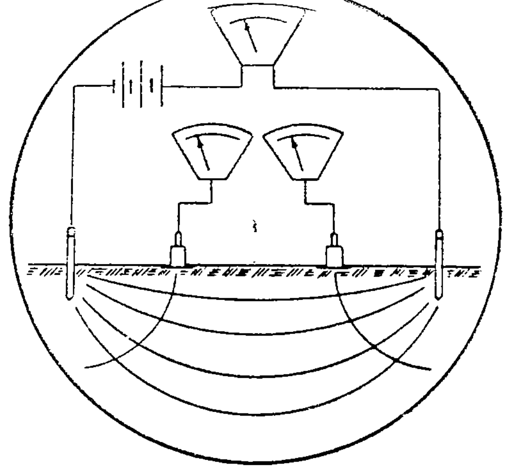

PROCEEDINGS
OF THE 12TH ANNUAL
SYMPOSIUM ON

GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING

BULLETIN NO. 24

OCTOBER, 1961

ENGINEERING EXPERIMENT STATION
THE UNIVERSITY OF TENNESSEE
KNOXVILLE

University Center, The University of Tennessee

February 10-11, 1961

PROGRAM

TWELFTH ANNUAL SYMPOSIUM ON GEOLOGY AS APPLIED TO HIGHWAY ENGINEERING

Engineering Experiment Station Bulletin Number 24

The University of Tennessee

Knoxville

PROGRAM

Friday, February 10, 1961

8:00 a.m. REGISTRATION........................Lobby of University Center

MORNING SESSION

W. T. PARROTT

Geologist, Virginia Department of Highways

Presiding

9:00 a.m. ANNOUNCEMENTS

9:10 a.m. WELCOME ....DR. HERMAN E. SPIVEY, Vice-President The University of Tennessee

9:30 a.m. *"Principal Highway Engineering Characteristics of Tennessee Formations"* . . . NORMAN C. YOUNG, and T. R. PIERCE, Geologists, Tennessee Department of Highways, Nashville

10:00 a.m. *"Evaluation of Pavement Aggregates for Non-Skid Qualities"* . . . WILLIAM A. GOODWIN, Research Engineer, Tennessee Highway Research Program, University of Tennessee, Knoxville

10:30 a.m. *"Highway Construction and Maintenance Problems in Permafrost Regions"* . . . DONALD R. NICHOLS and LYNN A. YEHLE, Geologists, Military Geology Branch, U. S. Geological Survey, Washington, D. C.

11:00 a.m. *"Highway Salvage Archaelogy"* . . . CHARLES MCNUTT, Assistant Professor of Anthropology, University of Tennessee, Knoxville

11:30 a.m. LUNCH

AFTERNOON SESSION

HUGH D. CHASE

Soils Engineer, Georgia State Highway Department

Presiding

1:15 p.m. *"Use of the Continuous Seismic Profiler (Sparker) in Geologic Investigations for Vehicular Tunnel and Bridge Crossings"* . . . CHARLES B. OFFICER, *President,* Marine Geophysical Services Corporation, Houston, Texas

1:45 p.m. *"The Madison River, Montana, Earthquake of 1959 and the Highway Problem"* . . . REED W. BAILEY, Director, Inter-mountain Forest and Range Experiment Station, U. S. Forest Service, Ogden, Utah

2:15 p.m. *"Geological Conditions Complicating Highway and Railroad Relocations in the Northwest"* . . . W. HAROLD STUART, Chief of Geology Soils and Materials Branch, U. S. Army Engineer Division, North Pacific, Portland, Oregon

2:45 p.m. *"Observations on Subsurface Explorations Using Direct Procedures and Geophysical Techniques"* . . . R. WOODWARD MOORE, Head, Geophysical Exploration Group, Division of Physical Research, U. S. Bureau of Public Roads, Washington, D. C.

3:15 p.m. INTERMISSION

3:30 p.m. *"The Economics of Natural Resource Valuation"* . . . C. W. DORMAN, American Appraisal Company, Milwaukee, Wisconsin

4:00 p.m. *"The Application of Engineering Geology and Soil Mechanics in the Design and Construction of Highways in the Great Lakes Region"* . . . FRANCISCO J. CORDOVA, Chief Engineer, Francisco J. Cordova Engineers and Geologists, Gary, Indiana

4:30 p.m. *"Altering Physico-Chemical Characteristics of Clay-Bearing Soils with Lime"* . . . KENNETH A. GUTSCHICK, Manager of Technical Services, National Lime Association, Washington, D. C.

5:00 p.m. ANNOUNCEMENTS, ADJOURNMENT

Saturday, February 11, 1961

9:00 to 11:30 a.m. The Tennessee Highway Research Program invites you to visit their laboratories and view the research work they are doing in conjunction with the Tennessee Department of Highways. The Research Program will have demonstrations of laboratory skid testing apparatus, dynamic testing of materials, and other interesting projects.

SPECIAL EVENTS

Registrants will be guests at an informal gathering in the Oak Room of the Farragut Hotel, Thursday evening, February 9, from 6:30 to 8:00 p.m.

NATIONAL STEERING COMMITTEE

Chairman: W. T. PARROTT, Virginia Department of Highways, Richmond, Va.

Vice-Chairman: W. F. TANNER, Florida State University, Tallahassee, Fla.

Secretary: H. D. CHASE, Georgia State Highway Department, Atlanta, Ga.

PAUL H. PRICE, West Virginia Geological and Economic Survey, Morgantown, W. Va.

R. E. NESBITT, Corps of Engineers, Washington, D. C.

A. C. DODSON, North Carolina Highway Department, Raleigh, N. C.

IAN CAMPBELL, State Division of Mines,
San Francisco, Calif.

C. W. UPHAM, Washington, D. C.

A. F. AGNEW, South Dakota State Geological Survey,
Vermillion, S. D.

R. A. LAURENCE, U. S. Geological Survey,
Knoxville, Tenn.

H. A. RADSIKOWSKI, U. S. Bureau of Public Roads,
Washington, D. C.

LOCAL COMMITTEE

Chairman: R. A. LAURENCE, U. S. Geological Survey

SAM L. BREEDEN, Tennessee Department of Highways

JOHN M. KELLBERG, Tennessee Valley Authority

STUART W. MAHER, Tennessee Division of Geology

E. CARL SHREVE, Department of Civil Engineering,
University of Tennessee

GEORGE D. SWINGLE, Department of Geology,
University of Tennessee

FOREWORD

The 12th Highway Geology Symposium was held at the University Center in Knoxville on February 10, 1961. The Symposium is an annual meeting, held in a different locality each year, and Tennessee is the ninth state in which it has been held. Eleven papers were presented, and ten of these are published in this bulletin.

Registered attendance at the Symposium was 167. Registrants were classified as follows:

State Highway Departments		41
Federal Government		28
U. S. Geological Survey	12	
TVA	7	
U. S. Bureau of Mines	4	
Corps of Engineers	3	
Others	2	
State Governments		15
Faculty of Universities		26
Consultants		11
Contractors		7
Producers		20
Students*		11
Others (Unclassified)		8

*Includes only those students who registered.

Attendance by states is classified as follows:

Tennessee	86	Ohio	6
Indiana	11	Georgia	5
North Carolina	9	Illinois	4
District of Columbia	7	Alabama	3
Virginia	7	Mississippi	3
Kentucky	6	West Virginia	3

Florida, Missouri, South Carolina, Texas—2 each
Arizona, Louisiana, Maryland, Minnesota, Oregon, Pennsylvania, Puerto Rico, South Dakota, Utah—1 each

The 1962 Symposium will be held at Phoenix, Arizona on March 16, under the joint sponsorship of the Arizona State Highway Department and Arizona State University.

Robert A. Laurence
May 26, 1961

CONTENTS

PRINCIPAL HIGHWAY ENGINEERING CHARACTERISTICS OF SOME TENNESSEE FORMATIONS

NORMAN C. YOUNG
and
T. R. PIERCE

Geologists, Tennessee Department of Highways
Nashville, Tennessee

Formerly highways were designed as much as possible to avoid natural obstacles. However, in recent years rock cuts of more than 100 feet and the tremendous weight of 4-lane fills up to 90 feet and even higher are commonplace.

Construction of these super highways has uncovered some adverse rock characteristics of Tennessee formations that were not a problem with the older roads. This paper deals with the principal highway engineering characteristics of those formations that have been, or are expected to be, especially troublesome during the construction of the Primary and Interstate Systems of the state highways.

The first part of this paper will deal with the highway construction problems of the portion of the state lying west of the Valley and Ridge province of East Tennessee.

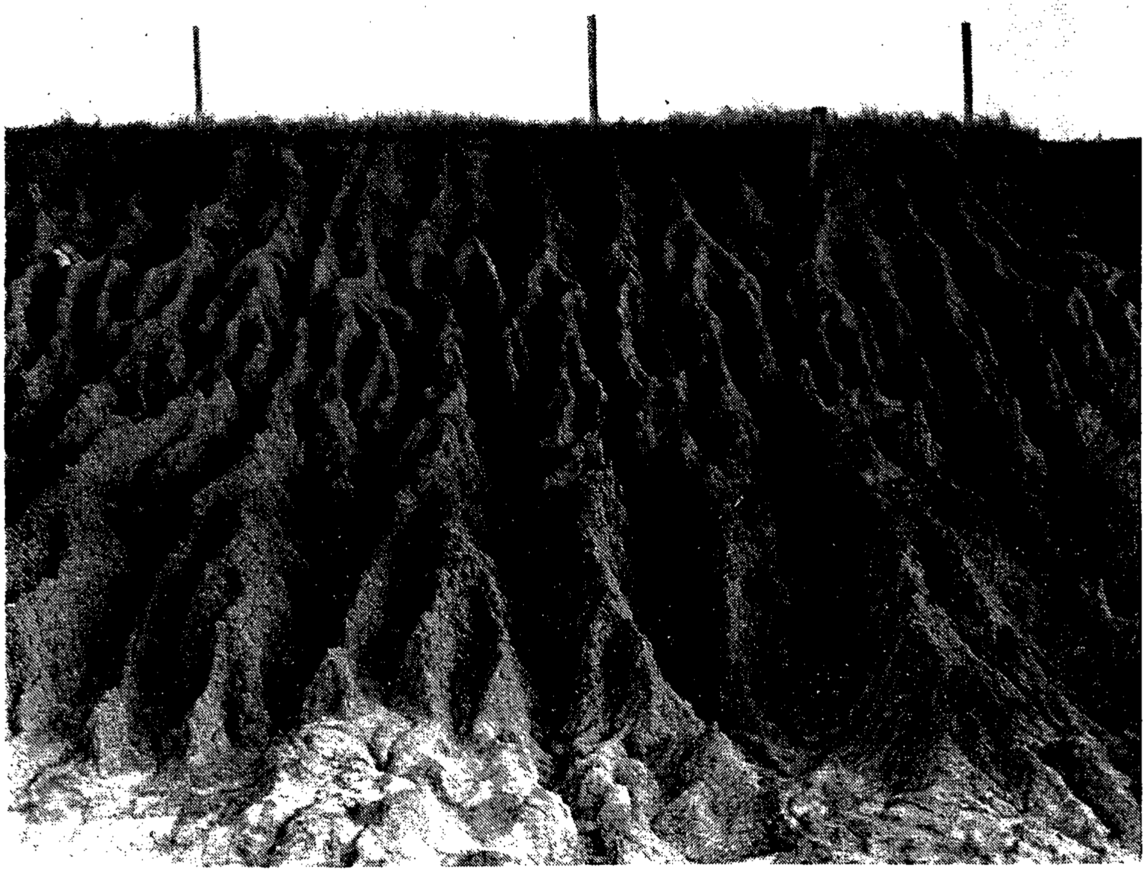

Figure 1. Loess on a 2:1 slope that was not properly sodded.

