prove beneficial, provided there is a porous material below the slide area where water can be drained.

STRAIGHT CREEK TUNNEL CONSTRUCTION, ROUTE I-70, COLORADO

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Abstract.--A tunnel through the Continental Divide at Straight Creek Pass was considered the safest and most feasible route for the extension of I-70 westward from Denver. After construction of a pioneer bore in 1963-64, construction of the main tunnel began in 1968. Portal elevation is 11,000 feet above sea level and length of the tunnel, including ventilation buildings at either end, is 8,900 feet. Even though bedrock conditions are generally good, problems developed in tunneling through faulted and jointed zones.

The completion of this tunnel will provide a safe and swift route through the Colorado Rockies, and the motorist will be offered an alternative to frequently hazardous U.S. 6 through Loveland Pass.

INTRODUCTION

After the advent of the Interstate Highway System, Colorado still lacked a complete east-west route as a part of this system. I-70 came in from the east, but terminated in Denver at the junction with I-25. The lack of continuity of a highway of this type through Colorado was recognized as a serious deficiency to this mountain state. It was evident that our neighboring states, Wyoming on the north and New Mexico on the south, would have a big advantage in their east-west Interstate roads. Also, tourism is a major business in Colorado, both for summer vacationers and for the ever-increasing populace who love skiing. With these thoughts in mind, a vigorous campaign was started to obtain a westward extension of I-70 from Denver. The efforts of many dedicated people resulted in approval of this hoped-for route in 1960.

An extensive study was made to determine the proper location for this important highway. The location found to be most practical generally parallels U.S. 40 from Denver westward for about 40 miles, thence along U.S. 6 to the Utah state line.

One formidable obstacle blocked this route when Interstate standards of design were considered--Loveland Pass. The approaches to this pass are steep, involving lengthy stretches of 7-percent grade. Four switchbacks exist on the east side, two of which have radii of about 100 feet with very little possibility for improvement. Four switchbacks exist on the western side of the pass with very steep grades. Again, these extremely sharp curves offer no appreciable possibility for improvement.

Often-active avalanches on both sides of the pass present a continual hazard, not only to the motoring public but also to our maintenance people. In order to minimize the danger, we have a control program that calls for periodic closing of sections of the highway to shoot potential avalanches with 75-mm howitzers. This procedure has been very successful in keeping snowslides small. One unavoidable hazard, however, still exists. Some of the avalanches build up and run during a storm when visibility is practically zero. We hesitate to shoot from firing platforms when visibility is poor because we will not expose our people to the possibility of a second run occurring. Nearly all the people we have lost in snowslides have been in second runs.

More than 4 miles of 11,988-foot Loveland Pass is without timber. Severe wind conditions often reduce visibility to zero, necessitating closing the road, sometimes for a few hours and occasionally for as much as 24 hours. The Division of Highways has always been concerned with the high pass and the problems it presents. Development of ski areas began in the late 1930's and the early 1940's, adding even more impetus to the desire for a safer, shorter route.

It was obvious that construction of a properly safe highway required a tunnel. With this in mind, a survey was made which included a tunnel under Loveland Pass. A pioneer bore was constructed. Construction began in the fall of 1941, with completion in 1943. The cost of this 7-foot x 7-foot bore was $281,000, approximately $52 per foot. Ground conditions encountered in the construction of the pioneer bore were so poor that the idea of a highway bore was abandoned.

With the advent of the Interstate System, a more feasible location in the same general area was needed for a tunnel under the Continental Divide. Available geological information indicated that a tunnel under Straight Creek Pass would be best. In order to prove that a highway bore could be constructed and to give prospective bidders all the geological information possible, it was decided to construct a pioneer bore. Construction of the pioneer bore (approximate dimensions, 10 feet x 10 feet) was started in the fall of 1963 and completed the following year. This tunnel was about 8,400 feet long, and the contract price was $1,292,000 (approximately $154 per foot).

A group of consultants was retained to design the first of twin bores. A major aspect of its design was ventilation. Portal elevation is 11,000 feet above sea level. Very little information was available as to amount of carbon monoxide emission by automotive equipment. Information was also lacking concerning human tolerance at these high altitudes. After a great deal of research, air requirements were determined to be 3,200,000 cfm at each end of the tunnel, or a total of 6,400,000 cfm.

THE TUNNEL

Location: Straight Creek Pass, about 60 miles west of Denver.

Project presently under contract: first of twin bores plus ventilation buildings.

Length, face to face of ventilation buildings: 8,900 feet.

Length of tunnel: approximately 8,100 feet.

Height of tunnel excavation: 38 to 48 feet.

Width: 41 to 47 feet plus overbreak.

The finished tunnel will be 34 feet between walls and will provide a 26-foot-wide travel width between curbs. The vertical clearance will be 16 feet, 5 inches, from the finished roadway to the ceiling. The space above the ceiling will be used for air. A divider is provided to separate fresh air from exhaust.

The support system consisted of straight-leg steel in good ground varying from 10-inch WF @ 66 pounds per foot to 12-inch WF @ 106 pounds per foot. Planned spacing varied from 3 to 5 feet. Specifications of 10-inch WF @ 29 pounds per foot were also included in the plans but were not used because of the danger of damage from blasting. An option was also available to use rock bolts and wire netting, but these were not used because of the hesitancy of workmen to work under this type of protection. The cost of the rock bolts was as much as for the steel supports.

For ground that exerted lateral pressure along the ribs, curved-leg steel with inverts was included in the plans. These sets consisted of 12-inch WF up to 190 pounds per foot and 14-inch WF up to 287 pounds per foot.

The specifications included a value-engineering clause, because it was felt that a job of this magnitude would involve work that might lend itself to savings both to the contractor and to the owner. Two change orders were prepared on the value-engineering concept. One of these, involving lining, did realize minor savings. The other, involving modification of the support system, has not been implemented.

GEOLOGY

The rock is about 75 percent granite and 25 percent gneiss and schist. A series of faults, dikes, and shear zones occurs along the length of the tunnel.

Several hundred feet of rock, known as the Loveland fault, has been severely faulted, altered, and crushed to form clays and swelling soil.

CONSTRUCTION

Five contracting firms submitted bids in October of 1967 on this large and complex project. The project was awarded, the contract bid price being approximately $50 million, and construction started in March 1968.

Initial work started at the west portal. The contractor had elected to use the top-heading and bench method. This work was started with a gantry-type top-heading jumbo. This jumbo utilized 12 drills on 2 decks. The normal drill pattern consisted of about 120 holes. The length of the holes was 6 to 12 feet, depending on ground conditions. After a round was pulled, muck was removed, using a 4-yard front-end loader. Hauling equipment consisted of 30-ton Athey wagons. The jumbo was equipped with power for the placing of steel sets. These crown portions of the steel sets were set on wall plates, which consisted of two 10-inch WF @ 35 pounds per foot connected with tie rods and fastened to arch steel and posts by toggle plates and bolts.

Even though ground conditions on the west heading were good, where bedrock consisted of granite, some problems were encountered where short faults were intersected by the tunnel. Two of these caused some delay while the resulting runs were caught up. It was necessary to spile to advance the heading in these areas and breast-boarding techniques were used while driving the spiling. The spiling used was 70-pound railroad rail and 1-3/4-inch reinforcing rods. Where the ground was not too heavy the re-bar was better because of ease in handling and better adaptability to driving.

As soon as the westerly 4,200 feet of top heading had been completed, the bench was removed by drilling down holes with air tracks. The center portion was removed, leaving a 4- to 6-foot shoulder on each side under the wall plate. This wall plate had been previously reinforced by pouring a concrete beam above the steel plate. The beam was secured to the rock wall by the use of rock bolts. Shoulders were removed by slabbing. Considerable overbreak resulted as the bench was removed, owing, no doubt, to both the jointing pattern in the rock and blasting damage. Even in this hard rock, jointing was such that large pieces of rock tended to slip out of the ribs into the tunnel. Selective rock bolting in the ribs was the solution to this problem.

Initial plans for the east heading were to drive full face using a shield. Preparatory work for this method of tunneling consisted of driving foundation drifts just below the floor in each rib. These drifts consisted of 8-foot x 8-foot tunnels. Steel sets with inverts were used to support the ground. About 5 feet of reinforced concrete was placed in the lower part of the drifts, with a channel embedded in the top of the concrete to form a guide for the shield to travel in.

The contractor abandoned the use of the shield on the east end because he felt the experience gained in the west heading would enable him to drive from the east by the top-heading and bench method. Progress on the east heading was much slower than on the west end. Even though ground encountered was similar, the jointing was such that large slabs slid out of the breast, some so large that they had to be drilled and shot in order to load and remove them.

Reinforcing steel was placed in the tunnel by the use of a re-bar jumbo developed by the contractor. The mat of reinforcing was assembled outside the tunnel, transported, and placed by use of this jumbo. Figure 1 is an eastward view down the tunnel.

Lining of the western end was started early in 1970. Six sets of forms were acquired by the contractor. The lining equipment consisted of a large batch plant at the west end of the tunnel, two Thompson pumps in the tunnel

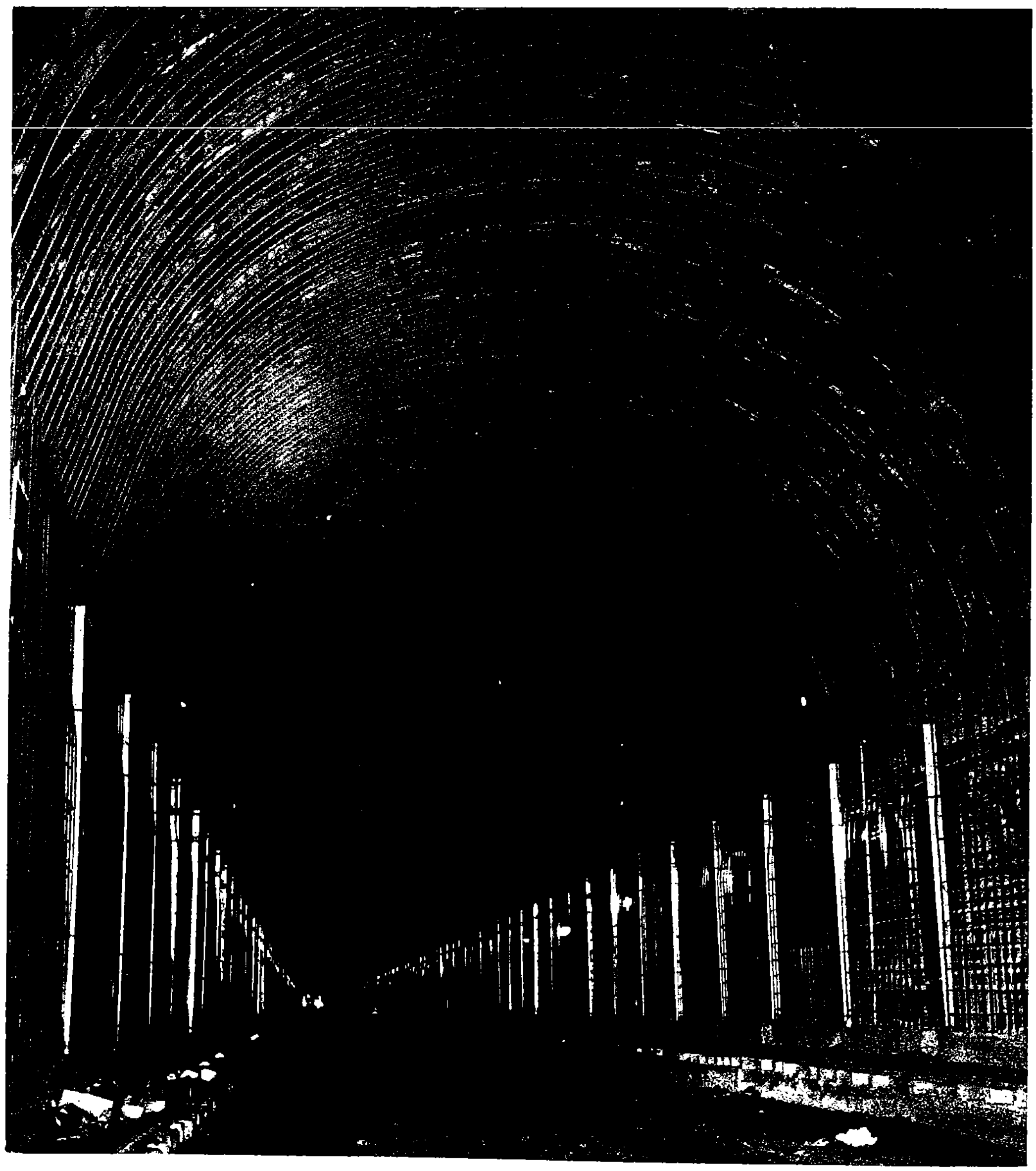

Figure 1. Looking eastward down tunnel, showing fresh-air flues and re-bar placement. Photograph courtesy of Straight Creek Constructors.

Figure 2. West ventilation building under construction.

with the appropriate slick lines. Concrete was transported in transit mix trucks. It was initially thought that the lining would be placed continuously, concrete placements to continue for perhaps a week. It was hoped to get 600 feet or more of tunnel lining per week. In actual practice it became apparent that a pour of two segments 100 feet at a time was all that could be accomplished, because a typical concrete pour of lining for 100 feet of tunnel took 10 to 12 hours. About 4,000 feet is now lined. The amount of concrete placed was 1,060 cubic yards. The amount of concrete within the "B" line (pay quantity) was 837.58 cubic yards. This indicates an overbreak of over 25 percent.

A ventilation building is being constructed on each end of the tunnel (fig. 2). These 2-story buildings are very large--252 feet wide and 182 feet deep. The eastern one is to have a partial basement with sewage-disposal equipment, etc.

As has been true with almost every tunnel constructed through the Continental Divide in Colorado, difficulties have been encountered; these have been aggravated by the enormous size of the tunnel. So far, 1,800 feet remains untouched except for drifts which were driven through it. The contractor and the Division of Highways are presently studying methods for successfully constructing this portion, which penetrates the most badly altered and crushed ground.

The tunnel will thus be constructed, and motorists will not have to face the fearsome blizzards and avalanches common to Loveland Pass.

HIGHWAY GEOLOGY FEASIBILITY STUDY, LULUABOURG TO MBUJI-MAYI, REPUBLIC OF CONGO (KINSHASA)

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Abstract.--In late 1969, geologic field studies were made, as part of an AID-sponsored economic survey, to determine the most feasible route between Luluabourg and Mbuji-Mayi in the Kasai Provinces of the Congo. A five-man team consisting of a transportation engineer (team leader), a transporation economist, a traffic engineer, an agri-business specialist, and a geologist conducted the field study.

The first phase of the geologic study was an engineering photogeologic study of existing government photography flown in 1950-53 at an approximate scale 1:30,000. Although the quality of available photography was poor, it was invaluable to the subsequent field geologic and soils investigations, which were necessarily brief.

The field work included an aerial reconnaissance prior to ground traverses. Northern (157 km), middle (150 km), and southern (173 km) routes were studied, along with minor associated alternates. Numerous 35-mm slides were taken of important features and were correlated with the 1:30,000-scale photography. Ground traverses were made by 4-wheel-drive vehicles. Field (pocket) penetrometer tests were made of the major soils types encountered along the roads, and samples were collected for laboratory analysis in the United States. Correlations were made between type of material, precent grade, and erosion conditions of existing roads.

Three general rock types were found in the area, and a simplified usage and terminology, based on engineering characteristics, were adopted for these units. In ascending order, from oldest to youngest, the units encountered were Crystalline Rocks (granites and gneisses), Carbonate Rocks (primarily limestones and dolomites), and Sandstones and Siltstones (including shales and conglomerates). The Carbonate Rocks are absent in the western part of the area, where Sandstones and Siltstones lie directly on Crystalline Rocks. All of the rock units mapped have been subjected to deep lateritic weathering, although numerous outcrops were found in the Crystalline and Carbonate Rocks.

Sources of natural and crushed aggregate were investigated, and the most dependable sources of material were determined to be Crystalline and Carbonate Rocks.

Early consideration of all factors (engineering, economic, and geologic) indicated that modification of one of three existing routes would provide the most economical access between the terminal cities. Based on geology, length, condition of existing route, required realignments, river crossings, and availability of aggregate, upgrading of the southern alignment was considered most feasible. The ultimate recommendation was for first-priority rehabilitation of existing northern and southern routes with provision for upgraded maintenance. Second or long-term priority was given to development of a shorter

central route which would be advantageous in an overall road-network plan.

INTRODUCTION

In October 1969, the Agency for International Development (AID) contracted with two international engineering firms to make highway feasibility studies in the Democratic Republic of Congo (Kinshasa). DeLeuw-Cather International, Inc. undertook studies in an area primarily between Luluabourg and Mbuji-Mayi in the Kasai Provinces, and Lyons Associates, Inc. made studies of a more westerly segment of the existing national highway system. The work decribed herein was done for DeLeuw-Cather International.

The geologic work for this project was part of an overall engineering feasibility study which included economics and agriculture. The project was conceived as part of a national objective to establish a modern trunk-highway network. National Route 1 is the major east-west axis of this system, which joins Kinshasa to the vast interior of the Congo. Luluabourg, a major railroad town, and Mbuji-Mayi are important centers and provincial capitals situated on National Route 1. The Mbuji-Mayi mines produce 60 percent (12 million carats per year) of the world's natural industrial diamond supply; thus the importance of the road system in this part of the Congo is apparent.

This paper is concerned primarily with geology and does not discuss details of engineering, economics, or agriculture.

TEAM CONCEPT UTILIZED

Inasmuch as rather stringent time limitations were set forth in the contract, a team concept was formulated for the project. The basic team consisted of a highway engineer (project manager), a traffic engineer, a transportation economist, and an engineering geologist. A prestudy meeting was held by AID officials for the project manager and economist in order to clarify certain aspects of the field work. This meeting proved to be of great value in the subsequent on-site work and report preparation. At a late stage in the field work, an agricultural economist was mobilized on the job.

The basic team formed a highly mobile group. Each individual had his own primary responsibility to the project, but each also provided needed help in performing the myriad small tasks that always must be done when field time is limited in a foreign country. The project manager kept all members informed of needs and requirements, and nonspecialty work was done by whichever team member was available to undertake it.

LOCATION AND ACCESS

The Kasai region is located in the south-central part of the Congo (fig. 1). It is bordered on the south and southeast by the province of Katanga, on the east and northeast by Kivu and Orientale provinces, on the north and west by the provinces of Equateur and Bandundu, and on the southwest by Angola.

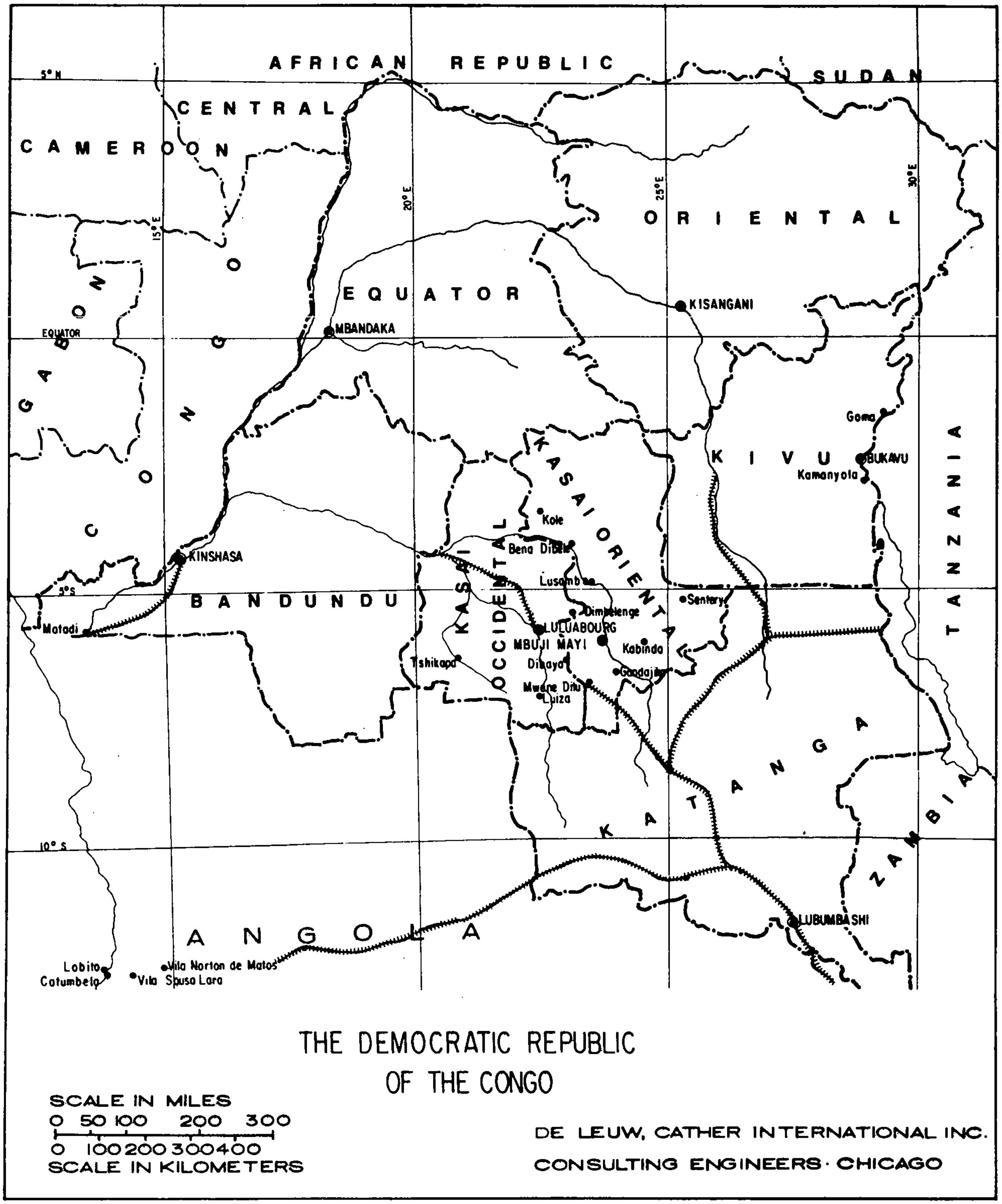


Figure 1. Index map.

Access to the region is provided overland by a road system connecting it to the northwest with Kinshasa and with Lubumbashi in the extreme southern part of the country. By most standards the quality of many sections of this road is very poor, and work is under way to improve the entire main road system of the southern part of the country. Much of the back country, well away from the main roads, can be reached by a system of interconnecting minor roads and trails.

The Kasai is crossed from northwest to southeast by a railroad extending from Port Francqui on the Kasai River to Lubumbashi, where connections are made to the neighboring countries of Angola and Zambia. Port Francqui is the transfer point for goods barged upriver to the railroad.

Air transport into the Kasai is provided by regularly scheduled flights from Kinshasa into Luluabourg and Mbuji-Mayi. Other cities are served by charter airlines. Flying time from Kinshasa nonstop to Mbuji-Mayi by DC-4 is about 2.5 hours.

VEGETATION AND CLIMATE

The rain forest found to the north in the lower parts of the Congo River basin gives way southward in the project area to a grass-covered savannah dissected by the main rivers and their tributaries. A general view of this savannah and forest vegetation is shown in figure 2 taken from the air between Luluabourg and Mbuji-Mayi. The average annual temperature in the area is about 87°F. The annual average precipitation measures 43 to 70 inches, with a dry season of 90 to 100 days, beginning in late May.

Subsistence crops are raised throughout the region, and the combination of soil and climate found throughout most of the area makes rudimentary agriculture a relatively simple matter.

PHYSIOGRAPHY

In the project area the total relief is between 200 and 300 feet, superimposed on elevations ranging from 2,247 feet msl at Luluabourg to 2,175 feet msl at Mbuji-Mayi.

The development of the land surface is controlled by geology and the climate. Most of the Kasai is underlain by poorly cemented sandstones and siltstones which weather easily under the influence of high temperature and humidity to the deep lateritic soils common to tropical Africa. Even the areas underlain by harder rocks, such as granite and gneiss, are so deeply weathered that coherent rock outcrops are uncommon. The generally sandy nature of the soil and rock allows the retention of much moisture and the development of dense stands of grass throughout the region.

SETTLEMENT AND POPULATION

Most settlement is along main roads. It is virtually impossible, in the savannah areas, to drive more than 1 or 2 km without encountering a few thatch huts or an organized community.

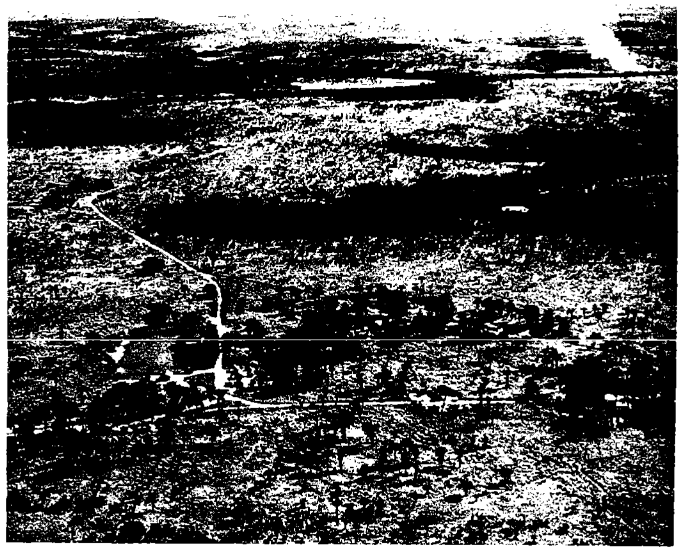


Figure 2. Grasslands developed on modified karst topography between Mbuji- Mayi and Lac Mukamba.

Apparently it is not uncommon for a whole settlement to move to a different location. This causes problems in making up-to-date maps showing named communities and in determining population distribution. A unique problem in the Kasai is that local inhabitants do not always recognize a settlement by the name given on an official government map. This probably stems from tribal customs and the whim of the current chieftan.

The critical nature of population and settlement patterns was an integral part of the economic aspects of this study.

PHASED STUDY

The first phase of the geologic study was an engineering photogeologic evaluation of existing government photography flown in 1950-53 at an approximate scale of 1:30,000 with a vertical exaggeration of about 4X. Although the quality of available photography was poor, it was invaluable to the subsequent field geologic and soils investigations, which were necessarily brief.

The field work included aerial reconnaissance in a light aircraft flying generally about 150 to 300 feet above the ground. Low-level flights enabled a close look at the terrain and road conditions prior to making surface traverses. In numerous geologic studies involving aerial reconnaissance, this was probably the most efficient use of the technique that the writer has ever experienced.

In the aerial reconnaissance three general routes were observed between the terminal cities as shown in figure 3. It has been determined that an existing northern route 157 km long, or an existing southern route 173 km long, might be modified into acceptable highways. Office photogeologic reconnaissance prior to field work had indicated a possibility of extending an incomplete central route about 11 km to form a more direct connection of 150 km. It was not until the aerial reconnaissance was made that it was discovered that in the time since the aerial photography, the central route had been completed along the alignment that had been proposed on the basis of engineering photogeology.

GENERAL GEOLOGY

The geology of the area is relatively simple, as shown on the geologic map (fig. 4), except that deep lateritic weathering obscures what might otherwise be easily defined contacts between the different rock types and their weathered products. The deep red and yellow or ocherous colors characteristic of lateritic weathering are universal in this area. The three broad rock types found are (1) Precambrian crystalline igneous and metamorphic rocks, (2) Precambrian carbonate strata consisting mainly of limestones and dolomites, and (3) Lower Cretaceous poorly cemented sandstones and siltstones. Boundaries between rock types generally are difficult to observe; however, in local areas topography enhances the different qualities of each type, making a clear contact. It should be understood that because of deep weathering, the map accompanying this report does not show bedrock outcrops as much as it does the influence of the weathered equivalent of each rock type. Descriptions of rock types for this paper are for practical engineering application, hence no attempt is made to discuss the more classical aspects of the geology.

The general distribution of rock types is observed on the geologic map. The Precambrian igneous and metamorphic rocks are overlain in the western two-thirds of the area by Lower Cretaceous clastic sedimentary rocks. In the eastern one-third of the area Precambrian carbonate strata overlie the crystalline rocks, and in turn both of the older units are overlapped by the Cretaceous rocks.

Crystalline Rocks

Crystalline rocks include granites and gneisses, which crop out in numerous places between Tshintshanku and Bayomba on the southern alignment. These rocks weather to a tan, clayey soil and form a good roadbed, except where large boulders are found, as in figure 5 on the road in the same area. This bedrock and its weathered equivalents have excellent engineering characteristics, and granite from the quarry at Luluabourg (fig. 6) is used for railroad ballast, base course in roads, and other construction purposes.

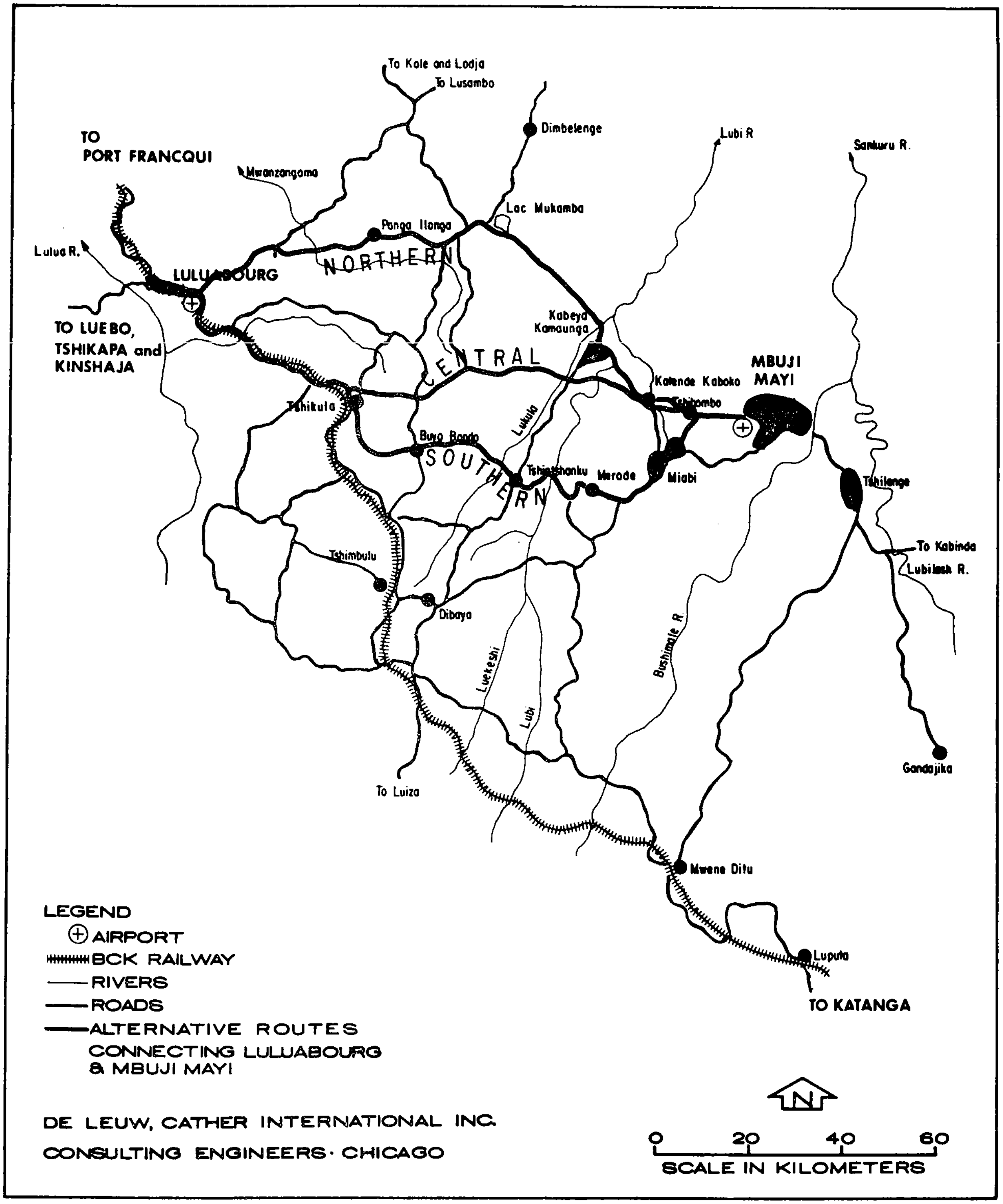


Figure 3. Route map.