Beginning at the top of the geologic column, the first formation for discussion is the Pleistocene loess in the Mississippi embayment area of West Tennessee. This material is ordinarily stable on an almost vertical cut slope with the top of the cut protected by a good plant cover. If the usual 2:1 soil slope ratio is used, a sod must be established at once or rainfall will gully the cut slopes and block the ditches. The near-vertical slope ratio will also result in a considerable savings in the purchase of rights-of-way.

Several Eocene and Upper Cretaceous formations in West Tennessee are expected to be troublesome. These formations have similar lithologic characteristics in that they all contain horizons of very undesirable construction materials such as "pipe-clay" and highly organic material while in some instances the sand members are very micaceous. Detailed laboratory work will be necessary to determine the best course to follow with these poor construction materials.

Rocks of the Pennsylvanian system in Tennessee are shales and sandstones with numerous coal beds. Many of the sandstone formations shatter considerably during blasting operations. This characteristic is especially troublesome when a soft shale bed occurs in a sandstone cut since the shale bed often weathers rapidly, removing the support from the overlying sandstone blocks. Benching at the top of the shale zone and the use of flatter cut slope ratios aid in solving this type problem.

The Pennsylvanian shales can also cause considerable trouble when used as fill material since they disintegrate quickly when exposed to air and moisture. To prevent abnormal settlement of the fill, the shale should be broken to a size of two feet or less and choke-filled.

Figure 2. Loess on near-vertical slope with top of cuts sodded.

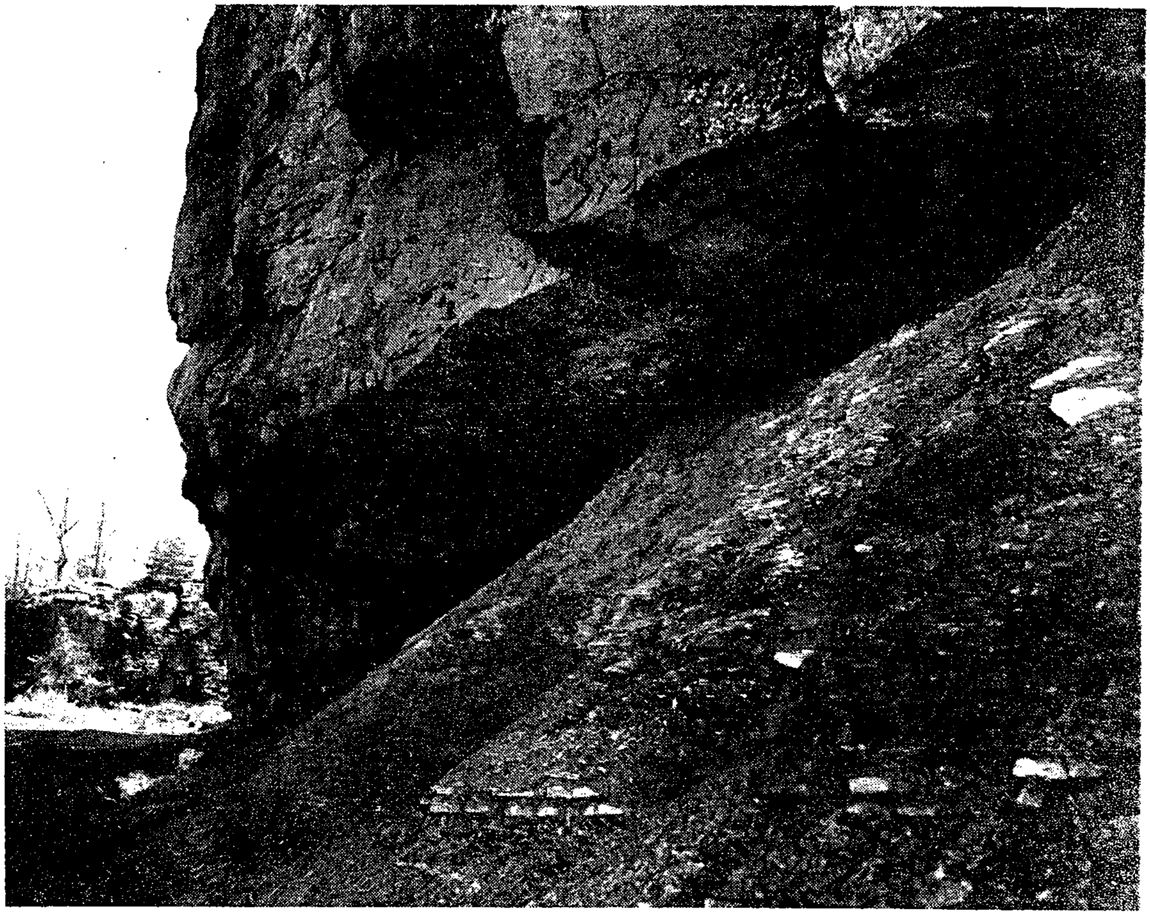

Figure 3. Outcrop along road showing how soft shales weather out, undermining the overlying beds.

Several of the Mississippian formations have undesirable highway construction characteristics. One of the worst of these formations is the Pennington. It consists of varying percentages of sandstone and clay shale with minor amounts of limestone. The clay shale beds present the same engineering problems as do the Pennsylvanian shales previously discussed.

The Golconda formation, consisting of vari-colored shale layers will also produce unstable cuts and fills unless carefully handled.

Another Mississippian formation that is often troublesome is the Ft. Payne. It consists of siliceous and calcareous shale, and sandy cherty limestone. The limestone phase of the formation contains a high percentage of chert occurring in layers up to a foot in thickness. In sound condition, a cut section with a 40 to 50 foot face can be expected to stand on a 1/2:1 slope ratio. However, if the calcareous matter has been leached, there remains only a series of minutely fractured chert layers separated by clay. When a high slope face of this material is exposed to heavy rains and freezing temperatures, a considerable amount of the chert will fall into the ditches and onto the roadway. Since the upper portion is the most severely weathered, a bench of 15 to 20 feet below the top of the first chert layers will catch most of the falling material.

These residual chert layers can also cause difficulty during soil sampling operations. Since it is impossible to drill through this material with a power auger the driller logs it as rock although it can easily be handled with earth moving equipment. An evaluation by a geologist can usually clarify this situation.

Often the lower portion of the Ft. Payne consists of limy shale which, although much harder than the younger shales mentioned, also disintegrates soon after excavation. Soils produced by the leaching of this limy shale often contain beds of "pipe-clay." The Ft. Payne must be handled in the same manner as the other shales when used as fill material.

Formations of the Upper and Middle Ordovician in the Central Basin may present construction difficulties under certain circumstances. One example is the Hermitage formation of Middle Ordovician age. It is composed of several interfingering facies, some of which consist of clay-shale and thin-bedded argillaceous limestone with shale partings. In high fills these rocks may disintegrate rapidly causing uneven settlement unless they are crushed and choke-filled. In steep cuts the shale material of the Hermitage weathers out leaving no support for the interbedded limestone layers which break up and fall during periods of bad weather. Flatter cut slope ratios and benching will aid in maintaining stability and reducing maintenance.

The Leipers and Catheys formations present similar problems since they both contain facies which resemble the Hermitage.

Within the lower few feet of the Hermitage, there usually occurs a bed of impure bentonite. When this zone lies near the surface, it has often been weathered to "pipe-clay" although the rock above and below may still be

Figure 4. A long-range shot of a high unbenched road cut in leached Ft. Payne, showing especially unstable chert layers at top.

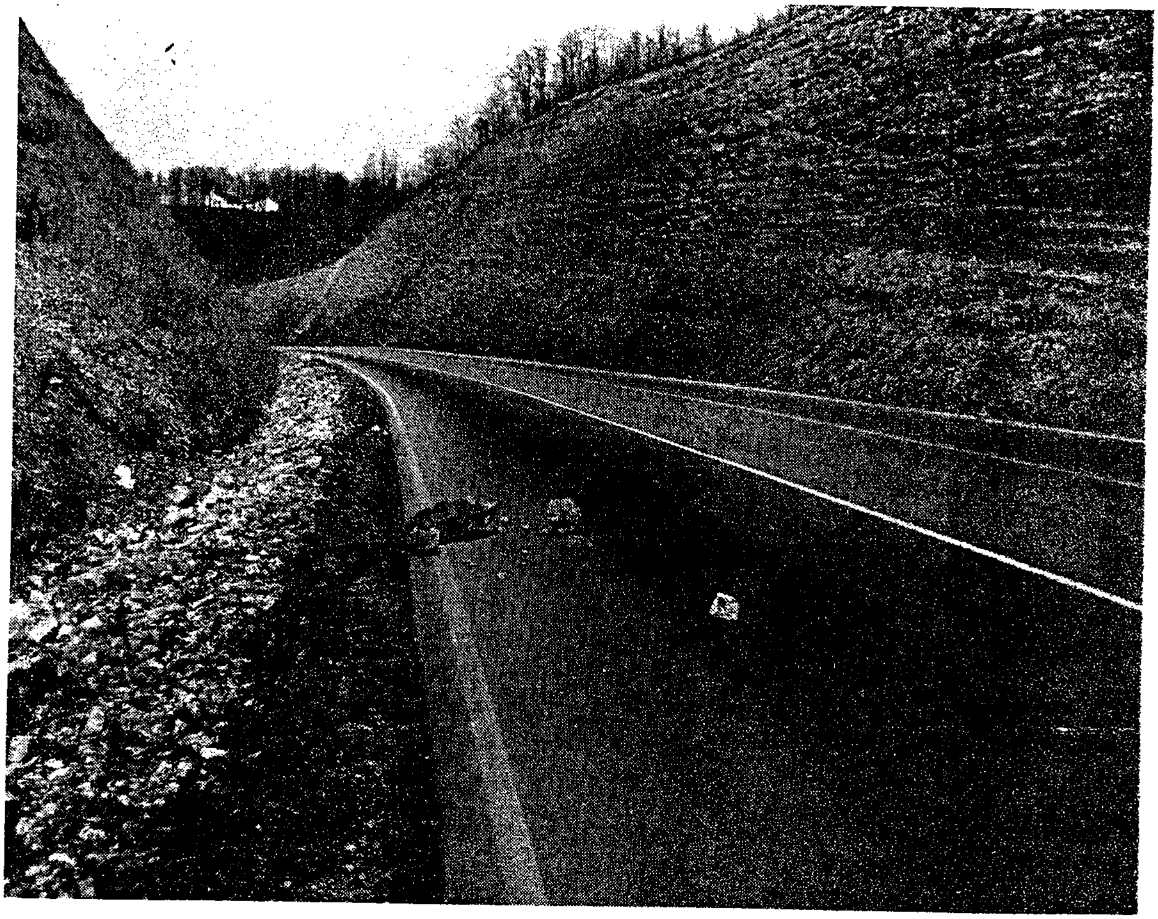

Figure 5. Close-up of chert fragments that have fallen into roadway. A bench near the top would have caught this material. This would eliminate day-to-day maintenance.

comparatively sound. Even if the bentonitic bed is solid when excavated, it will disintegrate rapidly in a fill.