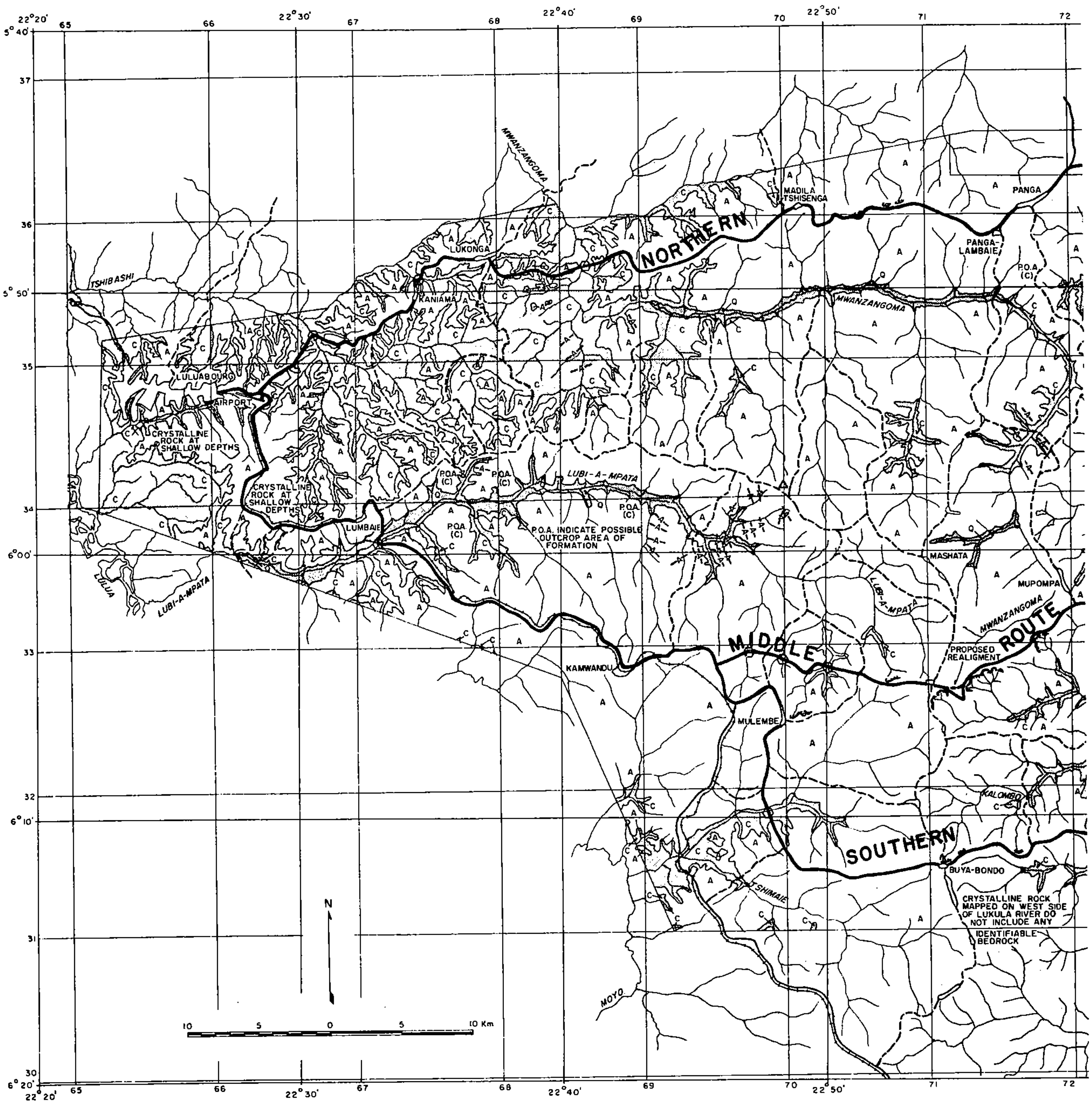
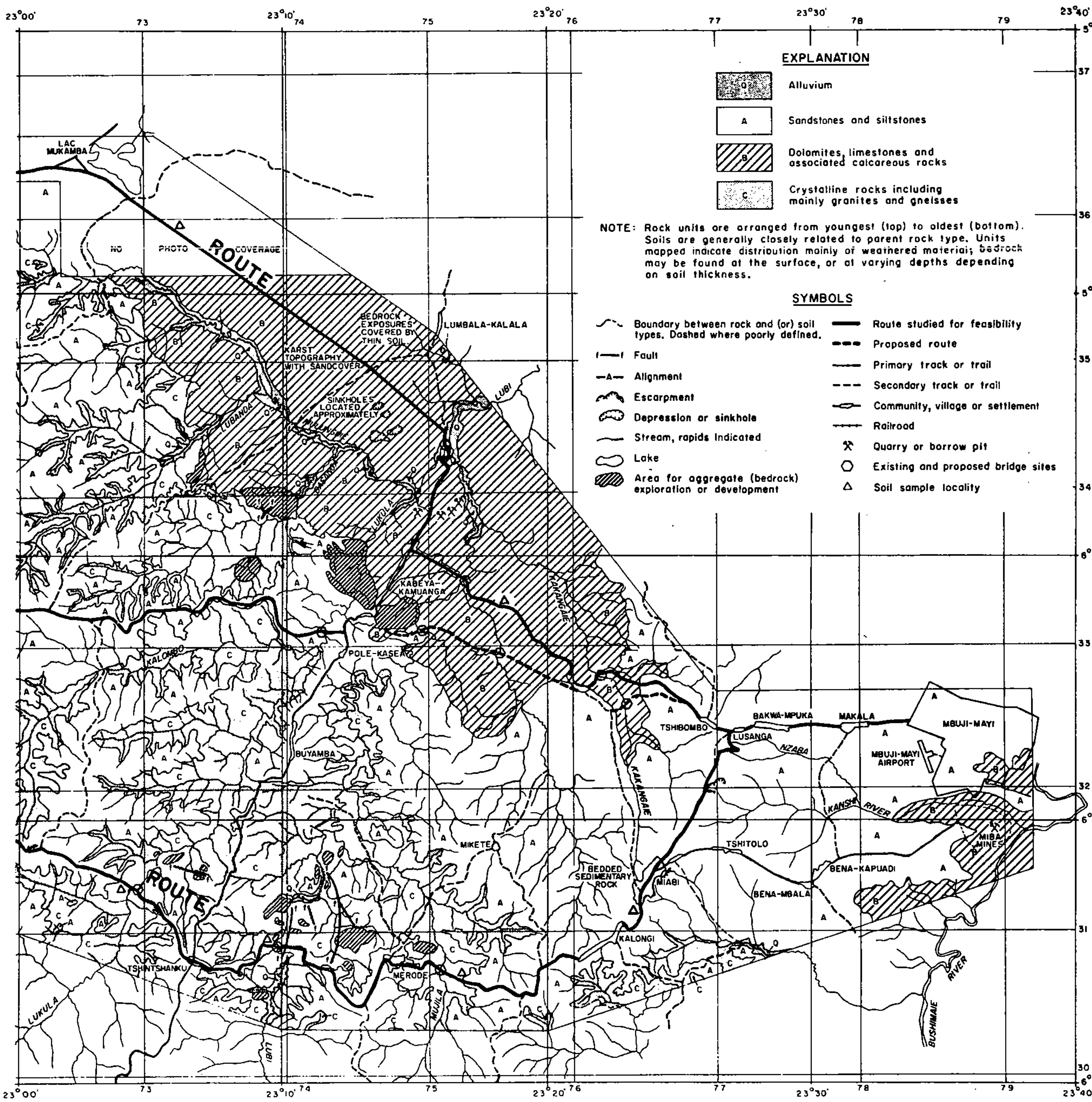


Figure 4. Geologic map.



EXPLANATION
Alluvium
Sandstones and siltstones
Dolomites, limestones and associated calcareous rocks
Crystalline rocks including mainly granites and gneisses
NOTE: Rock units are arranged from youngest (top) to oldest (bottom). Soils are generally closely related to parent rock type. Units mapped indicate distribution mainly of weathered material; bedrock may be found at the surface, or at varying depths depending on soil thickness.
SYMBOLS
Boundary between rock and (or) soil types. Dashed where poorly defined.
Fault
Alignment
Escarpment
Depression or sinkhole
Stream, rapids indicated
Lake
Area for aggregate (bedrock) exploration or development
Route studied for feasibility
Proposed route
Primary track or trail
Secondary track or trail
Community, village or settlement
Railroad
Quarry or borrow pit
Existing and proposed bridge sites
Soil sample locality
ROUTE
NO PHOTO COVERAGE
LAC MUKAMBA
BEDROCK EXPOSURES COVERED BY THIN SOIL
KARST TOPOGRAPHY WITH SANDCOVER
SINKHOLES LOCATED APPROXIMATELY
LUMBALA-KALALA
KABEYA-KAMUANGA
POLE-KASEA
BUYAMBA
MIKETE
MERODE
TSHINTSHANKU
KALONGI
MIABI
BEDDED SEDIMENTARY ROCK
TSHIBOMBO
LUSANGA
BAKWA-MPUKA
MAKALA
MBUJI-MAYI
MBUJI-MAYI AIRPORT
TSHITOLO
BENA-KAPUADI
BENA-MBALA
MIBA MINES
KANSHI RIVER
BUSHIMAIE RIVER
KAKANGAIE
LUBI
LUKULA
MUJILA
KALOMBO
NZABA
23°00'
23°10'
23°20'
23°30'
23°40'
5°40'
5°50'
6°00'
6°10'
6°20'

Calcareous Rocks

Calcareous rocks are not well exposed in road alignments but can be observed from the air. They form a karst topography as shown by the sinkholes in figure 7 along the Lubi River north of Kabeya Kamaunga.

Throughout much of the area mapped as Calcareous rock, a thin sand cover is found. This cover is thick enough in places to give a sandy appearance, but not so thick that it obscures the karst topography. In most of that part of the northern alignment between Lac Mukamba and Katende-Kaboko the sand is sufficiently thick to obscure any bedrock along or in the road. The origin of the sand is either outwash from the overlying weathered rock or blown sand similar, if not related, to the Kalahari sands.

Carbonate Rocks

Carbonate rocks form red clayey soils whose slickness is reduced in areas where the sand content is relatively high. The fresh limestones and dolomites are hard, dense, and relatively pure. These rocks have excellent engineering characteristics for the uses intended in this project. The cherty impurities found in these rocks might cause problems if the rock is to be used in concrete mixes. If slopes are cut in these rocks they probably will stand at ¼:1 unless adverse joints are present. Cuts in weathered material varied in slope from ½:1 to ¼:1. Penetrometer tests (SS-11-23-3) ranged from 0.0 to 1.2T/ft^2 in wet material.

Sandstones and Siltstones

Sandstones and Siltstones erode to dissected plateaus throughout most of the area and form erosional cirques as shown in figure 8. These rocks are poorly cemented, and only at a locality about 2 km east of Miabi was a reasonably fresh exposure found. Weathering is so complete and the parent rock so soft that exposures are uncommon.

The engineering qualities of the weathered sands were well observed in driving over the main roads between Luluabourg and the eastern part of the area. Continual traffic has compacted the weathered sands in the main tracks to the point where field penetrometer readings indicated unconfined compressive strength values in excess of 4.5 tons per square foot (62.5 psi) in most cases. After compaction of the road, erosion causes deep ruts locally. This is particularly true in those sections where the grade is in excess of 3 percent and there is inadequate side drainage. Erosion, though not so dramatic, will occur on a lesser scale where the grade is below 3 percent and drainage is inadequate.

In general, sidehill cuts are not advisable in this material. Natural slopes hold at ¼:1 to vertical, and cuts would hold similar slopes, but a high level of maintenance and relatively expensive construction would be required.

Figure 5. A section of road developed in weathered clayey and bouldery soil of the Crystalline rocks.

Figure 6. Railroad quarry, south edge of Luluabourg. All rock quarried is granite.

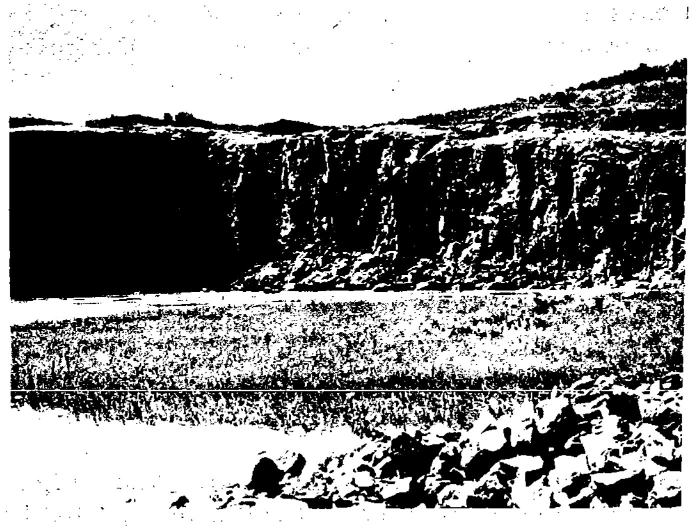


Figure 7. Sinkholes in Carbonate rocks on left bank of Lubi River north of Kabeya-Kamaunga.

Figure 8. Erosional cirques in the northern part of the area along the south side of the Mwanzangoma River.

Alluvial Deposits

Alluvial deposits of the major rivers are mapped and described for this study. The main areas where these deposits have formed in mappable size are along the Mwanzangoma, Mulunguie, Lubi-a-Mpata and Lubi Rivers. The flood plains of these rivers may contain natural aggregates of a size and quantity for reasonable use in highway construction. The only readily accessible alluvial deposits observed were at the bridge over the Lubi River east of Kabeya-Kamaunga. The materials were mostly subangular, and it was estimated that 60-70 percent was less than half an inch in diameter.

Structural Geology

The rocks exposed at the surface in this area are uncomplicated by folds, joints, and faults. Only a few definite faults were mapped in the Crystalline and Calcareous rocks; none were mapped in the Sandstones and Siltstones or in the Alluvium. The alignments shown on the geologic map have a structural connotation and probably indicate faults that are not easily defined at the surface. A correlation was established between linear drainage, marked in places by alignments, and the underlying Crystalline rocks. The linearity probably is due to joint systems. Similar correlations were observed in the Sandstones and Siltstones in areas where they are believed to form a thin cover over the Crystalline rocks.

ENGINEERING GEOLOGY

Each of the existing alignments is unpaved except for those sections in or near the terminal cities and poorly maintained approaches to two bridges on the northern route. Plans call for paving the approaches to all major river crossings on the chosen alignments.

One of the most striking characteristics of the existing highway systems is the almost invariable location of roads along drainage divides, without cut-and-fill sections. It is particularly important to get water off these roads as quickly as possible. Drainage ditches carry water off the shoulders, and at varying intervals side drains usually follow the contour and extend 20 to 60 feet away from the road. Some of these ditches feed into small sumps. The ditches dissipate the collected water and sediment and, if maintained properly, form a satisfactory means of dewatering road surfaces. Considering the type of construction used, the drainage-divide location has been the best for alignments located in sandy soils.

Most of the grades on all alignments were less than 5 percent. In the approaches to some of the bridges, short sections increased to between 10 and 15 percent. Where roads had been maintained, and (or) where adequate drainage had been provided, the roadbeds were in good condition, even with steep grades. Where maintenance and drainage were not adequate, even roads with grades 3 percent and below were in bad condition. This established

Figure 9. Section of well-maintained road at Bena Mbala on southern alignment. Waste gravels from diamond mining operations are used to surface road.

that proper maintenance was of critical importance. Figure 9 illustrates the condition of a well-maintained road near Bena Mbala. This road is used by MIBA, a diamond-mining company in Mbuji-Mayi, and is surfaced with waste gravel from alluvial mining operations.

A dramatic illustration of the value of proper maintenance is evident through a comparison of the travel time on the northern route before independence (1960) and today (1969). The trip used to take 2½ hours by passenger car. On our traverse the total actual driving time was 6½ hours.

Illustrations of side-drain erosion near Bakwa Mpuka on the northern alignment can be seen in figure 10, which also shows a section developed on the Sandstone and Siltstone unit and perhaps the ultimate in culverts.

IMPORTANT CONSIDERATIONS IN ROUTE SELECTION

Several factors were considered in arriving at conclusions relative to geologic feasibility of developing an upgraded route between the two cities. The geology, relative lengths, and general condition of the routes have been discussed. Required realignments to upgrade the general quality are shown on the geologic map. Other considerations are the number of river crossings, soils sampling and testing, and aggregate sources.

Number of River Crossings

Table 1 shows a tabulation of crossings. All river crossings on the routes under consideration are by bridges. The rivers in the area are not sufficiently wide or deep to require ferries.

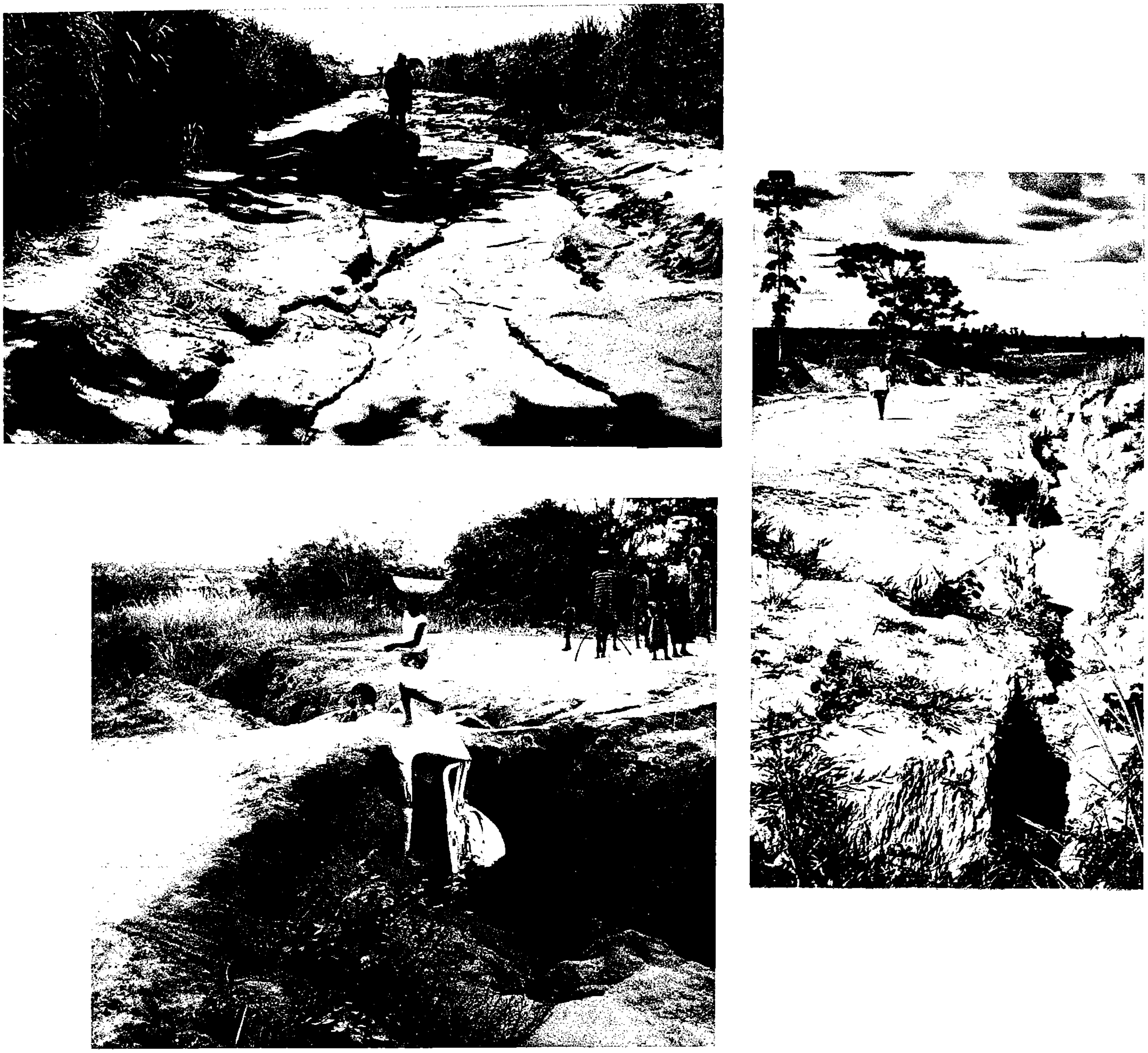


Figure 10. Examples of side-drain erosion: A, near Bakwa-Mpuka on northern alignment; B, on the Sandstone and Siltstone unit of another section on the northern alignment; and C, truck-body culvert in badly eroded side drain on southern alignment.

Bridge-construction techniques were more easily observed on the southern route because of Crystalline rock foundations. This rock forms an excellent foundation material and apparently is not reactive with concrete mixes used. It is sufficiently massive, i.e., without minor fractures, to withstand sloughing. A further indication of good foundation conditions is that, at some point in flood stage, water covers the floor of the bridge over the Lubi River without apparent adverse effect to the foundation or to the structure.

Soils Sampling and Testing

The area has been studied pedalogically, and a comprehensive report and maps have been published. In the preliminary photogeologic study a comparison of geologic detail with the soils maps indicated a very close coincidence between the distribution of rock types and soil on the different maps. The correlation of data was sufficiently close as to indicate that the geologic map represents the essentials of a soils map.

Generally the soils are described as "latosoils," which are characterized by red and yellow colors, a reduced amount of silica as compared to the parent rock, general absence of plasticity and cohesion in the soil owing to low clay content, and high permeability and porosity. A well-cemented crust often forms, and to a certain extent this explains the hardness of some well-compacted sections of roadbeds in the area.

Table 1. BRIDGES BETWEEN LULUABOURG AND MBUJI-MAYI

Route and River	Type	Span	Width	Foundation
Northern				
Mpaji	Wooden deck (Pic. V-5)			Bedrock or near bedrock; Crystalline rock
Pemba	Concrete deck			Alluvium
Mwanzangoma	Bailey (Pic. V-11)	83'	10'	Alluvium
Lukula	Stone piers, concrete deck (Pic. V-6)	67±'		Alluvium and (or) Calcareous bedrock
Lubi	Bailey (Pic. V-7)	126'	11'	Alluvium and (or) Calcareous bedrock
Kakangaie	Bailey, concrete piers	25'		Alluvium
Southern				
Mujila	Stone, double arch (Pic. V-10)	20-24'	8-9'	Granite bedrock
Lubi	Stone piers and concrete deck (Pic. V-8)	66'	10'	Crystalline bedrock
Lukula	Warren truss (Pic. V-9)	52'	12'	Granite bedrock
Lukusa	Wooden span and deck	20-24'		Alluvium
Lubi-a-Mpata	Stone, triple arch	40'	8-9'	Alluvium

Notes:

1. Foundation conditions are noted only as observed in the field. Information on construction was not available and in some cases, foundation conditions could only be estimated.
2. Data are not available for bridges on the Middle Route, except for air reconnaissance photography of the MIBA bridge over the Lukula River.

Nine soils samples were taken in the field for subsequent shipment to the United States for testing. Table 2 is a tabulation of these soils samples. Generally, red latosoils were derived from Sandstones and Siltstones and from Carbonate rocks. The yellow and ochre to red soils were derived from Crystalline rocks. Figure 11 shows the mechanical analysis and Atterberg limits for the samples tested.

Aggregate Sources

Potential quarry sites are shown on the geologic map where photogeologic and (or) field observation indicates reasonable conditions for rock extraction. The only active quarry of any size located in the area studied is the railroad quarry at Luluabourg, shown in figure 6.

The central part of the area mapped does not appear to hold reasonable promise for shallow, economically produced crushed stone. The weathering appears to be too deep in the Crystalline rock areas to afford economical extraction.

Table 2. SUMMARY OF SOILS SAMPLE DATA

Sample No.	Penetrometer Readings*	Parent Rock and Soil Type ** (3)
SS-11-21-1	+4.5T/sq.ft.	Sandstone and Siltstone, S-11, at crest of hill east of Lubi River
SS-11-22-1	3.3-4.5T/ " "	Sandstone and Siltstone, S-11, about 1 km south of Miabi
SS-11-22-2	0.6-+4.5/T " "	Sandstone and Siltstone, K, about 400 m. west of Mujila River
SS-11-22-3	2.5-+4.5T/ " "	Crystalline rock, T, about 1/3 km west of Lubi River
SS-11-22-4	+4.5T/ " "	Crystalline rock, ST, about 1 km west of Lukula River
SS-11-22-5	1.2-4.5T/ " "	Sandstone and Siltstone, S-11, between Nsadi and Kasonji
SS-11-23-2	+4.5T/ " "	Calcareous rock, SB, 20 minutes south of Lac Mukamba
SS-11-23-3	0,0-1.2T/ " " (wet)	Calcareous rock, SB, just south of Lukula River

* Unconfined compressive strength measured with a Soiltest CL-700, hand penetrometer

** S-11 Red to yellow latosoils, developed from Sandstones and Siltstones

SB Red Latosoils, derived from Calcareous rocks

K Red Latosoils, derived from Sandstones and Siltstones

T Yellow Latosoils, derived from Crystalline rocks

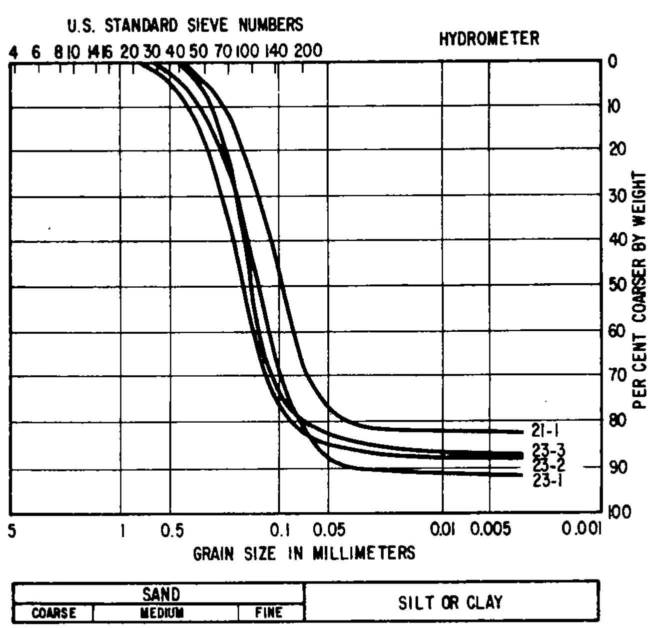
ST Ochre-red and yellow latosoils, derived from Crystalline rocks

A potential source of aggregate is the waste from certain of the MIBA alluvial diamond-mining operations. The beneficial use of this aggregate on a well-maintained road is illustrated in figure 9 at Bena-Mbala. The gravels are separated into an oversize of greater than 25 mm (about 1 inch) and the balance, which is further processed for diamonds. Only the oversize would be available, presumably, as the balance of the material is subjected to a final crushing well below the limit of road usability.

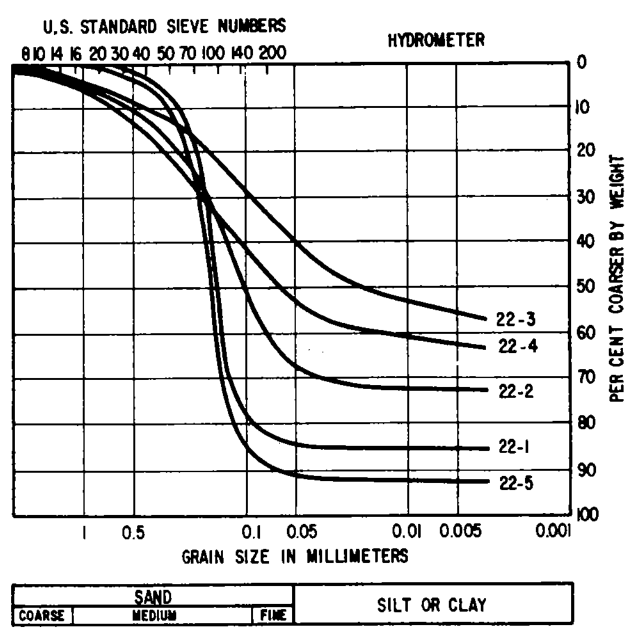
GEOLOGIC FEASIBILITY OF ALIGNMENTS

This consideration of feasibility relates only to geology and does not incorporate other important factors bearing on regional economic support for a particular route. The geologic feasibility of each alignment had already been established, to a certain extent, prior to the study--inasmuch as roads had been constructed. Thus feasibility related mostly to upgrading existing alignments and proposing realignments.

Based on geology, length, condition of existing route, required realignments, river crossings, and availability of aggregate, upgrading of the southern alignment was considered most feasible from the geologic point of view. The ultimate engineering-economic recommendation was for first-priority rehabilitation of existing northern and southern routes with provision for upgraded maintenance. Second or long-term priority was given to development of a shorter central route, which would be advantageous in an overall road network plan.



BORING NO.	SAMPLE NO.	CLASSIFICATION	NAT. W %	LL	PL	PI
SS-11-21	1	RED-BR. FINE SI SAND	-	-	-	NP
SS-11-23	1	LT BR. FINE SI SAND	-	-	-	NP
SS-11-23	2	BR. FINE SILTY SAND	-	-	-	NP
SS-11-23	3	RED-BR. FINE SI SAND	-	-	-	NP



BORING NO.	SAMPLE NO.	CLASSIFICATION	NAT. W %	LL	PL	PI
SS-11-22	1	RED-BR. FINE SI SAND	-	-	-	NP
SS-11-22	2	BR. SANDY SILT	-	26	21	5
SS-11-22	3	BR. SANDY SILT	-	33	26	7
SS-11-22	4	BR. SANDY SILT	-	31	25	6
SS-11-22	5	BR. FINE SAND	-	-	-	NP

Figure 11. Mechanical analysis and Atterberg limits.

ACKNOWLEDGMENTS

Thanks are expressed to DeLeuw-Cather International, Inc., and especially to Mr. Henry O. Johnson, for permission to publish this paper. The success of geologic field efforts for this work was due in large part to the cooperation received from team members, particularly John Gourley, the project manager. This paper is based on the report written for the project. The writer, although acknowledging the help of others, assumes full responsibility for the geologic statements in this paper.

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THE CAMINO MARGINAL, PERU[1]

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Abstract.--The Camino Marginal is an ambitious project designed to open up opportunities for development in Peru on the eastern slopes of the Andes and at the headwaters of the Amazon River.

Partially built, the project essentially ground to a halt in 1968 due to the magnitude of slope-stability problems and financial difficulties.

Some sections traverse the selva where dense jungle growth, high annual rainfall (108 inches in 1966), and excessive humidity make working conditions unusually poor. The more northerly sections face first the rolling foothills and then the incredibly steep slopes and deep canyons of the High Andes.

Soils in the selva sections have a high clay-mineral content, whereas the poorly cemented Mesozoic strata in the eastern approaches to the Andes present serious slope-stability problems.

This paper presents only data relative to the physical situation.

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[1]The author was unable to secure clearance to publish this paper; only the abstract is available.

A DESIGN APPROACH TO ROCK SLOPE STABILITY

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Abstract.--The use of rock mechanics in the design and construction of rock cut slopes is coming of age. Engineers now realize that the shearing resistance of a rock mass involves many difficult and not completely understood problems; however, they also realize that the ultimate safety of a rock slope is more dependent on the kinematics within the controlling geologic discontinuities than on the shear strength of the rock. This paper presents a general treatise on the subject of a rational approach to the design and construction of highway cut slopes in rock.

INTRODUCTION

"The 70-foot-deep cut that you propose at this location will most probably be unstable over the long term, and I don't know how to design the slope to assure stability." Would anyone like to hear such a statement from a design engineer? I think not, but this is exactly the situation that arose on a project in the eastern part of the State of Washington.

I would like to describe the procedures used to overcome such a frustrating problem and the methods which finally evolved for rock-slope design in this and similar geologic areas. The project involved an interstate route constrained for numerous reasons to a section which required several complex cuttings.

GEOLOGY

The regional geology is described as a series of long, anticlinal ridges alternating with synclinal valleys. Thin lava flows and interbedded sedimentary rocks have been twisted and folded, resulting in a broken structure. The lower slopes contain siltstone, sandstone, and claystone and are interspersed with thin, vesicular, badly weathered basalts. In most areas weathering has progressed to the extent that it is difficult to distinguish between overburden and badly deteriorated rock. The water table is far below the surface over much of the route.

Rock was classified for slope-design purposes by fracture density and joint spacing, realizing that the stability of the total slope would depend upon the interlock between joint blocks and the geometry of the cut. Of course, the stability of the sedimentary rock was significantly influenced by the dip of the bedding planes, especially in the non-swelling sandstone and siltstones.

TEST CUTS

At the time of original design for this project, the results of Mr. Bjerrum's work on preconsolidated clays and clayshales had not been published; however, it was well known from construction experiences in these materials that many problems could be anticipated in major cut sections. It was recognized that the known methods of stability analysis for soils using peak shear strengths and normal slope ratios would indicate some safety factors, yet some slopes could eventually fail.

Mr. Bjerrum (1968) describes the swelling tendency in shales as a function of stored energy due to their geologic history. In an unweathered shale the horizontal stress is small and shear strength is high. However, as weathering occurs, the bonds which tie the clay particles together are destroyed and the recoverable strain energy is gradually released. The shear strength of the shale decreases considerably as the shale weathers.

Serious doubts about the stability of the deep cuts resulted in a decision to select a number of the areas within the swelling formations for fully instrumented test sections.

Several test cuttings were constructed under a special contract; however, I will discuss the results of only one of these--one with typically exposed thick strata of shale interbedded with basalt flows and sandstone. A perspective drawing for the test section is shown in figure 1.

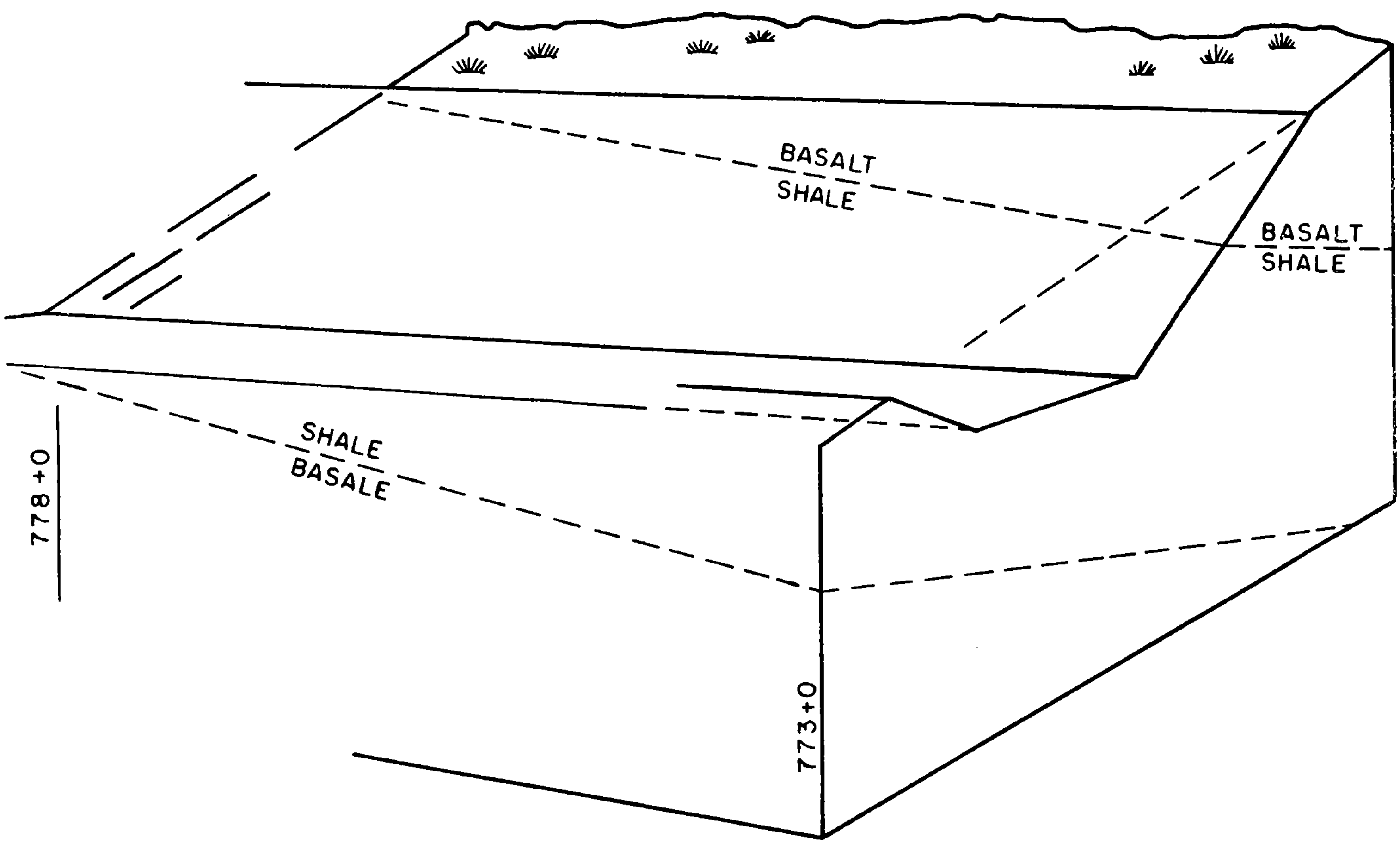


Figure 1. Perspective of Test Cut 775.

INSTRUMENTATION

The cut area, which extended for several hundred feet along centerline with a maximum depth of 80 feet as measured to the top of the cut slope at ditch line, was instrumented as follows.

1. Reference points in the slope face as the excavation progressed were used to measure both horizontal and vertical movements.

2. Several slope-indicator casing were needed to measure lateral displacement. Holes were predrilled to elevations corresponding to roadway grade prior to beginning the excavation process. Sounding of these points was done as the various stages of excavation progressed (fig. 2).

3. "Strain Islands" determined the state-of-stress ratio deep in the ground. The state-of-stress ratio, normally called K, is the horizontal pressure divided by the vertical overburden pressure. In soil mechanics, K at-rest is usually considered to be from 0.5 to 0.6. However, due to previous experience in this area, it was felt that K could be around 1.0. In hard rock, K is most often determined by overcoring techniques. In this instance, the rock was not considered to be competent enough for accurate use of such procedures. The "strain-island" concept, devised by the soils consultant, involved several 10-foot square areas with reference points set below the level surface of each island in a rectangular pattern similar to a standard 45-degree strain rosette.

These islands were placed near the bottom of the cut to reflect the influence of at least a 70-foot overburden. Readings from the strain islands were used to determine appropriate K values for design purposes. Final analysis of the readings resulted in a K of 1.3 where the roadway centerline paralleled the anticlines and where the centerline was perpendicular to the anticline, a K of 1.0.