To avoid disastrous construction results, this undesirable material must be recognized and wasted as it comes from cut or borrow areas.

The purer limestones of the Upper and Middle Ordovician often present problems in the Central Basin. They lie at or near the surface and are dissected by numerous near-vertical joints. Since the limestone is readily soluble, weathering has widened these joints tremendously and they are usually filled with clay. Often the situation is such that the limestone up to a depth of 30 to 40 feet has been reduced to a group of rock columns separated by clay. When a cut is made in this material, water action washes the clay from the vertical joints into the ditches. This, of course, blocks the ditches and causes the roadway to deteriorate. Also, as the clay is washed from the joints, the limestone columns become more and more unstable since excavation has already removed much of the lateral support. In high rock cuts of 25 feet or more, many falling rock fragments reach the roadway.

When large clay zones, due either to joints or sinks, are encountered in rock faces, excessive maintenance may be avoided by cutting back the soil zones to a 2:1 or flatter slope and sodding them.

Greater stability in high cuts in these Middle Ordovician limestones is obtained when the upper portion of the cut is put on a 1/2:1 or flatter slope ratio and benched at appropriate intervals.

Considerably less Interstate work has been started in the Valley and Ridge province of East Tennessee and the problems are just beginning. There Pennsylvanian and older formations are encountered. Because the rocks have been greatly folded and fractured, it is reasonable to expect difficulty from any formation in a deep cut, although thin-bedded and shaly facies of the formations have especially poor engineering characteristics. These facies have been fractured into small pieces and weather in cuts and fills even faster than they would ordinarily.

Under normal circumstances, proper slope ratios and benching at appropriate intervals can be expected to maintain cut slope stability although ravelling will occur. However, some sliding is almost certain when the cut is opened in instances where the formation dip is toward the roadway. This is especially true if thin-bedded or shaly strata are involved. Once the lateral support is removed these highly fractured beds begin moving down dip, opening large vertical fissures over the rock slope face. When water enters these fissures the slide is greatly accelerated.

Since several factors are involved in potential slide areas in East Tennessee, no one remedy can be presented. It is, of course, absolutely necessary to recognize the problems and, if possible, design the cut slopes properly before construction is begun.

Although many of the Tennessee formations present serious design and

Figure 6. A close-up showing recently excavated limy shale in comparison to the same material that has been exposed on a fill slope a few months.

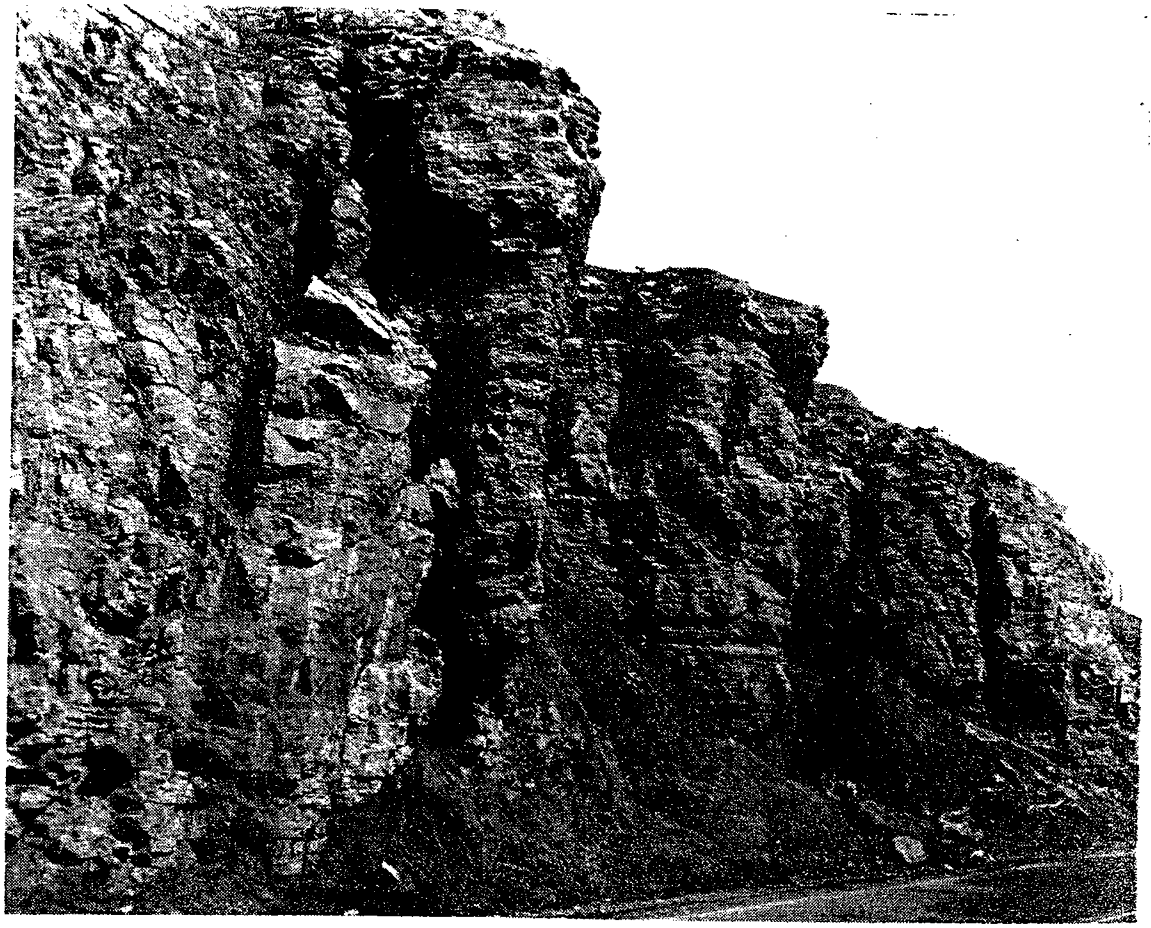

Figure 7. Road cut in Middle Ordovician limestone. Note clay washing out of enlarged joints, producing typical isolated columnar blocks of unstable rock.

construction problems the Highway Department is making every effort to cope with them. The Division of Materials and Tests employs several drill crews under the supervision of a Soils Engineer. They carry out a detailed soil sampling program on all Interstate and Primary highway projects. The proposed route is also examined by a geologist who makes a detailed report on probable rock conditions. All of these data are assembled in the form of a report and forwarded to the Design and Construction Divisions together with recommendations.

EVALUATION OF PAVEMENT AGGREGATES FOR NON-SKID QUALITIES

W. A. GOODWIN

Research Engineer. Tennessee Highway Research Program
The University of Tennessee

Introduction

The non-skid characteristics of pavement surfaces have been of interest to investigators since the advent of modern motor transportation. As early as 1924, a report by Agg[1] was published concerning "Tractive Resistance and Related Characteristics of Roadway Surfaces." Since that time various investigators[2] have published the results of their efforts in studying the phenomenon of pavement slipperiness. As a result of these studies it has been clearly demonstrated that slipperiness is related to many factors. These factors may be broadly grouped into those attributed to the roadway, geometric design, roadway surface composition and design, vehicle condition and operation, and tire composition and design. Of these many factors, considerable effort in recent years has been directed toward a better understanding of the skid or non-skid qualities of pavement aggregates.

For the purpose of this paper, pavement aggregates may be defined as the inert material such as an aggregation of sand, gravel, crushed stone, slag, and expanded shale with which bituminous or portland-cement cementing materials are mixed to form concrete. Characteristics of aggregates which affect skid resistance include type, size (including gradation), shape, texture, hardness, resistance to polishing, durability to weathering, and mineral composition. Many of these characteristics are interrelated to such an extent that their separate evaluation would be difficult and seemingly impossible to determine. Data presented in this paper will serve to indicate the possible influence of some of the more significant characteristics.

Aggregate Characteristics

The evaluation of aggregate quality as related to the non-skid performance of pavement surfaces may be accomplished by laboratory analysis of individual particles or by laboratory and field testing of pavement surfaces which include the aggregates in question in their surface mixes. The advantage of laboratory testing as opposed to field testing is that of pre-evaluation of an aggregate prior to its use in service.

In most instances final evaluation must await the results of field testing under actual traffic conditions. The following comments relate to laboratory and field testing of aggregates in general.