4. Finally, down-hole bearing tests were performed to evluate shear modulus. Young's modulus was then computed from these data using a Poisson's ratio equal to 0.25. This 0.25 value was assumed to account for very small catch-point movements; the value was later verified by finite-element analysis.

SHEAR STRENGTH VALUES

In this particular geologic formation, the claystones are highly plastic but not fissured. The predominant clay mineral is montmorillonite with measurable quantities of quartz and feldspar. The swelling pressures developed during laboratory expansion tests did not exceed 2,000 pounds per square foot with significant rebound ceasing after four cycles of loading and unloading. Mr. Bjerrum uses this criteria as an indication of strength and bonding between individual clay particles. This bonding has the character of a welding of contact points between minerals. The welding is gradually destroyed as the claystone is exposed to weathering and the locked-in strain energy is ultimately released. This phenomenon can and does result in progressive slope failures.

Stress-strain test results on intact samples of the claystone are shown in figure 3. Claystones are extremely brittle materials compared to ordinary

Figure 2. Typical cross section of test cut.

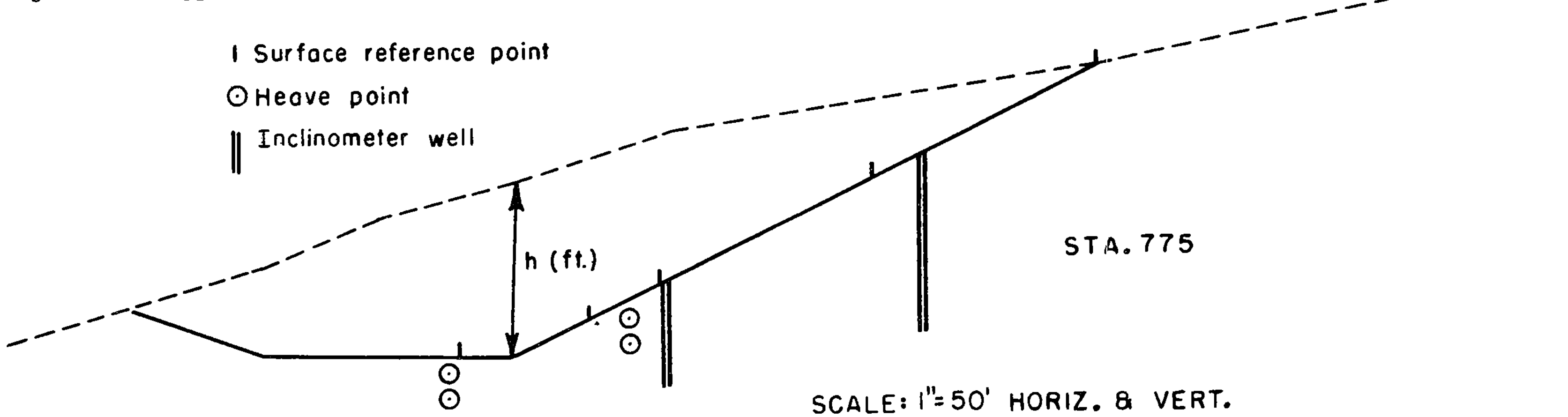
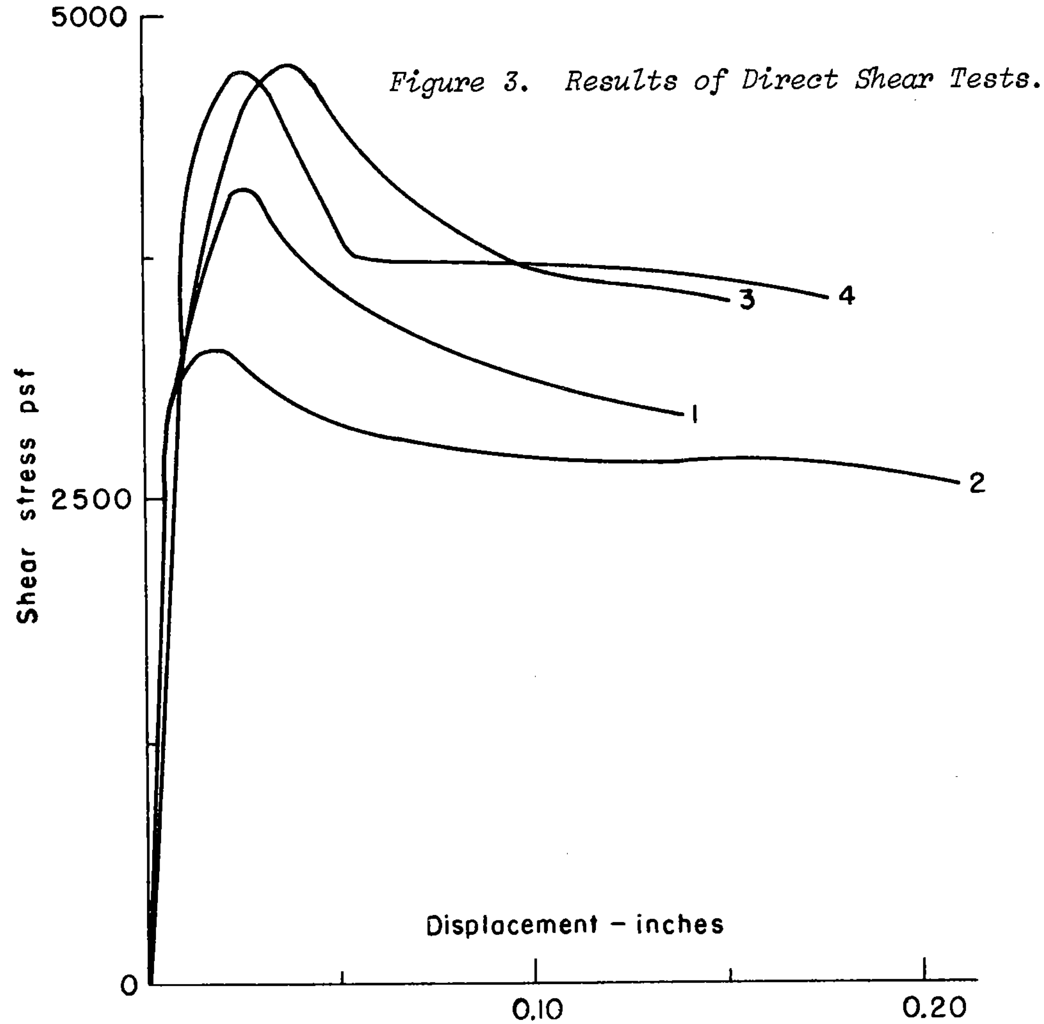


Figure 3. Results of Direct Shear Tests.



Direct Shear Tests
Undisturbed Samples
From Sta. 773 L[R]
Norman Load 1800 PSF

No.	Moist. Cont.	Dry Density
1	26.9%	90 PCF
2	28.4	87
3	22.1	95
4	20.0	97

clays and are subject to failure at very small strain values. As strain increases, the strength exceeds the so-called peak values and is reduced to the residual strength--a fraction of the peak strength. Measured residual strength angles in the claystone averaged about 10 degrees.

STABILITY ANALYSIS

Materials in which values of K exceed 0.7 cannot be accurately analyzed by the commonly used limit-equilibrium methods of stability analysis. Quite simply, the assumptions would not take into account the obvious overstress conditions which would occur at the toe of the cut section (fig. 4).

With development of the finite element technique and its subsequent application to soil mechanics, it is possible to approximate stress in the ground. In the past, most published finite element analyses have been for purely elastic cases. In this approach, if the computed maximum shear stress does not exceed the shear strength of the slope-forming material, the slope will be stable.

Looking more critically at the elastic procedure, if the shear stress at the toe exceeds the shear strength of the material, the analysis would not necessarily show the total slope to be unsafe. In reality, the material at the toe begins to yield plastically, with the stresses at the yielding section held to a maximum of the material strength. A redistribution of stresses adjacent to the overstressed zone results. Thus, from a purely elastic approach there are two possibilities.

1. The shear stresses never exceed the allowable strength at any point along the potential failure surface, giving the total slope a very high safety factor; or

2. local overstress occurs at the toe area, but because the upper part of the slope is understressed the total slope safety factor remains greater than one. In this condition, plastic yield is occurring in the overstressed elements and the stresses computed for adjacent elements are lower than exist in the natural position.

The obvious solution to such a problem is the development of an elastic-plastic procedure. The consultant for this project developed a technique wherein iteration of the various material moduli values leads to a reduction of the computed stresses in the elements at the toe to a maximum of the shear strength of the claystone. This is the process which would take place in the ground. However, as the stress redistribution occurs in the iterative process, the computed changes in strain increase--particularly in the area of the toe. When the computed stress at the toe is considerably greater than the maximum shear strength, the change in shearing strain continues to increase greatly in value; a condition which warns of impending instability.

The stress-strain properties of the claystones found on this project are typical of a brittle material. The peak shear strength is reached after a low change in strain (about 1.5 percent); afterward, the strength drops

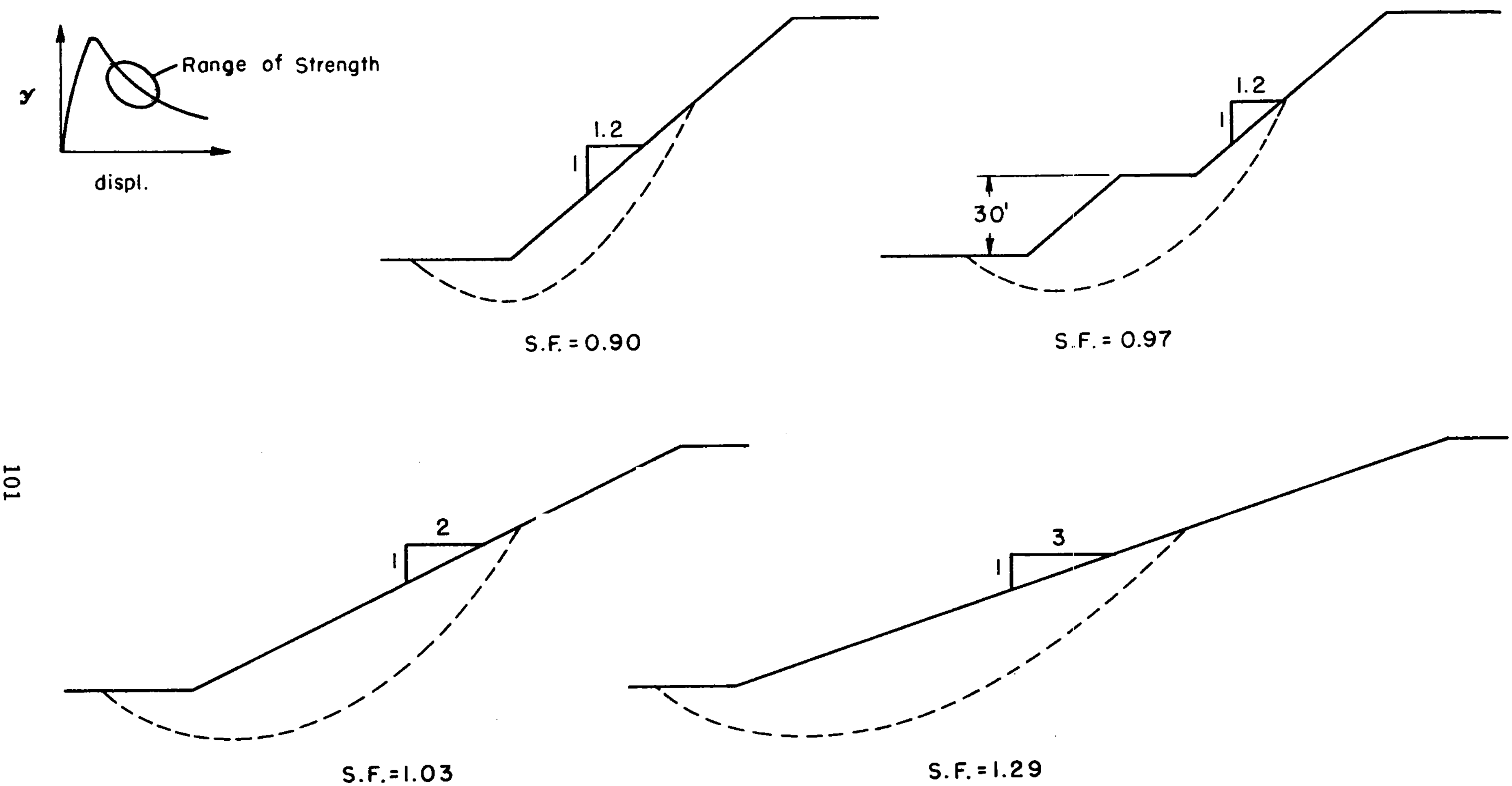


Figure 4. Relative safety factor against flow failure after local yield of overstrained toe. By conventional slip-plane analysis.

off to a lesser residual value for large changes in strain. Thus, for this analysis a 1.5 percent change in strain was selected as a limiting value for any element. A trilinear substitution was used in the finite element method to approximate the actual stress-strain relationship (fig. 5).

Figure 6 shows the consultant's plot of shearing strain versus cut height. Note that the curves show rapid increases in shearing strain with small increases in cut heights beyond certain critical values. Obviously, such conditions would ultimately result in failure. Safe slopes must be relegated to cut heights below this critical area of the curves.

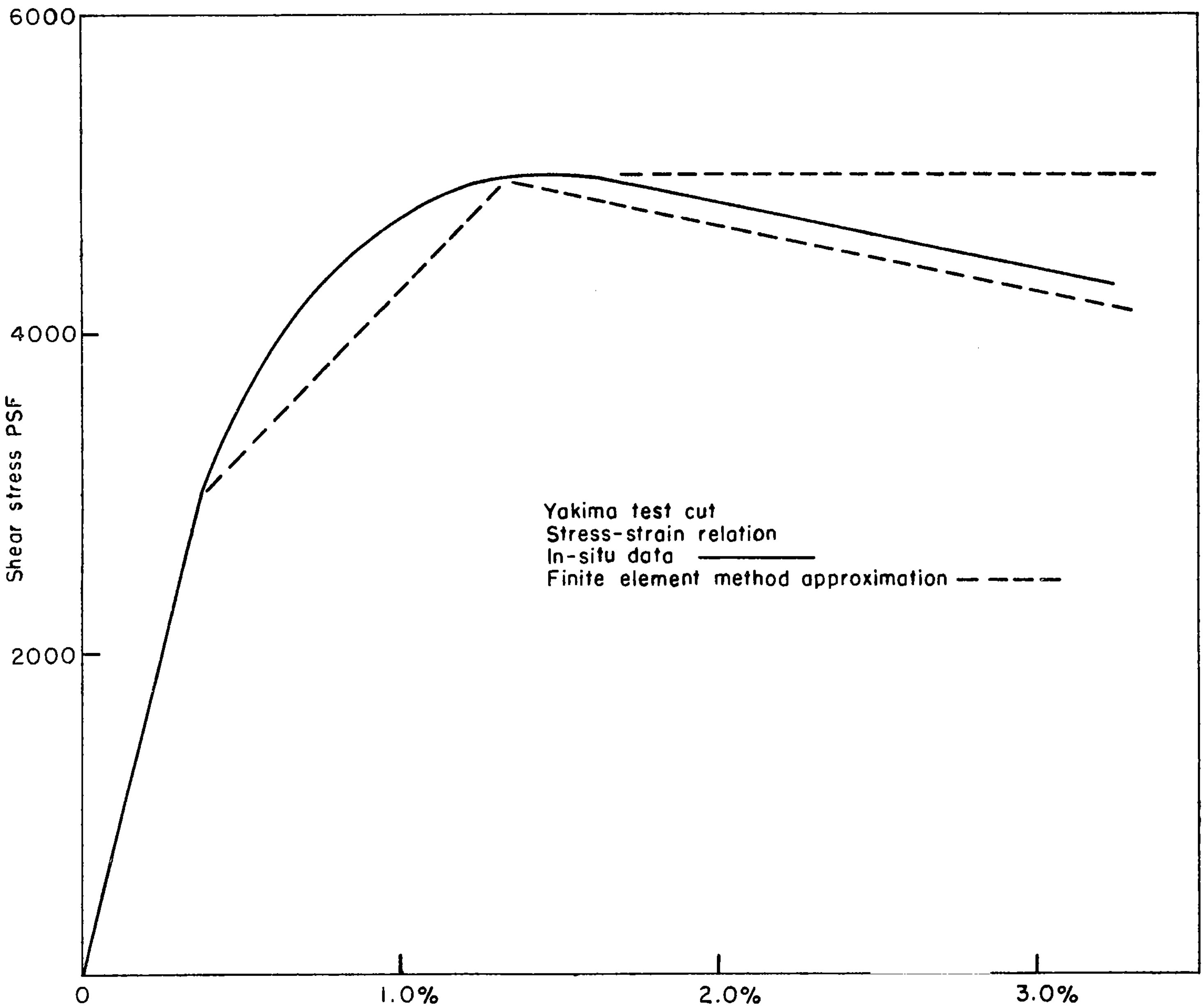


Figure 5. Shear strain--percent.