Size

The significance of aggregate size (and gradation) may be noted by referring to Figures 1 and 2. Figure 1 contains a photomicrograph of a 100 per cent limestone bituminous seal coat surface which contains a ¾-in. maximum size aggregate. The wet coefficient of friction for this surface as measured with a two-wheeled trailer (3) equipped with a smooth tread tire and travelling at 40 mph was 0.24. At the time of test, this surface had been constructed for approximately thirty-seven months, and it had an age-

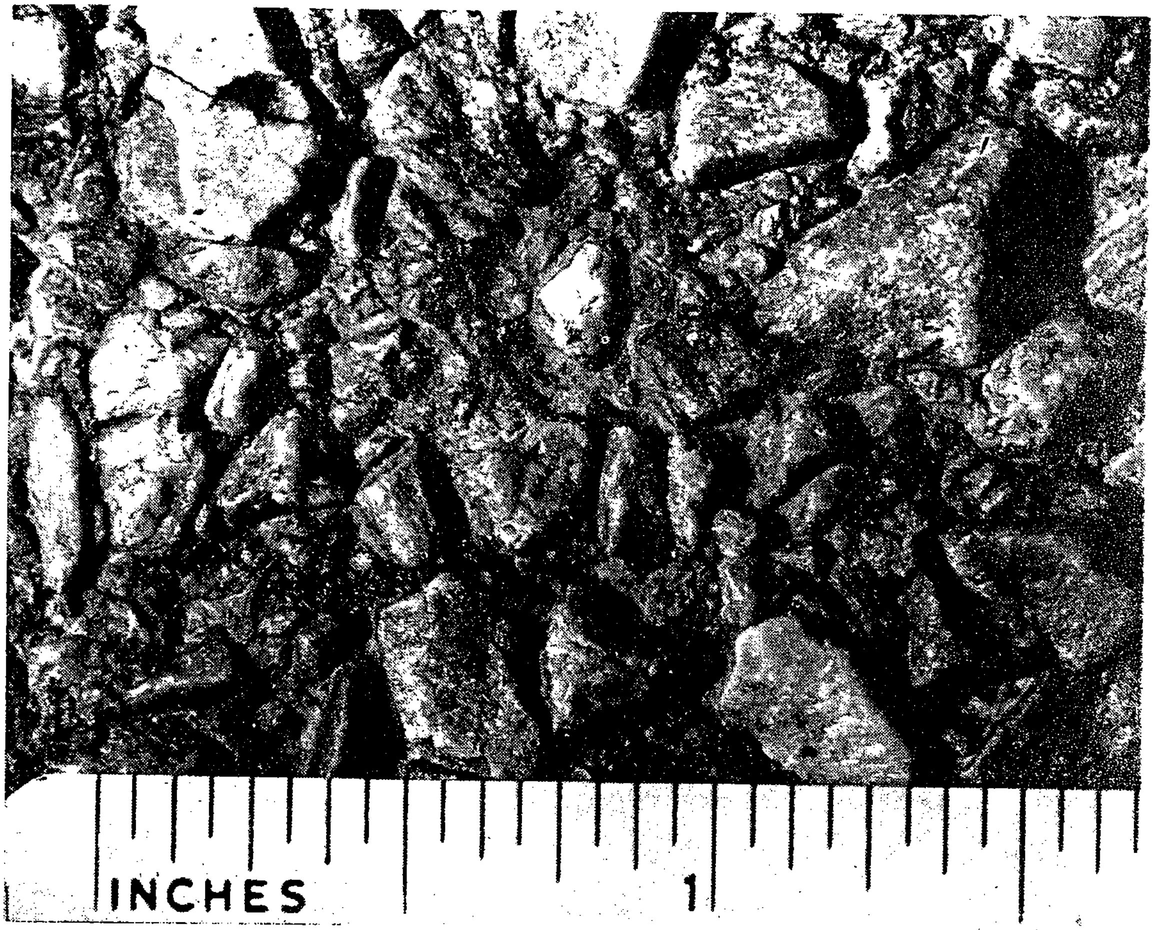

Figure 1. Photomicrograph of a 100 per cent limestone bituminous seal coat surface.

traffic index of 690.** Even though the surface aggregate was highly polished, the surface maintained a suitable skid resistance. This resistance is attributed to the "water escape channels" initially built into the roadway surface by using large aggregate and seal coat type construction. These "channels" relieved the surface water to such an extent that the tire was enveloping the peaks of relatively dry aggregate. Surfaces of this type of construction have proven suitably skid resistant, but they are quite noisy and tend to wear tires rather rapidly. Referring to Figure 2, this surface contained 92 per cent sand which was predominately of a siliceous nature and 8 per cent limestone dust. Trailer tests at 40 mph using a smooth tread tire produced a wet coefficient of friction of 0.12. At the time of test this surface also had an age-traffic index of 690. Polished siliceous aggregates may be observed by viewing the surface photomicrograph. The low skid resistance of this surface is attributed to the "tightness" of the surface and to the low void content of the surface mix. Since the surface aggregates were tightly retained in the surface, they were polished by the action of traffic and lost their initial angularity. It is also postulated that the low void content of the surface mix prevented the expulsion of surface water from under the test tire during the test. In a surface with a high void content, the water would be forced into the surface by the tire which would reduce the lubricating effect of the water. The effect of the surface void

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$$**ATI = \frac{\text{Surface Age in Months x ADT}}{100}$$ See reference 3.

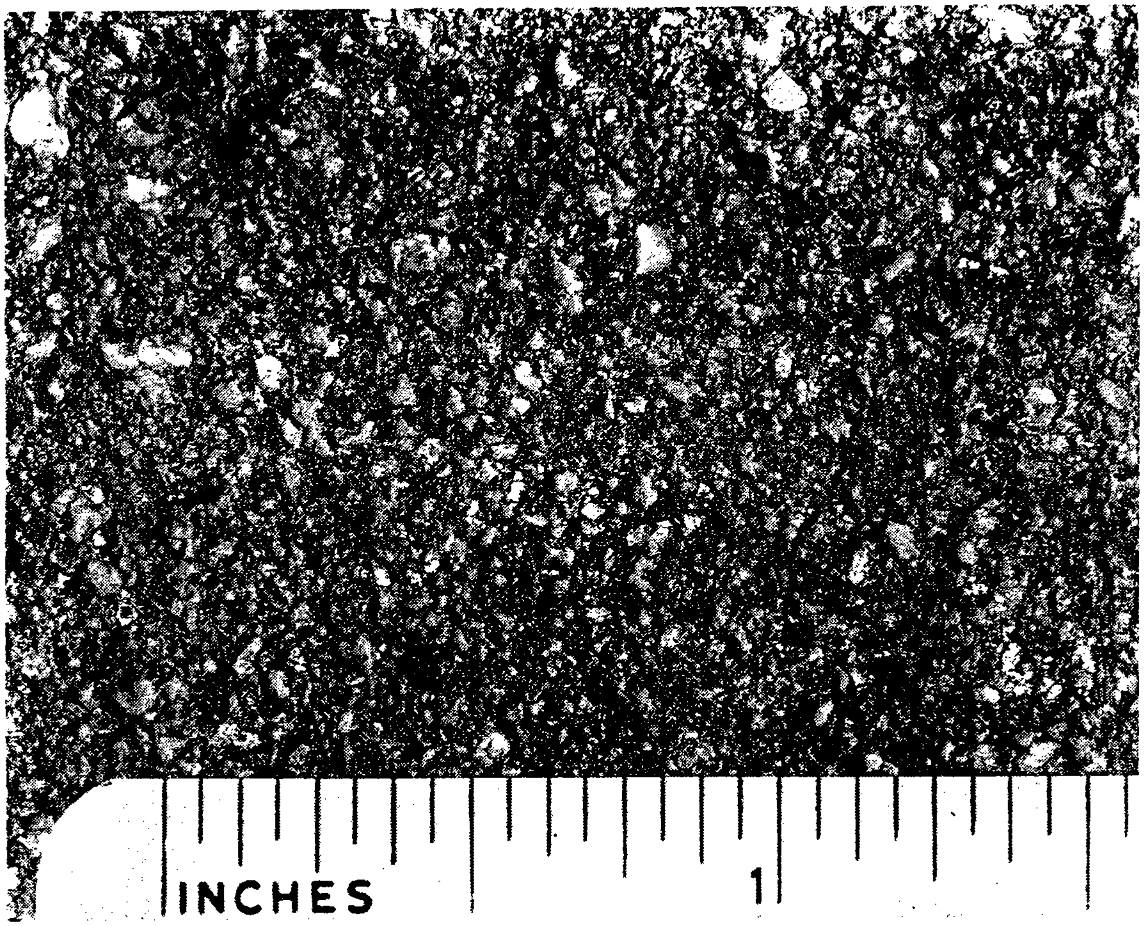

Figure 2. Photomicrograph of a 92 per cent sand and 8 per cent limestone bituminous plant mix surface.

content has been observed by Williams and Havens (4) in regard to studies on Kentucky Rock Asphalt.

Even though the above examples concerning particle size and aggregate gradation favor the large size aggregates, it is recognized that Kentucky Rock Asphalt, which is a natural impregnated sandstone with a top aggregate size of the No. 8 sieve, exhibits the highest coefficient of friction of most bituminous surfaces. This continued retention of a high coefficient of friction of Kentucky Rock Asphalt is attributed to the ability of the surface to wear and expose unpolished particles and to the surface mix's high void content of 15 to 20 per cent[4]. Data currently being reported by several investigators seem to favor fine aggregate mixtures[5] for skid resistant surfaces, and a range of particle sizes are preferred to a uniform size gradation.

Shape

The angularity of pavement aggregates initially contributes considerable skid resistance to a surface, but it is usually of a short range benefit. Laboratory data derived from an angularity test on several fine aggregates indicated that an aggregate which was known to rapidly polish under traffic exhibited the greatest initial angularity. The angularity test[6,7] used for this evaluation was essentially a flow test which measured the time in seconds required for 500 gm of the sample (20-30 sieve size) to flow through a 3/8-in. orifice. The essential components of the test apparatus are shown in Figure 3. The flow time index is the ratio of test sample flow rate to a

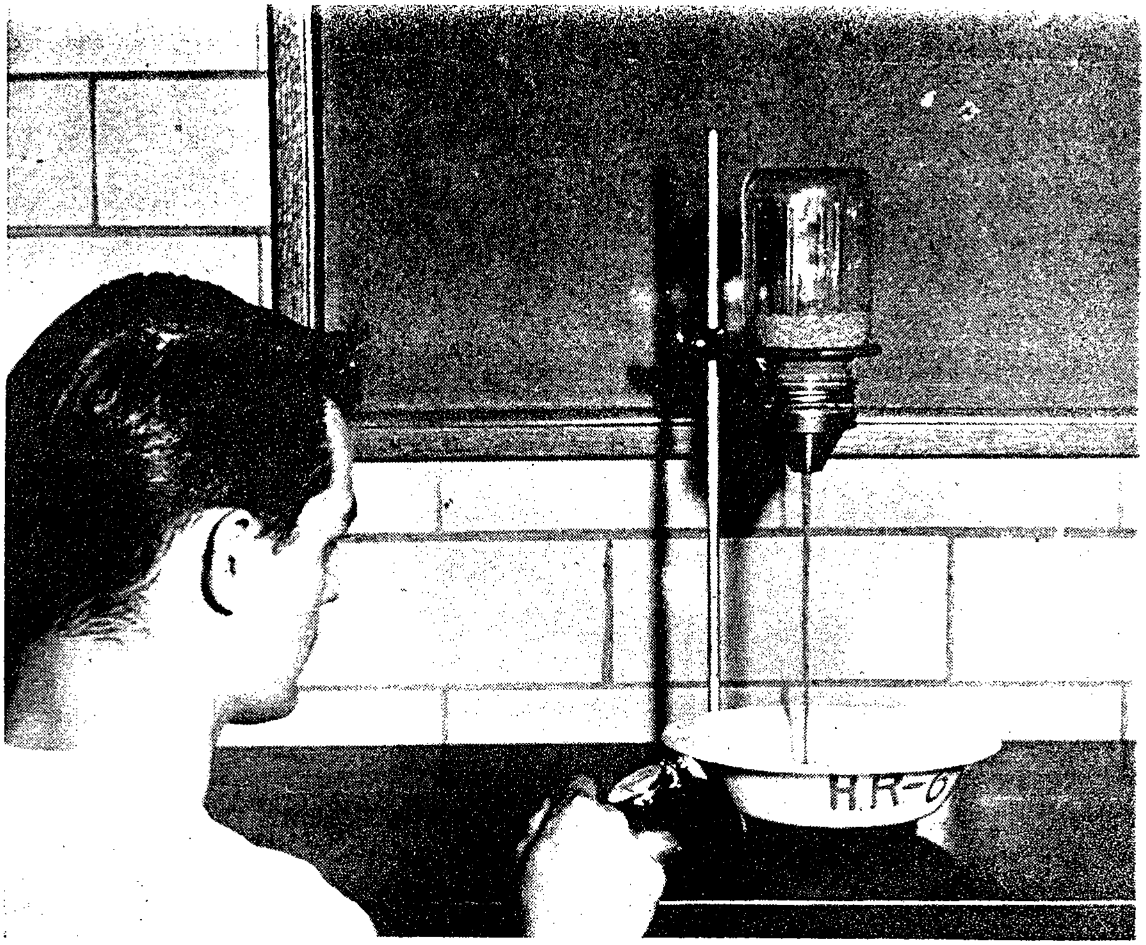

Figure 3. Angularity test apparatus.