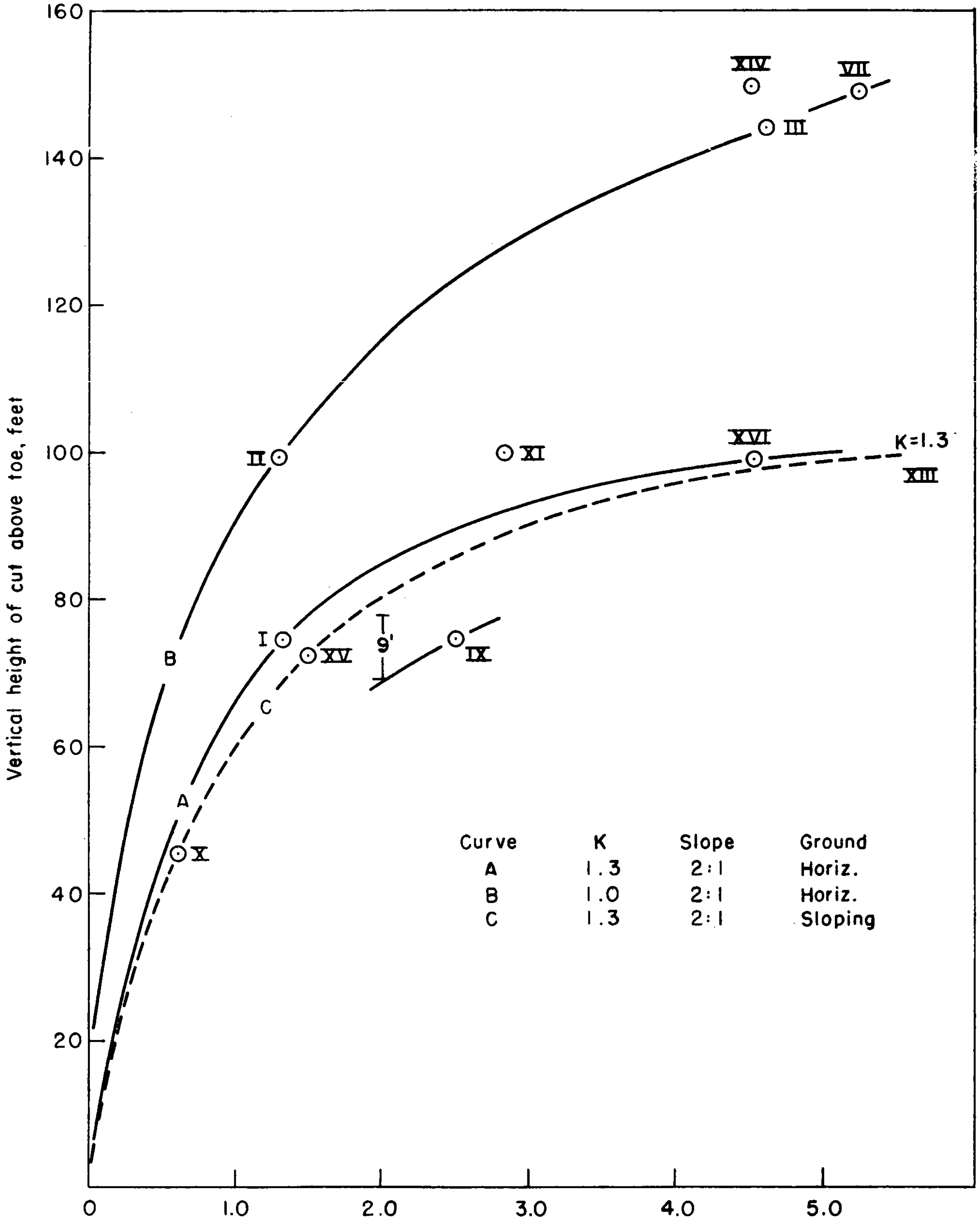


Figure 6. Shear strain--percent.

CONCLUSIONS

Perhaps the most important result of this test-cut project was the realization that stability was primarily controlled by height of cut. Slope angle played a very minor role, i.e., flattening a slope from 1½:1 to 2:1 permits an addition of 10 feet to the maximum safe cut height. A second conclusion is related to the inefficacy of buttresses to control excessive shear strains. A rather large buttress (30 to 50 feet high) has less effect than flattening the slope to 2:1.

Thus, on this project the most effective means of assuring slope stability in overconsolidated clays, claystones, and shales is to limit the maximum height of cut. Flattening existing slopes will reduce ultimate strains only slightly but will reduce the chances for partial failure at the toe to progress into a large flow slide. Benches are not effective due to the stress concentrations at the toe of each bench.

Since the water table has a considerable influence on values of Poisson's ratio and effective stresses, the general conclusions suggested above do not apply in areas where a ground-water table is shallow. Also, in an ancient slide area where the slip plane has been clearly defined, conventional stability analysis should be used, with residual stress values as upper limits for allowable strength, instead of the finite element elastic-plastic approach.

REFERENCE CITED

Bjerrum, L., 1968, Progressive failure in slopes of overconsolidated plastic clay and clay shales: Norwegian Geotechnical Inst. Pub. 77.

THE USE OF AIR AS A DRILLING MEDIUM FOR SUBSURFACE INVESTIGATION

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Abstract.--The paper presents a summary of methods and techniques utilized in the development of air drilling in West Virginia for foundations and slope design during the past decade.

The report presents such pertinent information as the advantages and disadvantages of the use of air as compared with other types of drilling fluids. Other information reported concerns the types of bits and air compressors that have proved satisfactory, along with other necessary support equipment.

INTRODUCTION

The purpose of most highway geologic drilling is to classify the rock for slope and foundation design; therefore, knowledge of the quality of the rock may or may not be available prior to taking the borings. If the quality, thickness, and locations were known prior to obtaining the borings, there would be no need for the borings. Many of the borings in the Mississippian, Pennsylvanian, and Permian rocks of West Virginia show that these rocks often grade from one type to another in minimal distances owing to differences in sedimentation (see fig. 1), and that changes occur also because of structural deformation. Thus borings anticipated to penetrate shale may actually penetrate sandstone or vice versa. The terrain is not ideal for moving on and setting up at many drilling locations owing to the slope of the ground line and the differential elevation. The site may be many feet from a source of drilling fluid horizontally as well as a great distance vertically (see fig. 2). Because of these restraints the drilling is often done with air as a drilling medium. There has been considerable interest in regard to the use of air when the use of water becomes impractical or quite costly. This report has been developed to provide a comparison of air as a drilling medium with several types of drilling fluids.

Three types of drilling media have been used to cool the bit and remove the cuttings. They are water, bentonite mud, and air. Water is most often used because of its availability, low cost, and--in most instances--adequate performance. Often, however, there is a need to keep the hole clean, prevent the loss of drilling fluid, or maintain an open hole without the use of casing; bentonite mud performs these functions very well. It does, however, require a recirculating area, and it involves the cost of the mud and usually of different pumping equipment and core barrels. In mountainous terrain, where borings are relatively shallow (less than 150 feet) and water is either very scarce or not available, air becomes a very satisfactory substitute for water. Air can also substitute for bentonite mud where drilling is in slaking shales or where there is a slight loss of drilling fluid.

Figure 1.

Figure 2.

ADVANTAGES OF AIR

The use of air for drilling during the winter months offers considerable advantages over the other media, since temperatures below freezing have little effect on the flow of air and care need not be exercised to prevent freezing during short shut-down periods. Air can be acquired at any drilling site without payment or laying long lengths of hose, whereas water in urban areas or during the dry season may require hauling and payment or may be difficult to obtain. Mud may be equally difficult to obtain and may cause additional problems of cleanup in urban areas. Air performs excellently in drilling soft shales that disintegrate when in contact with water. The air tends to dry the shale and keep it dry during the drilling period.

Air assists the geologist or driller responsible for logging the boring in that there is immediate return of cuttings to the surface in shallow borings. The cuttings indicate immediately any change in color and texture. Cuttings from roller bits, when penetrating medium-hard to hard rock, are usually aggregates of particles, and the formation being penetrated can be logged by examining these particles. The softer formations normally degrade to a fine dust. Thus the type of return can often provide an indication of the hardness of the material. Since the diamond core bit produces footage either by abrasion or destruction of the bond between rock particles, only dust or particles are returned to the surface, and it is usually difficult to surmise the cementing material between particles until the actual core is removed.

The air pump or compressor can be located at any convenient location, even on the drilling rig (see fig. 3), and it is not restricted by having to lift a column of heavier drilling fluid to the drill head. Conversely, the use of water in mountainous terrain may require two pumps to supply the

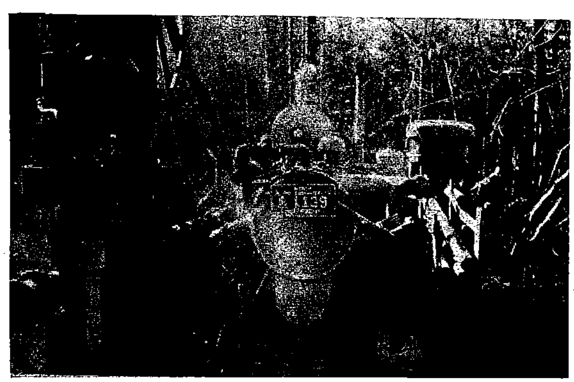


Figure 3.

volume and pressure. Both water and mud pumps must be close to the supply or recirculation area owing to suction lift limitations. Air is not restricted by these limitations because it does not reduce the weight of the tools as do water and bentonite mud and because there is no problem with settling out of the cuttings during recirculation.

The pumping characteristics of air and water remain constant, whereas mud viscosity must be maintained for proper performance. Also, with air there is no waiting period for the proper viscosity to be reached.

DISADVANTAGES OF AIR

Unfortunately, air is not a perfect medium either, and has several inherent characteristics that cause performance problems.

Most borings contain small amounts of moisture that will collect overnight; therefore, swelling or caving shales are best drilled during a single shift. A major problem can occur when drilling with air if small amounts of moisture enter the boring from aquifers. The cuttings collect at the point of ingress and restrict the annulus. This causes a reduction in the flow of air and often makes removal of tools difficult. For example, where this condition prevails a ring of cuttings builds up, and when the larger diameter portion of the tools, such as the core barrel, tries to pass during removal it may jam or lock the tools and cause a fishing job. The normal remedy is to dump several barrels of water in the annulus to lubricate the tools and float the cuttings away from the built-up area.

Larger amounts of moisture do not restrict the use of air; however, the cuttings are not carried to the surface as well with smaller compressors, and when sufficient pressure and volume are built up the air is accompanied by an occasional surge of water. Large compressors normally expel the accumulated water from the boring immediately when air circulation is started. The cuttings from the annulus, however, continue to be damp or wet.

Larger amounts of moisture also cause problems with the surface exhauster system. The exhauster system is a blower, which takes the cuttings and air away from the work area.

The cost of equipment needed for utilizing air is considerably higher than for other media. Air compressors are both more expensive at initial purchase than water or mud pumps and have higher maintenance and operational costs.

As mentioned earlier, an exhauster system is required at the surface for carrying the cuttings and air from the annulus outside the work area or other objectionable areas. When an exhauster is not used the cuttings will contaminate the bearings, gears, and frictional surfaces of the drill-head compressor and motors, considerably reducing the life expectancy of the machinery. The air and cuttings must also be blown into an area where they will not be inhaled by animals or humans for health reasons.

The operating temperature is considerably higher when drilling with air than with bentonite mud or water. The temperature is usually not a detrimental factor in cooling the bit except in very hard formations. The operating crew, however, must use gloves for protection when handling the tools.

EQUIPMENT REQUIRED

Borings with NX-size tools utilizing air as a drilling medium require accessory tools of a certain size for proper performance during optimum conditions. The smallest compressor that is adequate for removing the cuttings from borings that do not penetrate more than approximately 40 feet should be capable of furnishing 105 cubic feet per minute (cfm) at an operating pressure of 90 pounds per square inch (psi). The compressor size that has been found to perform satisfactorily over long periods of time with minimum maintenance is a compressor similar to the Gardner Denver Model WCR1000, which produces 213 cfm at pressures of 50 to 100 psi. This compressor has provided sufficient volume in borings that do not exceed 200 feet in depth. The Department experimented with a rotary vane type of compressor, which produced 227 cfm at a working pressure of 50 psi; however, this compressor was never successful for borings that penetrated more than 50 feet. One of the core-barrel manufacturers recommends compressors producing 315 cfm and capable of pressures between 90 and 200 psi for depths less than 500 feet.

The type of core barrel that is most successful with air is the same type that is used for drilling with bentonite mud. The heavy-duty double-tube barrel usually has an adjustable inner barrel that allows for control of flow characteristics and proper clearance with the bit. The Department does, however, use regular series M double-tube barrels for drilling with air. Barrels capable of making both 5- and 10-foot core runs are used, depending on the condition of the rock and the capabilities of the drilling rig.

The bit design normally used is similar to that for drilling with bentonite mud. The present design used by West Virginia utilizes a wider kerf than that specified by the manufacturer of the barrel. The manufacturer's design will also perform well, but the wider kerf reduces the abrasion on the barrel and eliminates the need for a diamond-set reaming shell. The development of the wider kerf during the last 10 years has produced a bit that is satisfactory for most sedimentary rock found in West Virginia, and since most borings are shallow there usually is no need for changing to a new bit during completion of a boring. West Virginia does not have igneous rocks of any sizable quantity, so we are unable to offer insight regarding a suitable design for these harder rocks. We are occasionally required to drill in rock that has been subjected to some low-grade metamorphism; although this material can be penetrated using air, it is usually more economical to use water, at least with our present bit design.

The quality of diamonds used in the core bits is determined by the U.S.

Army Corps of Engineers grading system, which has quality ratings of WA-1, WA-2, and WA-3.

The design for bits used in the past to perform borings in soft to medium-hard shales and soft sandstone incorporated a stone density of 15 stones per carat (fig. 4). The setting utilized in the past for hard shale and medium-hard to hard sandstone and limestone was a density of 20 stones per carat (fig. 5). For very hard sandstone and limestone the normal density of stones per carat was designated at 40 (fig. 6). WA-1-quality diamonds are specified for settings for drilling very hard sandstone and limestone. WA-2-quality diamonds are specified for softer formations. The Department is currently researching a new design for the bits that will utilize nine stones per carat for drilling the soft to medium-hard shales and soft sandstone (fig. 7). The 20 stones per carat have been maintained for the medium-hard to hard sandstone and limestone (fig. 8). The face for the very hard sandstone and limestone has been designed to include 54 stones per carat (fig. 9). A sparse setting has been used in all cases in order to obtain as much loading per stone as possible with the limited loads available with our light skid drills and shallow borings. If the shale, sandstone, or limestone has undergone metamorphism, it will usually require the best quality of diamonds and a smaller stone size.

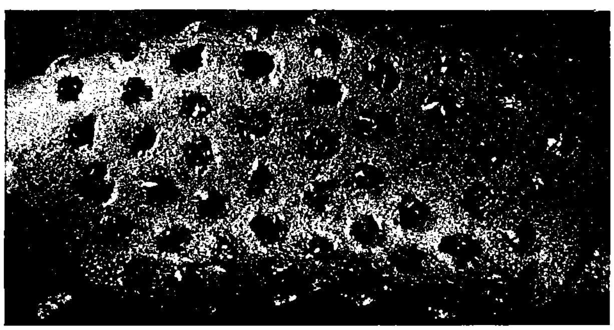
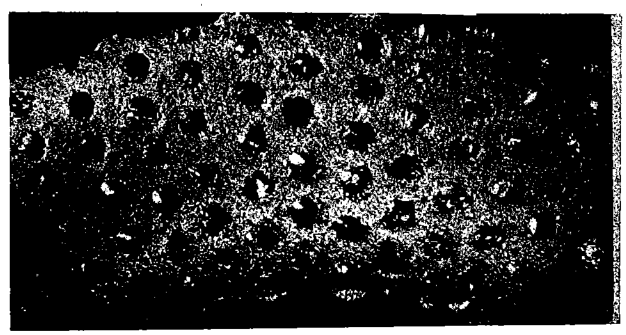


Figure 4.



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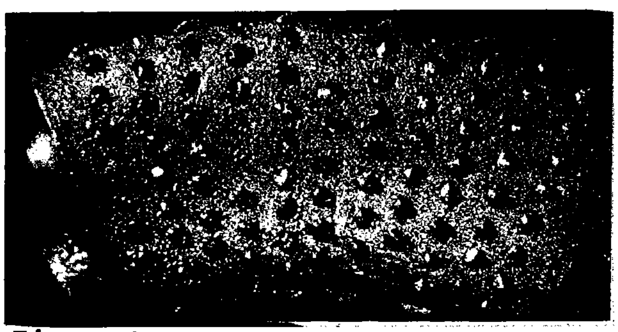

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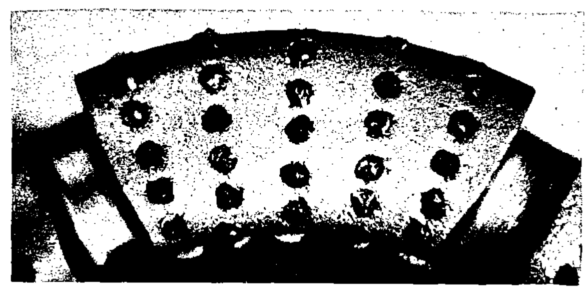

Figure 7.

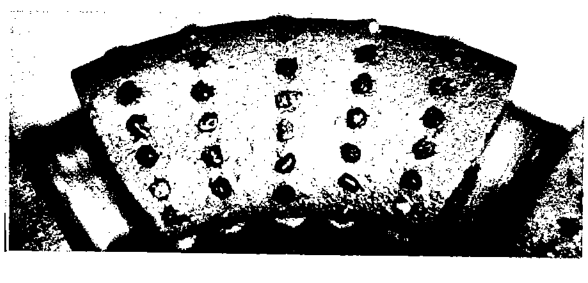

Figure 8.

Figure 9.

The face design currently being used is a semiround face with six unreinforced airways (fig. 10). The current dimensions of the airways are 3/16 inch wide by 3/32 inch deep. For both types of NX-size barrels, the outside diameter has been increased. For the NXM barrel it has been increased from 2.9375 to 3.125 inches. For the heavy-duty mud and air barrel it has been increased from 2.980 to 3.125 inches. The carat weight varies with the size of stones used. The original bit design recommended by the bit manufacturer for the 20-stone-per-carat density for drilling with air contained approximately 35 carats. It was discovered that this was too high a carat weight, which resulted in a low loading per stone. It has since been reduced in stages from 32 carats to 27 carats, and a current redesign is being developed that will have only 19 carats for the 20-stones-per-carat bit, which is considered a general-purpose bit. It is quite important, especially in dealing with the harder formations, to tailor the total carat-contact area in the face of the bit to the available down pressure of the drill rig. Another method is to determine the pounds per square inch per diamond necessary for embedding the diamond into the formation to be drilled. The bits just described work equally well when drilling with water.