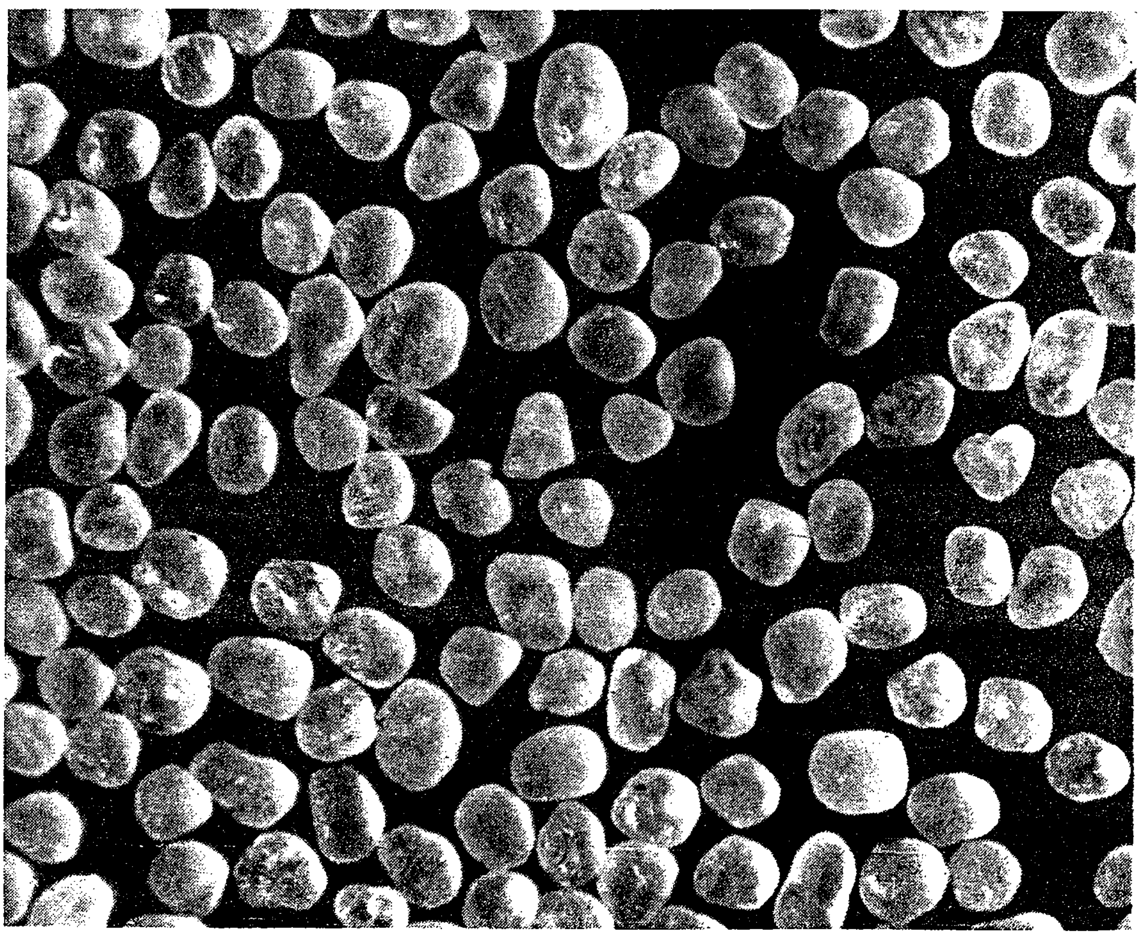

Figure 4. Ottawa sand used as standard in angularity test.

TABLE I

MEASURE OF FINE AGGREGATE ANGULARITY

Aggregate (No. 20-30 Sieve Size)		Flow Time Index*
NATURAL,	Ottawa	1.00
	Caryville	1.08
	Cumberland	1.18
	Sewanee	1.24
RIVER,	Tennessee	1.22
	French Broad	1.25
	Hiwassee	1.34
MANUFACTURED,	Limestone, A	1.35
	Limestone, B	1.46

*Ottawa as unity

standard sample, expressed in seconds per 100 cu. cm. The standard used in this study was a rounded Ottawa sand as shown in Figure 4. Table I contains a summary of flow time indices for several fine aggregates. Indices above unity indicate that the aggregate is more angular than Ottawa sand. Photomicrographs for a Sewanee mountain sand, a Hiwassee River Sand, and a crushed limestone sand are shown in Figures 5, 6, and 7. Flow indices for these sands are 1.24, 1.34, and 1.46, respectively. These data indicate that the manufactured limestone sand (Sample B) has the highest angularity and should, on the basis of angularity, have the best non-skid quality. Laboratory skid test data reported on Figure 8 indicate the resistance to sliding of this fine aggregate as influenced by its angularity;

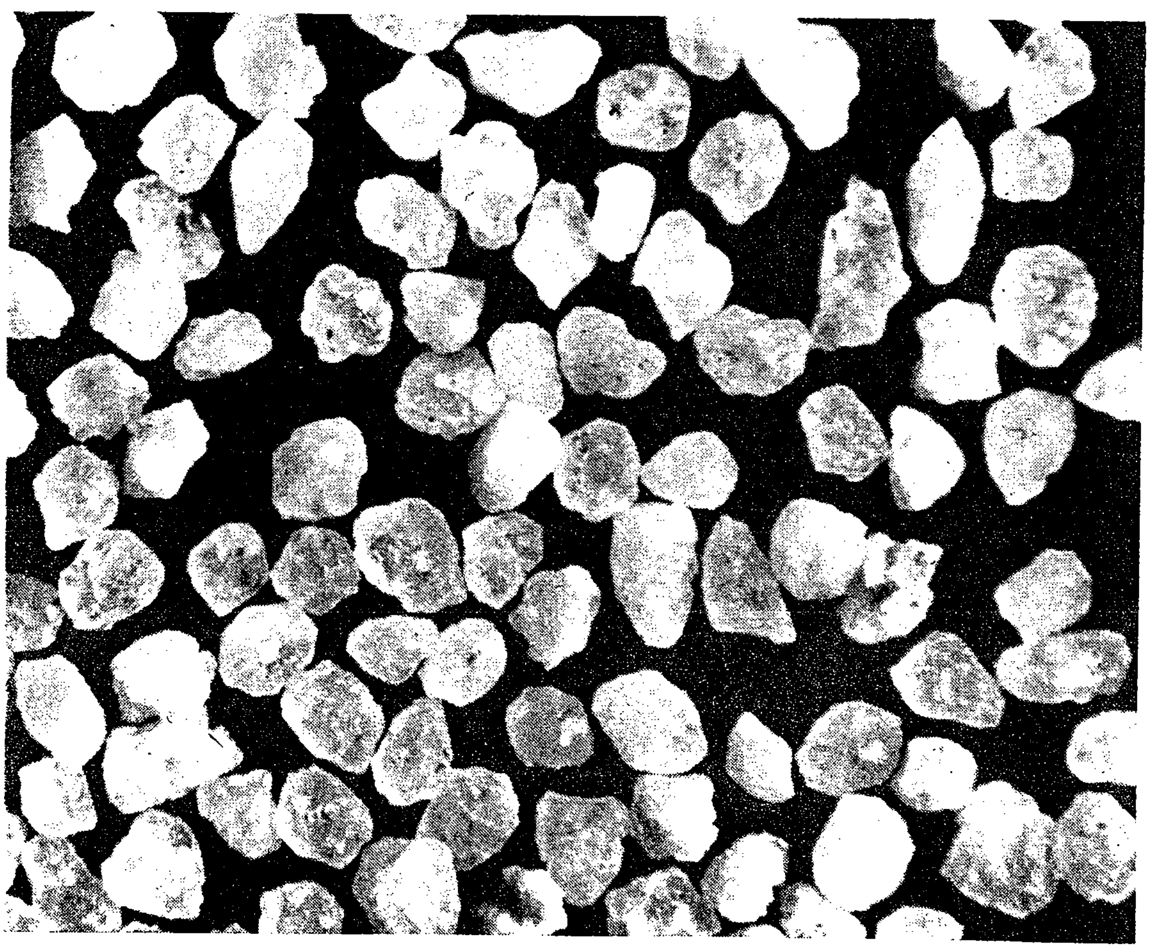

Figure 5. Photomicrograph of Sewanee mountain sand.

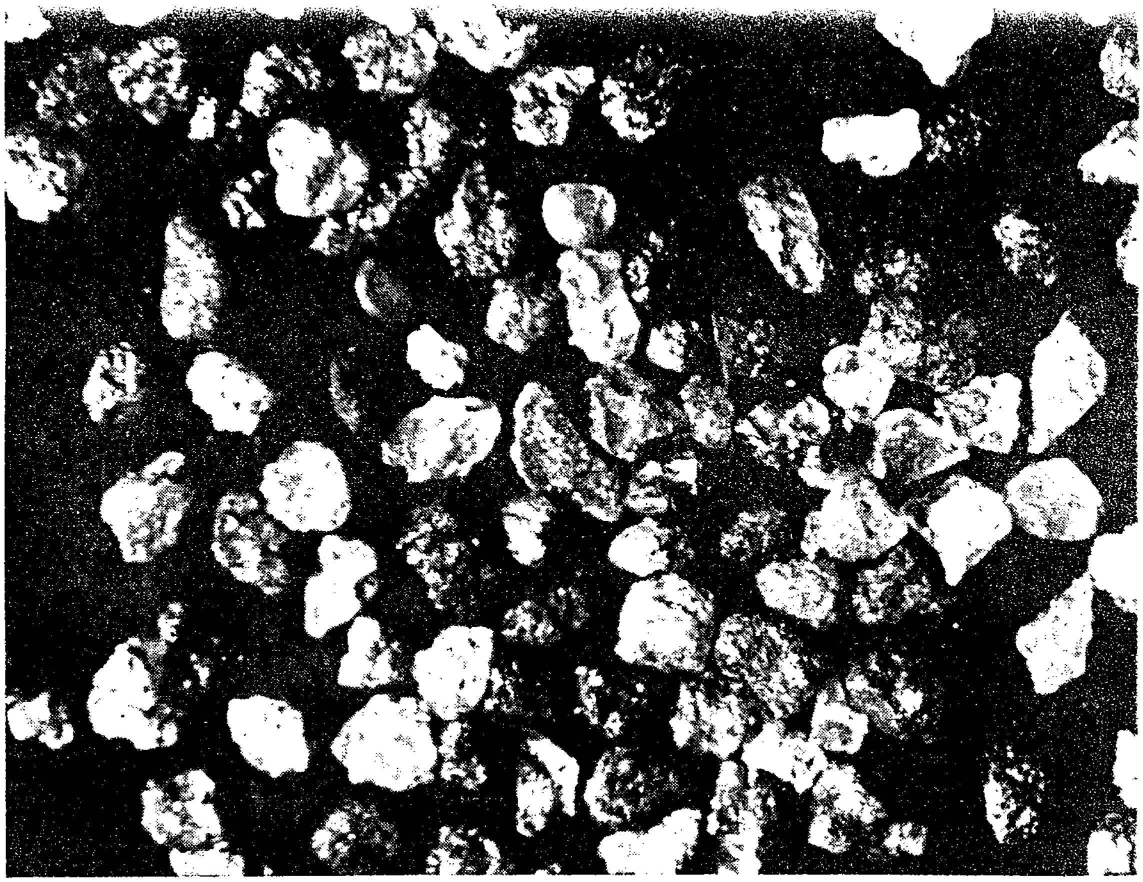

Figure 6. Photomicrograph of Hiwassee river sand.

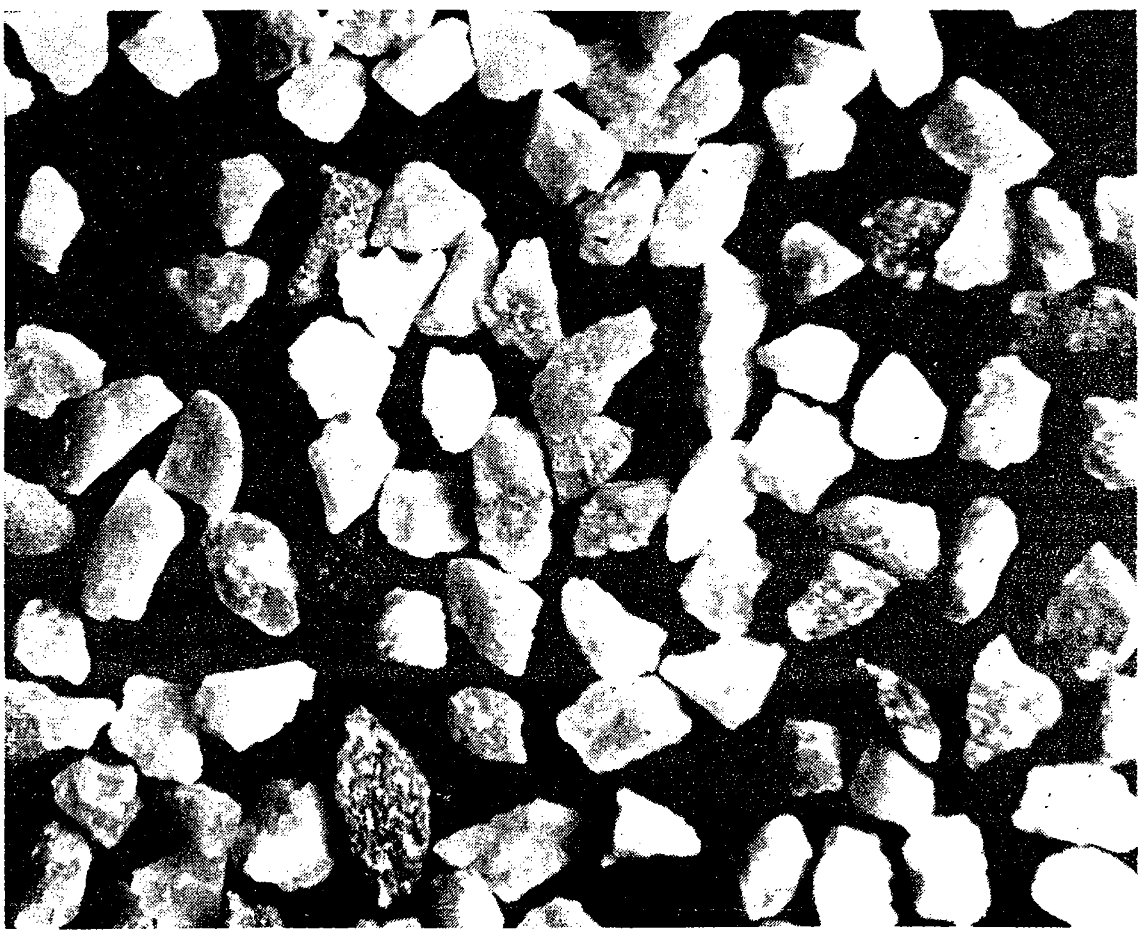

Figure 7. Photomicrograph of a crushed limestone sand (Sample B).

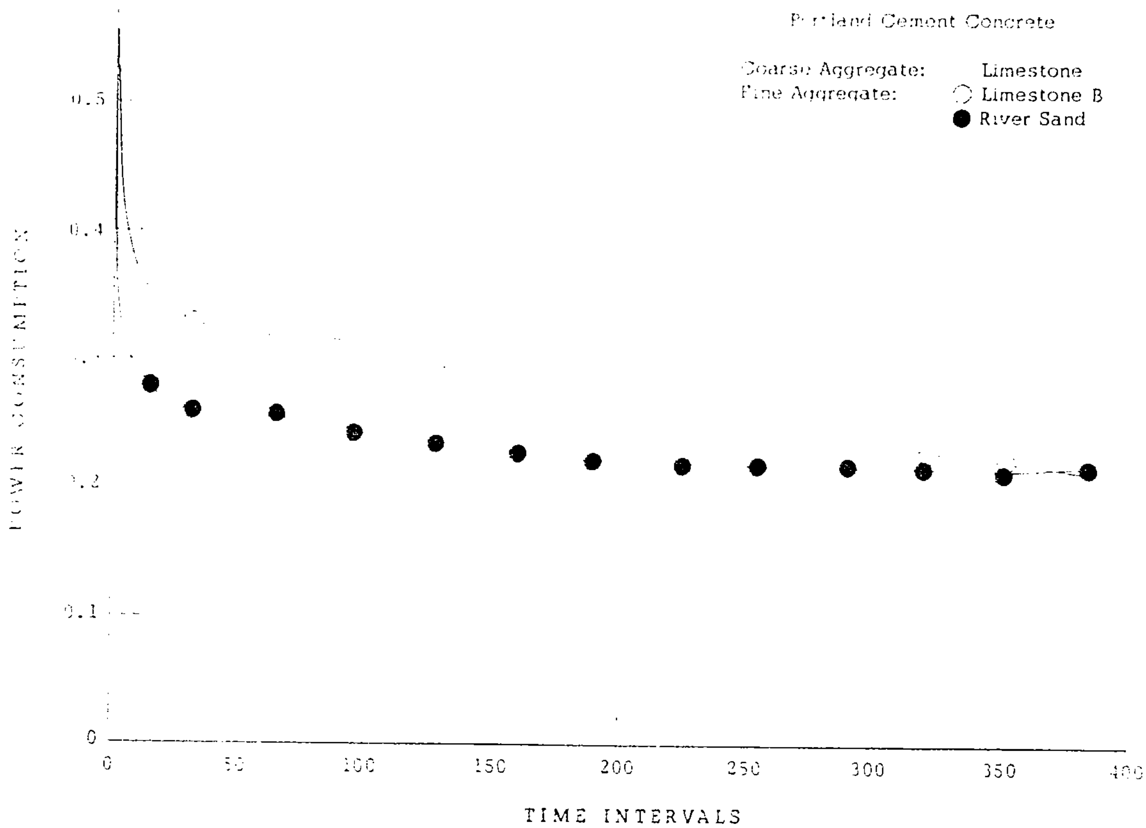

Figure 8. Effect of an angular limestone fine aggregate and a somewhat less angular river sand on slipperiness.