The rotational speed is controlled to deliver as smooth an operation as possible between a range in revolutions per minute of 100 to 350. The rotation recommended for NX-size tools by many authorities is about 700 rpm. However, drilling equipment that is capable of both coring and augering will often not perform above 500 rpm. Our experience in regard to rotational speeds has indicated that most of our operating and formation conditions require rotation speeds below 300 rpm because of vibration. Since the drilling rate varies approximately linearly, and proportionately with rpm for a given load, the obvious result of slower rotation is a lesser amount of footage per unit of time; often, however, it also ensures a better core recovery. The down-pressure or loading capability of our current drills varies from a low of 2,200 pounds to a high of 5,650 pounds. The down pressure must be varied during drilling according to the rock hardness and the carat and stone density and condition of the bit being used. In other words, when the total area of diamonds on the face of the bit increases, a greater down pressure is required. This becomes much more critical in the harder formations, since the diamonds must have sufficient force applied to be embedded into the formation. Insufficient down pressure or applied load can result in increased temperatures at the diamond contact area. If the diamonds are not embedded sufficiently in the formation to properly destroy the bond or abrade the formation being drilled, considerable friction can develop in the reduced contact area. This condition can result in poor cooling and removal of abraded material, owing to enlargement of the area between the bit face and the formation. Since air does not cool the bit as well as do the other media, and since it does provide oxygen, it may result in subliming the diamonds within the area of the contact if the temperature approaches 3,500°F. When drilling the harder formations with either improper bit design or loading, the bit is often damaged by polishing, abrasion, or burning, as some agencies claim. The polishing and abrasion of the diamonds is accelerated if any of the material displaced from the bit or the formation is not removed.

In evaluating the face of our former bit design, it was discovered that the area of the stones set in the face for the hard formation was the same as that for the softer formation, even though the carat weight had been reduced. It is recommended that for harder formations the area be reduced as much as possible by either reducing the carat weight or reducing the stone size and carat weight.

The size of the tricone roller bits used have either a 3 1/2-inch or 3 7/8-inch outside diameter (fig. 11). Only bits of one hardness are purchased, and they are designated as being capable of drilling very hard rock. The tricone roller bits can normally operate at the maximum down pressure available for most small drilling rigs, since there is usually insufficient down pressure to cause the teeth to be fully embedded in the formation being drilled except for very soft shale. These roller rock bits perform relatively well with the larger rigs and moderately well with the smaller skid rigs. They also perform equally well with water. However, bits designed to use water may need modifications to be used with air, such as closing the ports to the bearings and (or) welding a restrictive washer inside the bit.

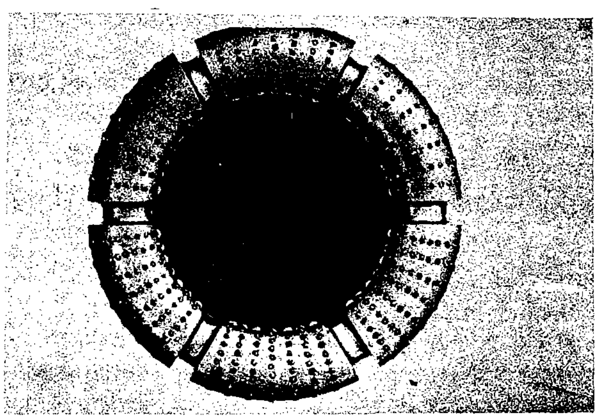

Figure 10.

Figure 11.

Where the weight of the drill is not sufficient to provide penetration of the diamonds or teeth, the rig may have to be restrained by tie downs.

COST COMPARISON

In order to evaluate the effectiveness of two of the media, water and air, drilling time and cost per foot were compared. The cost per foot includes maintenance and operational cost, bit and drilling-tool cost, and salaries for the crew. Twenty-three projects drilled with eight different rigs were evaluated for a period between November 1969 and January 1971. The projects selected had a preponderance of rock drilling, and only the most recent work was used so that the data would reflect current costs rather than preinflationary values. The drills used in the evaluation were three Acker Hillbilly skid drills, one 12B Joy skid drill, one Acker MP 50 skid drill, one Reich drill mounted on a D-6 bulldozer, one Mobile Strokemaster drill mounted on a D-6 bulldozer, and one Mobile B-40 skid drill.

When comparing the cost per foot for the projects evaluated, the bulldozer-mounted drills operated at an average cost of $5.65 per foot as compared to $9.71 per foot for the skid-mounted drills. The average cost per foot to drill with air was $9.04 as compared with $7.11 for drilling with water or an increase of about 27 percent (table 1). These averages were obtained by simply summing the project average cost per foot for each type of medium and dividing by the number of projects for each medium.

Another method, which may be more realistic, is to total the expenditures for air drilling, which were $50,874.63. When this expenditure is divided by the total footage for air of 5,864.2 feet, the resultant cost per foot is $8.68. To determine the cost per foot for water by this method, the expenditure for drilling with water, $43,520.04, was divided by a total footage of 5,894.2 feet, resulting in a cost per foot of $7.38.

The average feet per day drilled by all the drills with air was 18.3, as compared with 20.5 feet per day with water. The skid drills averaged 15.6 feet per day drilling with air and 17.4 feet per day with water. The heavier bulldozer drills averaged 28.3 feet per day drilling with air and 25.6 feet per day drilling with water.

Since almost half the projects drilled with air were drilled during the winter months, the cost for the winter operations was extracted for evaluation. It was $10.35 as compared with $7.55 for projects drilled with air during the remainder of the year, or an increase of approximately 37 percent. Since winter operations are dependent upon weather, they are normally more costly. Experience available concerning use of water on winter projects is insufficient to provide significant data at this time.

The cost in drilling with air varied considerably, from $3.67 to $17.17 per foot. This wide range can probably be attributed to the time of year involved and the type of drilling used.

Seven of the projects drilled with air were drilled in areas of hard to very hard stone as compared with only three projects drilled with water, which had formation hardnesses of hard to very hard. The comparison of the projects drilled with air in hard to very hard formations resulted in a cost of $9.99 per foot as compared to $6.60 when drilled with water. Also, much of the drilling performed by the Department's own forces comprises projects that have a high priority and either are too small to contract or are projects where conditions are such that commercial drillers have indicated by their bid prices or lack of bidding that the work will entail higher risks and (or) greater projected costs than normally encountered. Therefore, the prices included are higher because of the noted reasons.

OPERATIONAL PROBLEMS

Experience with air for flushing the boring has pointed to several areas

Table 1.

COST COMPARISON

AIR

COST PER FT.	% CORE	% ROLLER BIT	SEASON	ROCK HARDNESS	TYPE INVESTIGATION
14.36	33	67	WINTER	M.H. SHALE & M.H. SANDSTONE	WALL FOUNDATION
14.06	—	100	WINTER	M.H. SHALE	LANDSLIDE
3.67	56	44	WINTER	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION & SLOPE DESIGN
6.33	50	50	WINTER	V.H. SHALE & V.H. SANDSTONE	BRG. FOUNDATION & SLOPE DESIGN
* 14.84	32	58	WINTER	M.H. SHALE & H. SANDSTONE	SLOPE DESIGN
17.12	53	47	WINTER	V.H. SANDSTONE	BRG. FOUNDATION
11.62	—	100	SUMMER	M.H. SHALE & V.H. LIMESTONE	SLOPE DESIGN
7.90	41	59	SUMMER	V.H. SANDSTONE & V.H. LIMESTONE	BRG. FOUNDATION
* 5.92	40	13	WINTER	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION & SLOPE DESIGN
* 6.50	38	58	WINTER	V.H. SHALE-SANDSTONE & LIMESTONE	SLOPE DESIGN
* 5.90	—	61	SUMMER	SHALE & V.H. LIMESTONE	SLOPE DESIGN
* 4.89	—	82	SPRING	M.H. SHALE & M.H. SANDSTONE	LANDSLIDE
* 6.44	47	35	SUMMER	M.H. SHALE & M.H. SANDSTONE	SLOPE DESIGN
* 8.78	33	1	FALL	M.H. SHALE - COAL & M.H. SANDSTONE	SLOPE DESIGN
7.20	—	100	FALL	M.H. SHALE	LANDSLIDE

AVERAGE COST PER FOOT = 9.04

AVERAGE DRILLING TIME = 18.3 FEET PER DAY

AVERAGE % CORE = 28.2 PER DRILL PER PROJECT

AVERAGE % ROLLER ROCK BIT = 58.3 PER DRILL PER PROJECT

* REMAINDER OF BORING SOIL SAMPLING OR AUGER BORING

WATER

COST PER FT.	% CORE	% ROLLER BIT	SEASON	ROCK HARDNESS	TYPE INVESTIGATION
4.85	42	58	SPRING	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION
8.40	19	81	SUMMER	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION & SLOPE DESIGN
9.46	46	54	SPRING	H. SANDSTONE	BRG. FOUNDATION
5.70	43	57	SUMMER	M.H. SHALE & BLDRS.	SLOPE DESIGN
5.70	43	57	SUMMER	M.H. SHALE & BLDRS.	SLOPE DESIGN
* 5.91	58	24	SUMMER	M.H. SHALE & M.H. SANDSTONE	SLOPE DESIGN
* 5.60	33	44	SUMMER	M.H. SHALE & V.H. SANDSTONE	BRG. FOUNDATION
* 4.74	—	79	SUMMER	M.H. SHALE & V.H. LIMESTONE	SLOPE DESIGN
9.30	5	95	SUMMER	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION
8.02	46	54	FALL	M.H. SHALE & M.H. SANDSTONE	SLOPE DESIGN
9.24	38	62	FALL	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION
7.88	22	78	WINTER	M.H. SHALE & M.H. SANDSTONE	BRG. FOUNDATION

AVERAGE COST PER FOOT = 7.11

AVERAGE DRILLING TIME = 20.5 FEET PER DAY

AVERAGE % CORE = 24.6 PER DRILL PER PROJECT

AVERAGE % ROLLER ROCK BIT = 61.9 PER DRILL PER PROJECT

* REMAINDER OF BORING SOIL SAMPLING OR AUGER BORING

where considerable supervision is necessary to maintain satisfactory drilling rates and conditions.

Equipment maintenance is essential to insure that the compressors are kept in good condition, since they normally operate more continuously than do compressors for other types of service. Also, the exhauster system must be periodically cleaned because the hoses tend to collect dust and if not serviced occasionally will not perform their function properly.

There are drilling conditions during which the use of air for a drilling medium is marginally successful. This may occur when extremely hard formations are to be drilled with light equipment or where small amounts of moisture are entering the borehole. In such instances, supervisory decisions are necessary in regard to continuing with air or changing to fluids. Since the hard formations and light equipment tend to cause the diamond bits to polish somewhat sooner because of the higher operational temperatures, a cost comparison should probably be conducted to determine if the use of air should be continued.

SUMMARY

The foregoing cost comparison appears to indicate a higher cost for drilling with air. However, several factors have adversely affected these costs, which should be noted in reviewing the comparison, namely the type of stone drilled, the time of year, and the availability of drilling fluids.

All costs for the Department may be slightly higher than those incurred by larger operations and (or) by other organizations, since the projects used for this comparison varied in total footage from 72.1 to 2,114.6 and are thus relatively small projects. These projects represent operations where in many cases the locations were quite difficult and (or) the project was designated as rush or high priority.

The setup time for a site can often be considerably reduced using air, since there is no limitation as to where the compressor has to be located. Although air performs better if the core barrels and bits have been designed for that use, air drilling can be successfully performed with regular double-tube core barrels. It must be stressed, however, that the air compressor must always be of sufficient size; otherwise the operation will be unsatisfactory.

PENETROHAMMER, THE PENETROMETER MACHINE AND ITS APPLICATION

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Abstract.--The Penetrohammer is a penetrometer used and sold by the Mobile Drilling Company. The State of Kansas uses this tool to penetrate subsurface strata and measure the amount of resistance in various materials. The results obtained from the Penetrohammer, called the penetrometer drive, are available in one-third to one-eighth the time required to drill and sample a hole of the same depth. The penetrometer drive indicates resistant zones, but geologic interpretation is necessary to reveal exact composition. However, geologic-investigation time is shortened by using changes in density as indicated by the Penetrohammer to determine areas for sampling. The 50-second opinion, formulated through Penetrohammer usage, stipulates that when the drive reaches and stays beyond the 50-second point for a depth of 5 feet, pile will probably obtain the necessary bearing based on 40 tons per pile. This guideline is especially useful for granular materials. The penetrometer is also used to indicate the weathered depth of shales and to locate breakoffs in bedrock. Use of the Penetrohammer at temperatures below 20°F is not recommended.

INTRODUCTION

The purpose of this paper is to explain the operation and use of the Penetrohammer. The Penetrohammer is a tool which penetrates subsurface strata and indicates the amount of resistance in various materials. A penetrometer drive, the results obtained from the Penetrohammer, is basically a dynamic-penetration test. The application of the penetrometer drive as related to pile and spread-footing conditions will be shown.

MACHINE

The Penetrohammer used by the State of Kansas is a trailer-mounted unit. The component parts are a Davey Model 125 Hydrovane portable compressor, a Gardner-Denver air winch, and the no. 2 McKiernan-Terry air hammer.

A penetrometer of this type was first built and used by the State of California. The State of Kansas began to use a machine of the California design in the early 1960's. The name Penetrohammer was given to the penetrometer used by Mobile Drilling Company, which now commercially sells a machine of this type.

The compressor is operated at an air pressure of 110 pounds per square inch. "The small double-acting hammers do not have a rated catalog energy because small variations can change the rating easily. By calculation, the no. 2 pile hammer should deliver 143 foot-pounds per blow at 100 pounds per square inch in the hammer cylinder and 464 blows per minute" (written comm., McKiernan-

Terry, 1960). The air hammer's total weight is 543 pounds, which includes the addition of 200 pounds of weights to the commercially-purchased hammer. The air winch is used to lift the hammer when adding another rod and to remove the rods from the ground upon completion of a drive.

A welded double washer, 2¼ inches in diameter, is driven into the ground on the bottom of an adaption to (1 5/8 inches od) "A" size drill rod. The washer--which is larger than the drill rod to give a theoretical end resistance to the drive and eliminate side friction--is left buried in the ground when the rod is removed.

The Penetrohammer is generally operated by a crew of two, an operator and a recorder. It will complete a drive in one-third to one-eighth of the time required to drill and sample a hole of the same depth (Davis, 1964).

INTERPRETATION

The penetration rate is timed for each foot with a stopwatch and recorded to the nearest second. The refusal time for a drive is 2 minutes for one-tenth foot of penetration. There are locations within the state where difficulty in removing the rods from the ground, causing damage to the equipment, has prevented the 2-minute refusal time from being used. The refusal point was set, therefore, at 100 seconds per foot for a depth of 5 feet.

Information is recorded on a special drive sheet and then plotted on a work sheet. The vertical scale on the work sheet is generally 10 feet per inch and the horizontal scale 100 seconds per inch. As the first 5 feet of the drive is pushed into the ground, no time is plotted. The drive then indicates very low resistance until a few feet above the refusal point. A point of resistance is now determined in the subsurface strata.

The penetrometer drive alone does not give an excellent indication of the type of footing material. The drive indicates resistant zones at different elevations with various configurations. The geologic interpretation of the drill soundings reveals what material is encountered in resistant zones of the penetrometer drive (fig. 1). Drive no. 1 indicates low density in the material until elevation 1270. The geology at this point reveals some cementation, either more dense sand or silty clay with granular material. The material below this buildup is again low in resistance, as shown on the drive record. The refusal point of drive no. 1 is reached very suddenly. This generally indicates hard, dense material such as sandstone, hard bedrock, or large float blocks. Drive no. 2 is similar to the first drive, but the cementation of the material appears to be greater. The curve obtained in drive no. 3 is common in some silty clays, clays, and weathered shales. The penetrometer drive's configurations are meaningless without geologic knowledge.

Through usage of the Penetrohammer, a theory has been formulated concerning what penetration rate indicates. When the drive reaches and stays beyond the 50-second point for a depth of 5 feet, pile will probably obtain the necessary bearing based on 40 tons per pile. While this guideline is not utilized in a hard-bedrock condition, it is usually a reliable formulation with granular materials.