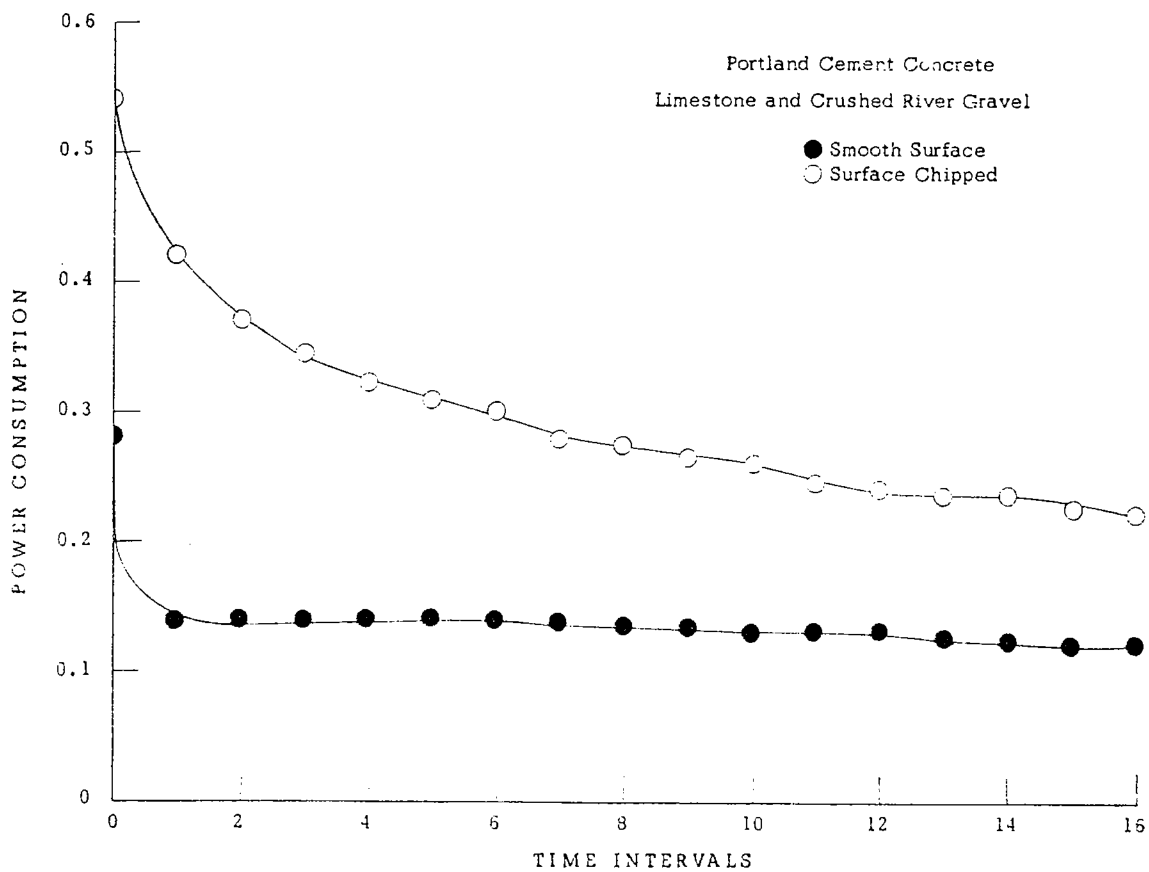

Figure 9. Effect of surface roughness on slipperiness.

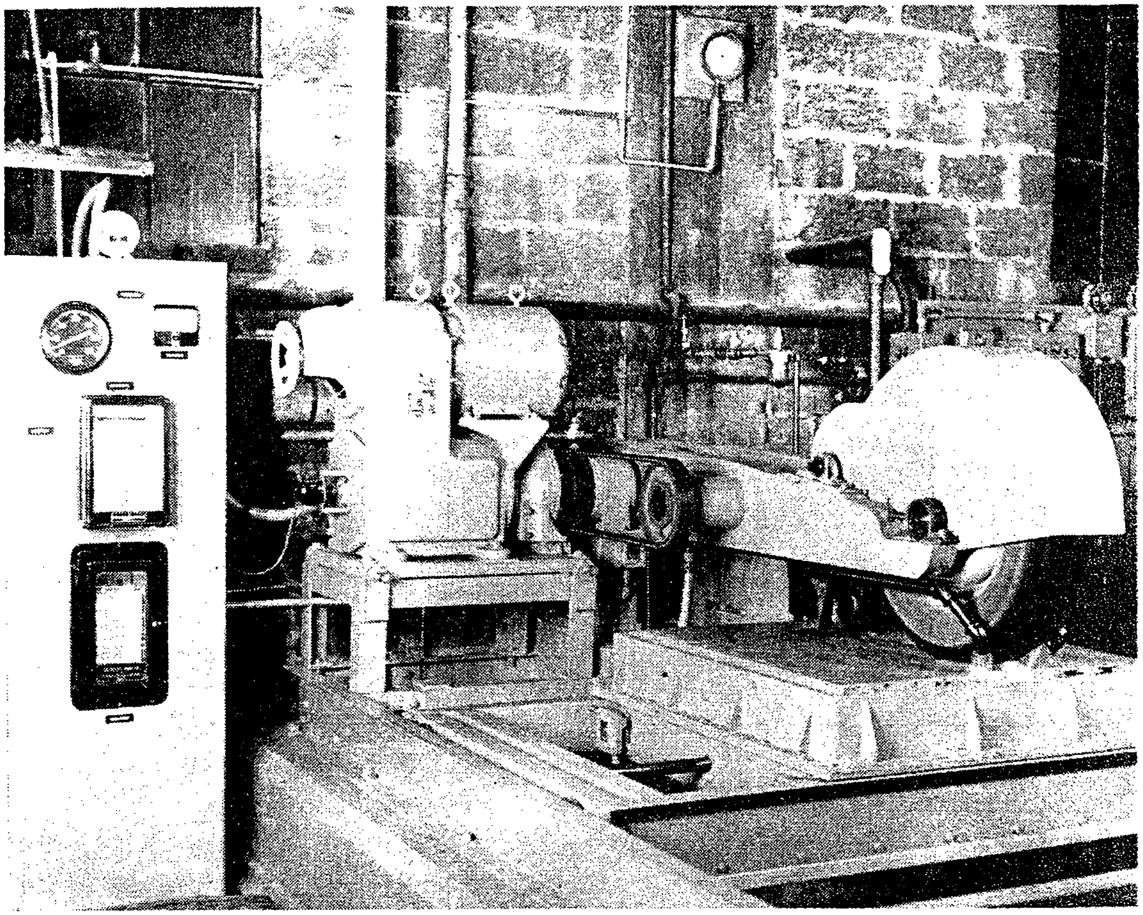

Figure 10. Laboratory skid test machine.

however, after about 24 hours of testing, the initial angularity was worn off and the aggregate became less resistant to sliding and approached that of a less angular river sand.

Figure 9 contains laboratory skid test data on an initially smooth portland-cement concrete surface along with results on the same surface after it had been chipped with a chisel. The temporary nature of the roughened (angular) surface may also be observed from these data. The resistance of the chipped surface rapidly approached that of the original smooth surface.

The laboratory skid test machine used for studying the affect of aggregate angularity is shown in Figure 10. The operation of this machine has been reported elsewhere.[5] Essentially this machine consists of monitoring the power consumption of a motor as it drives a standard automobile tire against the test specimen surface. As the specimen becomes polished, the power consumption decreases. Angularity data developed with the laboratory skid test machine are also influenced by the hardness of the aggregate, texture, ability to polish, and similar characteristics.

Durability to Weathering

For many years investigators have reported that whenever a surface wears differentially, the surface will be more skid resistant than one which does not wear but polishes. Data developed by the writer supports this conclusion and, in addition, indicate that a surface consisting of a poor quality stone, or a stone that will weather and fracture, will also maintain a high degree of skid resistance. Figure 11 contains a photomicrograph of

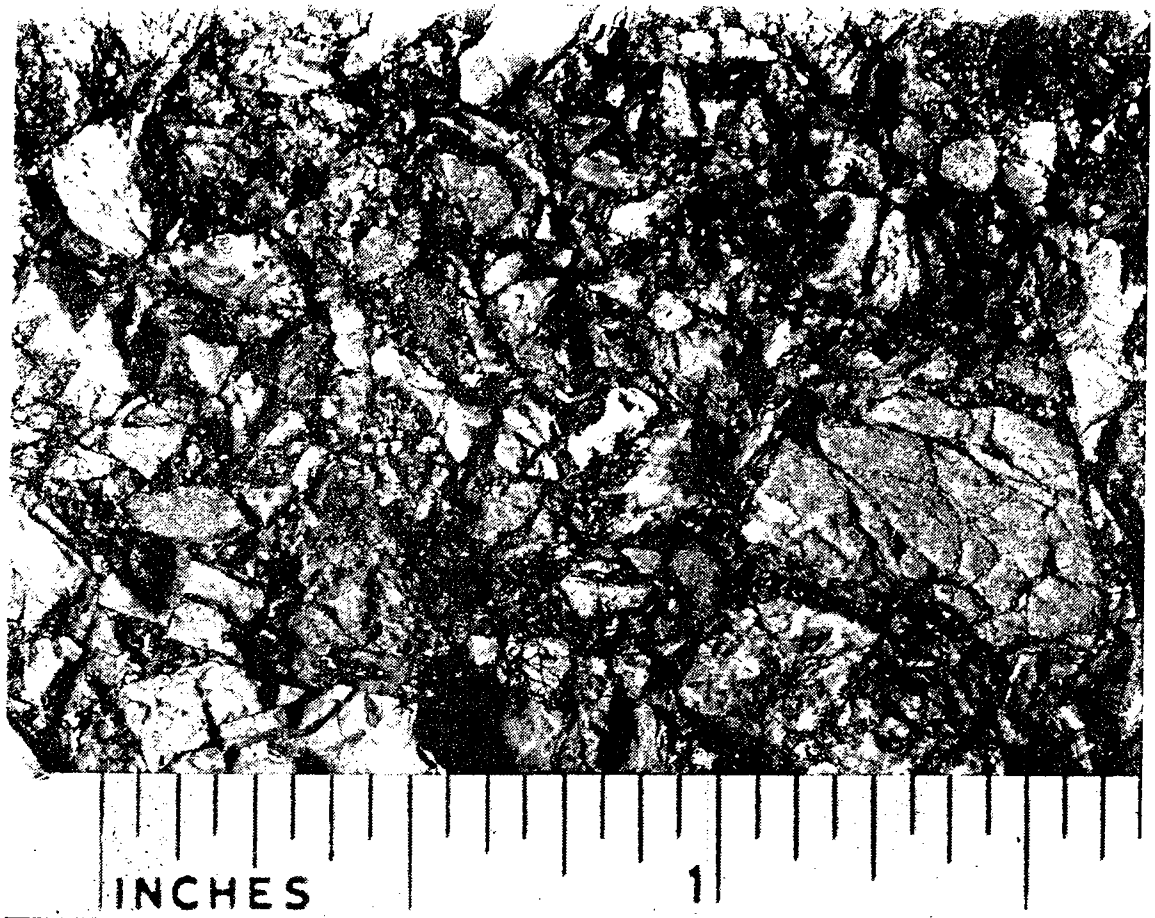

Figure 11. Photomicrograph of a fractured (weathered) limestone aggregate bituminous surface.

such a surface. The aggregate represented in the Figure has extensively fractured under rather moderate weathering conditions. The coefficient of friction measured with the Program skid trailer remains at a moderate level for this all-limestone aggregate surface. Weathering of the surface aggregate continually exposes new unpolished aggregate surfaces. These unpolished surfaces contribute to the non-skid quality of this aggregate.

Mineral Composition

The mineral composition of aggregates influence their non-skid qualities. Test results reported by Stephens and Goetz[5] indicate that the carbonates give low relative resistance values and that silicates give high values. Studies of several limestones as reported by Gray and Renniger[9] also indicate the influence of mineral composition. These studies were directed toward the determination of the amount of insoluble residue and its grain size. Data obtained on five limestones are shown in Figure 12. These data show the slipperiness (as measured by the NCSA laboratory skid tester) versus per cent of sand size particles (0.05 to 20 mm.) contained in the insoluble residue for the five limestone aggregates. The lower values of slipperiness, in degrees, indicate the non-skid quality of the aggregate. This information rather clearly demonstrates the influence of sand size particles in an aggregate on slipperiness.