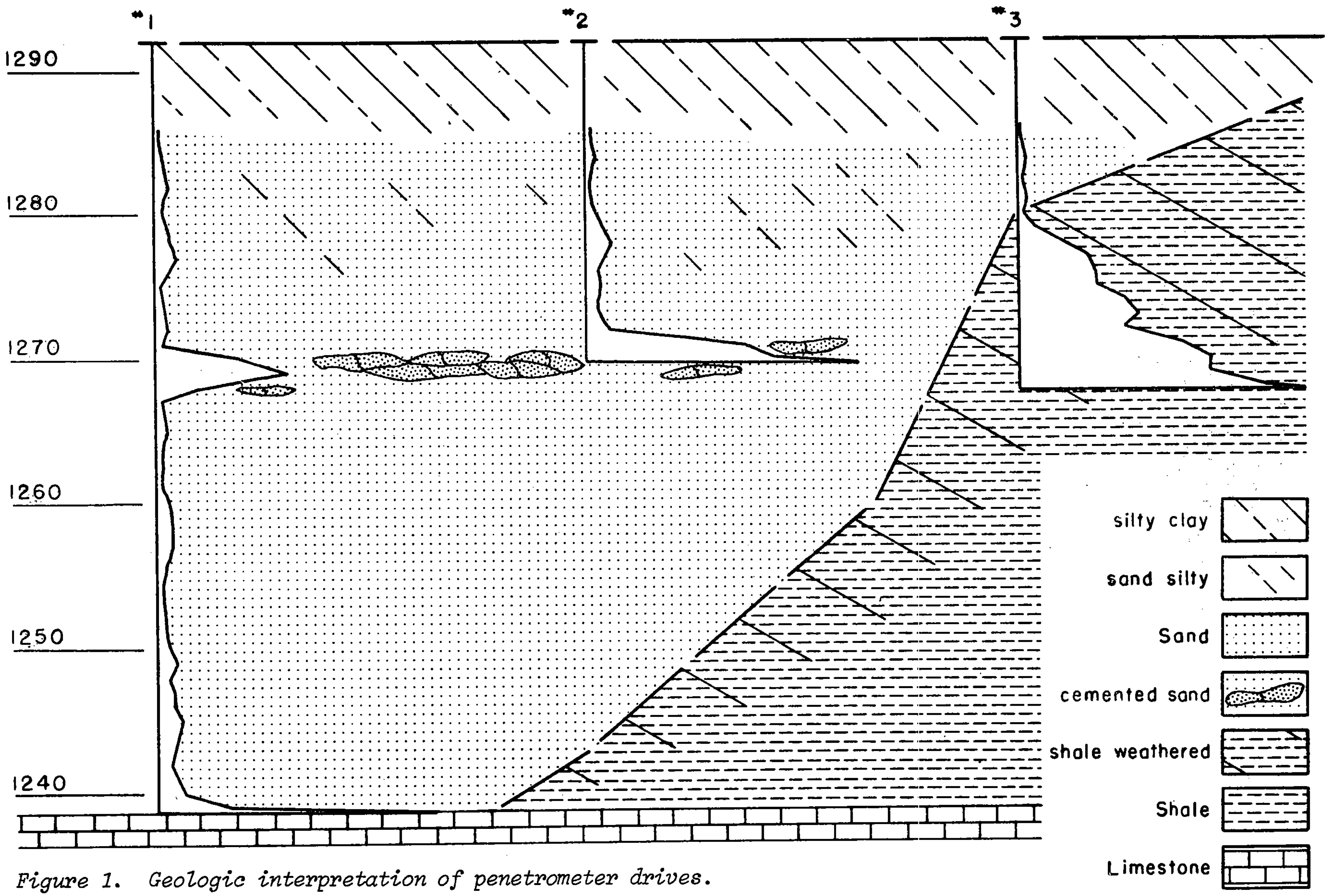

Figure 1. Geologic interpretation of penetrometer drives.

A brief study was made of the 50-second opinion by reviewing the as-built records of bridge locations. These records showed many different types of pile, different types of driving hammers, and varying geologic conditions. Only bridges that had both test-pile and penetrometer drives were selected for consideration. Thirty test piles, using the 10 3/4-inch pipe pile and the Delmag D-12 hammer in sediments of Tertiary and Quaternary systems, were used to arrive at some conclusions about tonnage versus Penetrohammer. The comparison tonnage was set at 40 tons versus the Penetrohammer. Results are shown in table 1.

Table 1. Evaluation of 50-Second Opinion
Comparison of Test-Pile and Penetrometer Drives[1]

	Pentrohammer (seconds per foot)	Pile (feet below cutoff elevation)
All drives used	52.4	38
Pier only	64.4	36.4
Abutment only	40	39.6

[1]Based on 40 tons bearing

These are averages and should be used as such because of many variables. No study has been made of the Penetrohammer results for different types of material, because of the lack of laboratory information. Generally the results of drives are used as trends evident in different areas of the state. The information from this brief study indicates the 50-second opinion is a starting point in picking pile lengths.

The pile on most of our bridges are designed for 37 tons of computed bearing and are planned for some cutoff rather than splicing. The point at which the required tonnage is obtained is not shown on the completed pile reports. This results in an information gap between where the pile obtains the necessary bearing and the designed lengths.

PRACTICAL APPLICATION OF THE PENETROHAMMER

An examination of completed bridge sites will indicate the practical application of the Penetrohammer. It was stated previously that the penetrometer drive alone does not give all of the necessary information; reverse the situation and use only the drill-sounding information (fig. 2). Can the 40-ton bearing point be determined from only the drill-sounding information? The addition of the penetrohammer drives gives a much clearer picture of resistant zones (fig. 3). In figure 4 the test piles have been plotted against the penetrometer drives and the drill soundings to indicate the usefulness of the penetrometer drive.

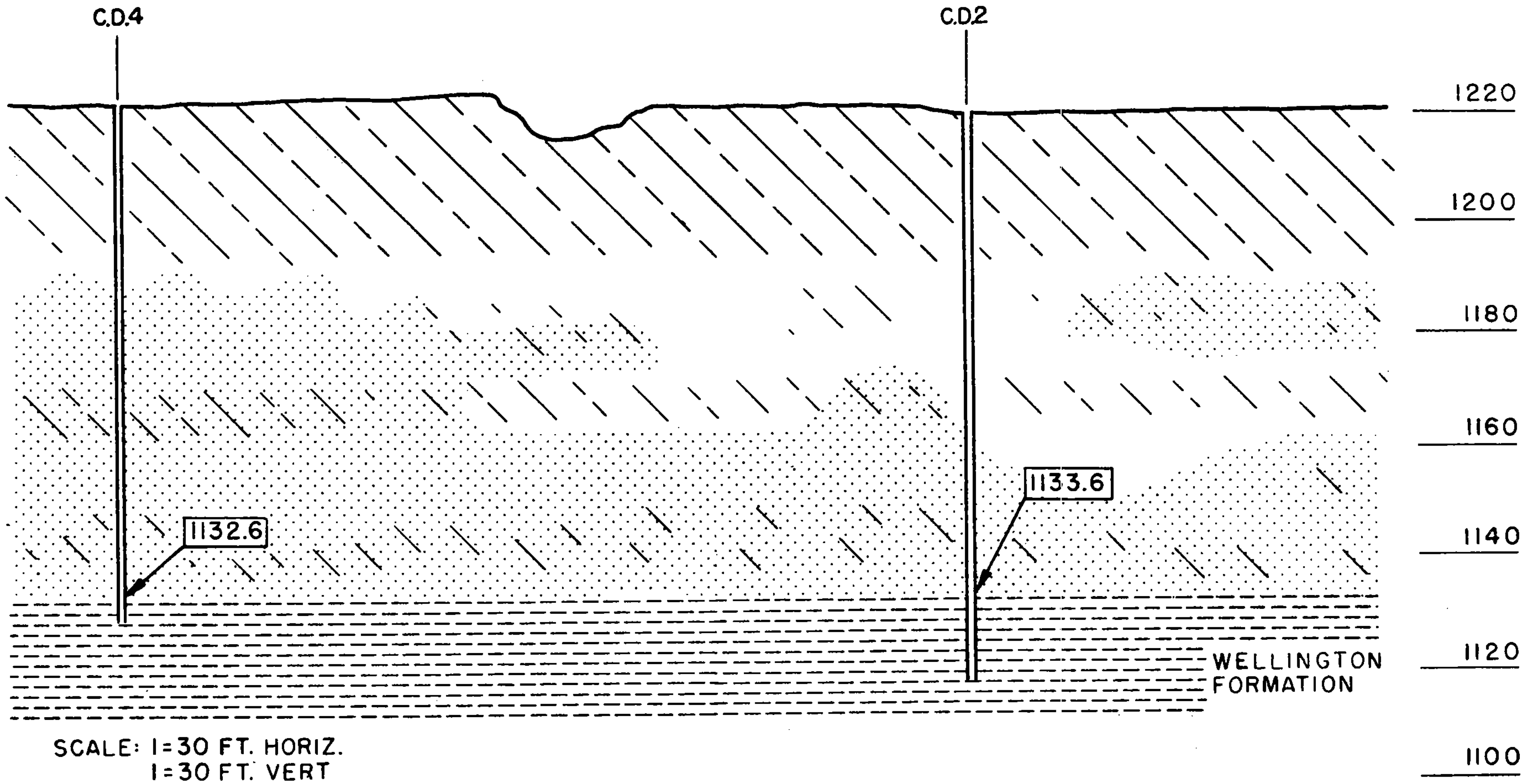

Figure 2. Determination of depth to formational contact.

An evaluation of the complete geologic situation must be made in using the penetrometer drives. The different trends of the penetrometer drives are used along with an individual drive to determine the design lengths of the pile. The changes in the density of the material as indicated by the penetrometer drive determine the sampling phase of the investigation. This use of the Penetrohammer is very important, as it can shorten the geologic-investigation time considerably.

The Penetrohammer is used to determine the pile lengths in some bedrock conditions. There are some geological situations where a bridge site could be investigated with only two core-drill soundings, but the Penetrohammer can be used for evidence of all the footings. The Penetrohammer drives are excellent indicators of the weathered depth of shales. This application of the drive is valuable on a spread-footing condition. Breakoffs in the bedrock are also located with the Penetrohammer.

The pile may obtain the necessary bearing in sediments where the penetrometer drive may indicate low resistance or may drive below the refusal point of the penetrometer drive. This may be due to misinterpretation of the geology or inefficiency of the hammer due to worn parts or moisture in the lines. The use of the Penetrohammer at temperatures below 20°F is not recommended. Drives in clay can indicate large resistant zones which the pile will penetrate with little resistance. The end results of the Penetrohammer depend upon the type of material and the experience of the personnel.

Brief studies have been made of the penetrometer drives using a longer washer and a pile formula (Worley and Taylor, 1970, and Rockers and Ubel, 1961). The results of the test did not warrant continued use of either method.

The Penetrohammer is used throughout the western two-thirds of Kansas, in sediments varying from loosely consolidated to bedrock. In the eastern section the machine is used mainly along large stream valleys. The Penetrohammer is used before, after, or during the drilling phase of the investigation--geologic conditions at the foundation site and the availability of the machine determine time of use.

CONCLUSION

The Penetrohammer is a useful tool in foundation investigation. The 50-second opinion is a good indicator of proper pile length, especially in granular material, and the Penetrohammer quickly provides good information. More research is needed, however, because the accuracy of the results depends upon the type of material being investigated and the experience of the geologist.

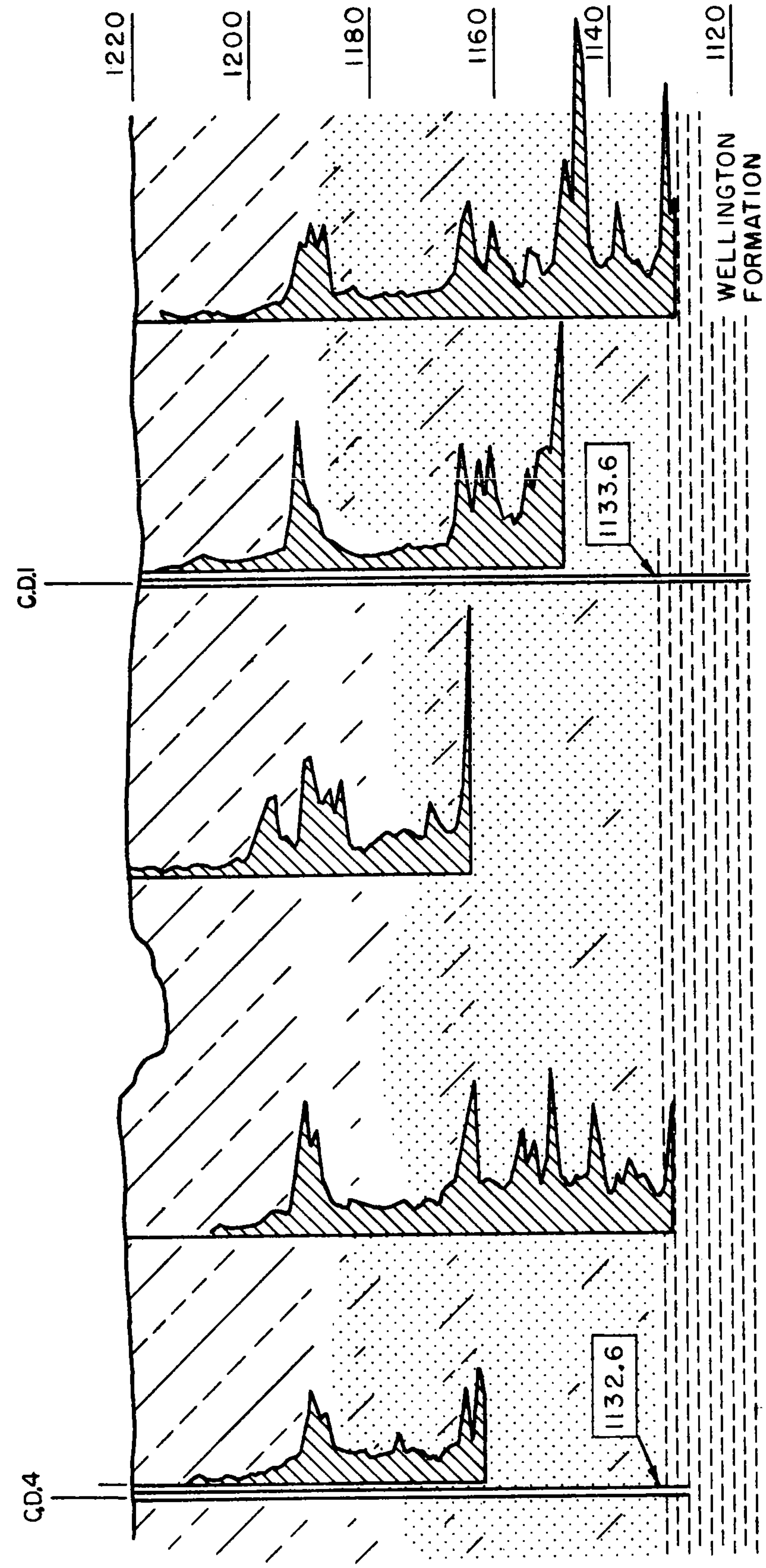

Figure 3. Five penetrometer drive tests.

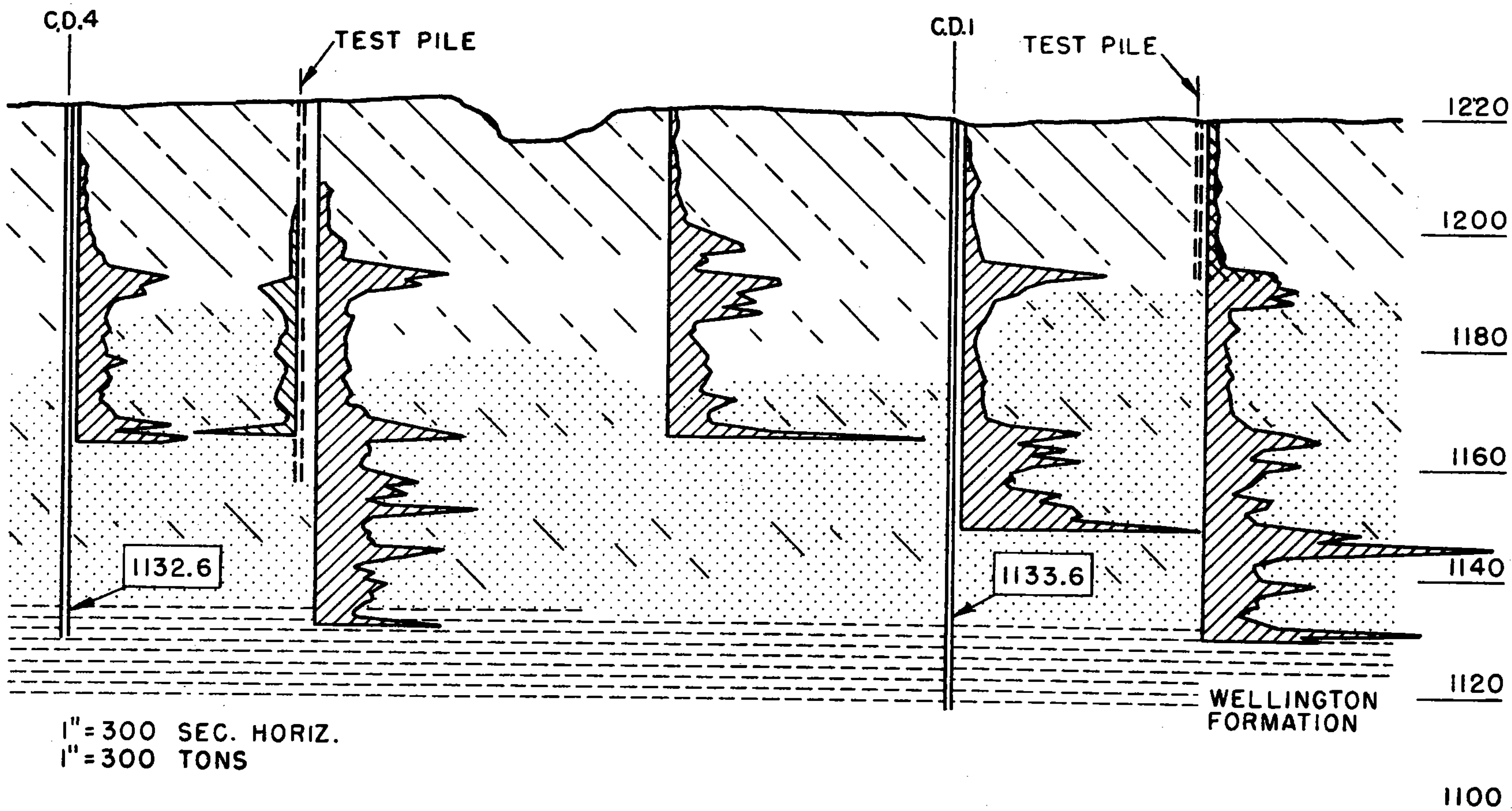

Figure 4. Computed bearing power in tons, plotted at 1-foot increments.

ACKNOWLEDGMENT

The study of the penetrometer drives was begun under the direction and inspiration of Mr. Virgil A. Burgat, Chief Geologist of the Kansas Highway Commission. The foresight of the leaders of the Geology Section in building the machine and trying something different has been an asset to the Kansas Highway Commission. Ideas for using penetrometer drives have come through the hard work and dedication of many employees. I wish to thank the following fellow employees for their suggestions and review of this report: D. G. Brison, L. W. Fowler, G. R. Koontz, A. L. Milner, W. H. Miller, L. A. Rockers, W. K. Taylor, and R. P. Worley. Appreciation is also extended to the office staff for their assistance in preparing this material for presentation.

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