Laboratory skid test data reported on Figure 13 indicate that limestones

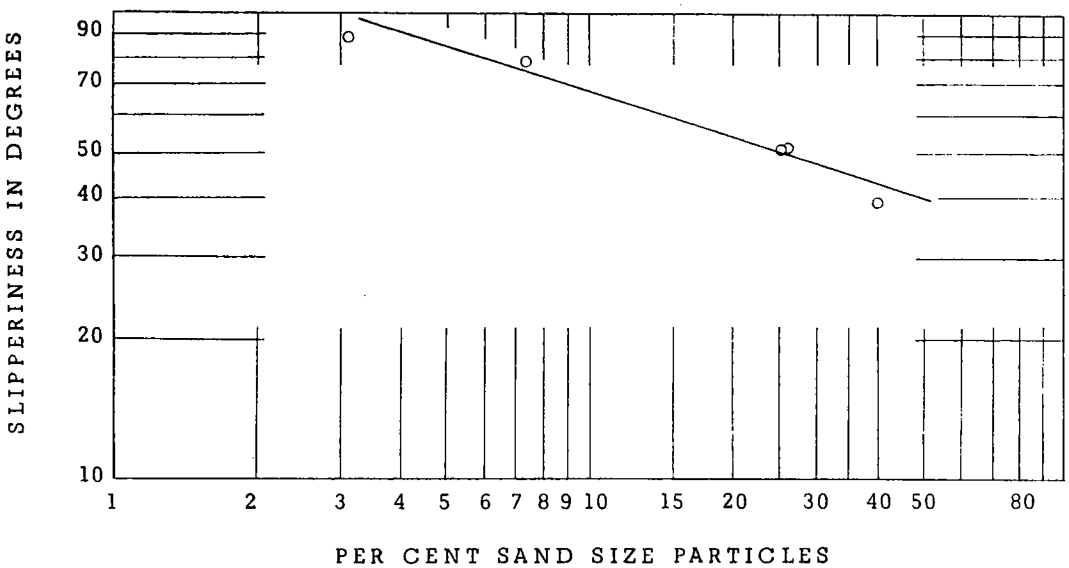

Figure 12. Relationship between slipperiness and amount of insoluble sand size particles (per Gray and Renniger, ref. 10).

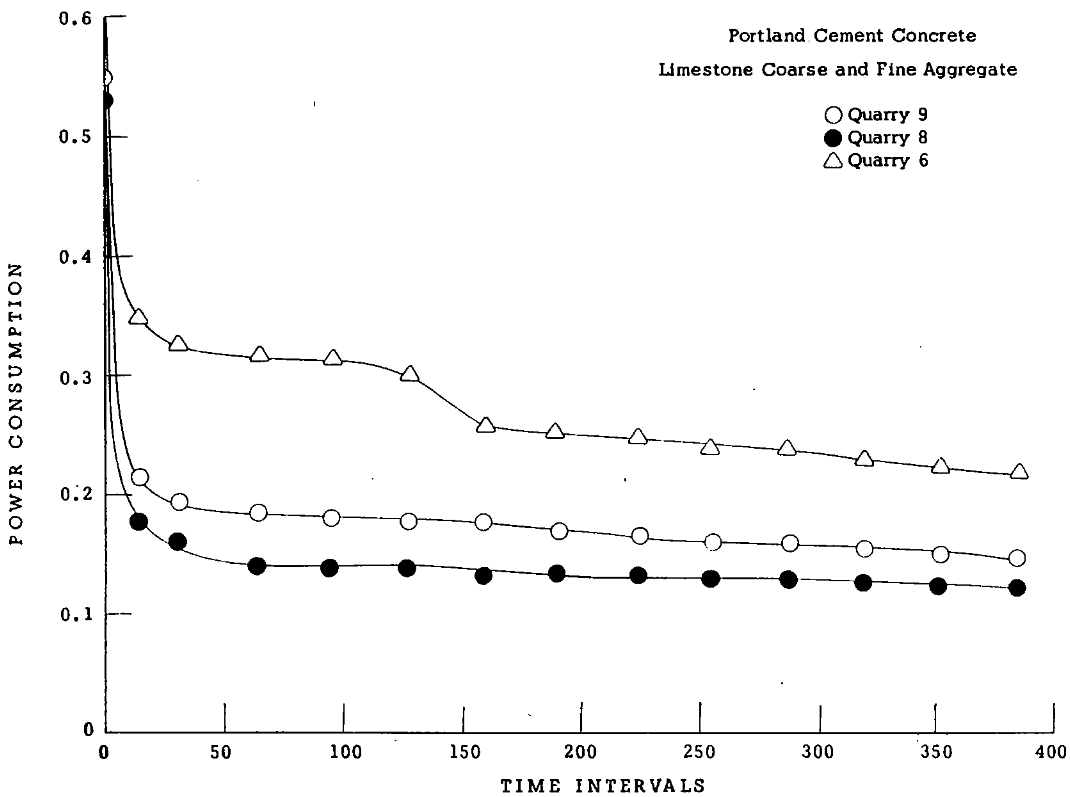

Figure 13. Relative skid resistance of three different limestones as measured with the laboratory skid test machine.

do not necessarily exhibit similar non-skid qualities. These data show that the limestone from Quarry 6 is superior to that obtained from Quarries 8 and 9. Further, it indicates that the limestone from Quarry 9 is slightly better than that obtained from Quarry 8. These limestones are being further studied to determine the cause of these differences; however, preliminary

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HIGHWAY CONSTRUCTION AND MAINTENANCE PROBLEMS IN PERMAFROST REGIONS*

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ABSTRACT

Construction in many places in arctic and subarctic regions is on poorly drained, fine-grained soils, which are perennially frozen a few feet below the surface. Frost heave, landslides, and icings, familiar highway maintenance problems even in temperate regions, are intensified under these conditions. Solifluction, and the differential heave and settlement due to the growth and disappearance of ground ice constitute an additional threat to stability of road surfaces in permafrost areas.

Most construction in permafrost regions is of pioneer roads across virgin terrain; detailed permafrost and engineering-geological investigations are important prerequisites. Highways should be planned and constructed in order to insure good drainage, to avoid areas of ice-rich frozen ground, and to favor sites underlain by coarse-grained foundation materials. Excavation in frozen ground is difficult and expensive, and roads built in sidehill or V-shaped cuts are hard to maintain. The permafrost surface gradually adjusts itself subparallel to cut slopes and becomes a potential glide plane for landsliding or solifluction movement.

Beneath roadbeds the permafrost surface gradually assumes a trough-like form owing to melting by heat reserves built up by the conduction and absorption qualities of road surfacing materials and by abnormally warm seasons. Moisture is trapped in this trough and, unless drained off, may produce serious problems as the result of frost heave and differential thaw of ground ice. Thus, the character of both the drainage and the frozen ground along the alignment should be considered carefully in highway design, and provisions should be made either to reduce or control degradation of permafrost or to eliminate it effectively before construction.

Introduction

Highway construction and maintenance in permafrost regions are characterized by a wide range of unique problems in addition to the difficulties usually experienced elsewhere. Both builders and users of roads in arctic and subarctic areas are plagued by extremely severe frost heaving, subsidence due to melting ground ice, solifluction, and icings caused by the presence of permafrost. These problems basically are the result of construction on saturated, commonly plastic, foundation materials which rest on an impermeable, perennially frozen surface.

Permafrost has been defined variously on the basis of ground temperature, distribution, and physical character. From a scientific standpoint, Muller's definition is the most realistic and useful. He describes permafrost as a thickness of soil or even bedrock at a variable depth beneath the surface of the earth in which a temperature below freezing has existed from two to tens of thousands of years. Thus, it is based largely on "temperature,

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irrespective of texture, degree of induration, water content, or lithologic character" (1945, p. 3). So-called "dry permafrost" is ground at a temperature below freezing but with insufficient frozen moisture to bind the constituent particles and to cause settlement upon thawing. From an engineering view these dry-frozen unconsolidated materials apparently dicer little from unfrozen materials and thus many engineers feel that a definition based on temperature is an abstract concept. If, as the result of construction activity, water is allowed to enter foundations the temperature concept can become extremely important because heaving may result, particularly if the dry-frozen materials are fine-grained.

Permafrost occurs over 24 percent of the total land area of the earth (Black, 1953, p. 127). It is present to some extent in approximately 66 percent of Alaska (Alter, 1955, p. 763) and underlies about 47 percent of the entire U.S.S.R. (Tsytovich and Sumgin, 1937, p. 191) and 40 to 50 percent of Canada (Brown, 1960, p. 163). It occurs nearly everywhere north of the Arctic Circle except beneath ocean basins and the larger lakes and rivers, and it may range in thickness from an average of 800 to 1,200 feet in Alaska and Canada to a reported maximum of 2,000 feet in Siberia (Black, 1953, p. 127). Permafrost thins southward and becomes increasingly discontinuous. At about 60° latitude, it becomes sporadic and generally is less than 100 feet thick. Only locally in Siberia has it been reported south to about 47° latitude (Black, 1950, fig. 1). A recent study by the National Research Council of Canada (Brown, 1960, p. 172) does not indicate its presence south of about 53° latitude in Canada, although inference by Ives (1960, p. 790) places the boundary at about 50° latitude along the Laurentian Scarp in Quebec. The permafrost table may occur anywhere from a few to many feet beneath the surface, but in interior lowland areas underlain by fine-grained soils it generally lies from 2 to 10 feet below the surface.

The temperature of permafrost may range from -0.5°C or slightly higher (Nichols, 1956, p. 8) to -12°C or colder in northern Siberia (Black, 1953, p. 127). Similarly, ground ice content may range from less than 5 percent to more than 1,000 percent relative to the weight of the soil (Nichols, 1956, p. 8). This moisture may occur as finely disseminated ice crystals, as vertical ice wedges, as laminated, thinly bedded ice lenses, or as large tabular vertical or horizontal ice masses (fig. 1). Ground ice may comprise as much as 80 percent of the total volume of the ground in some areas (Taber 1943b, p. 247).

Taber has suggested that permafrost formed in the early Pleistocene, and that areas of perennially frozen ground have been decreasing since that time (1943b, p. 247). This view, however, lacks general acceptance today. Observations by the authors and others (Hopkins and Karlstrom, 1955, p. 115) throughout Alaska suggest that, while permafrost formed, and thawed, at various times during the Pleistocene, it is forming today wherever conditions are favorable. Locally, even relatively coarse deposits in low alluvial terraces with little vegetation cover have become firmly frozen. Unfortunately, direct observations are difficult to make because natural vertical exposures of permafrost are rare and ephermeral. Recent cuts in river and lake banks and artificial cuts associated with placer mining operations permit us to study frozen ground for short periods but these exposures generally show only the upper part and do not reveal the character of the base of permafrost. Well logs and cuttings provide considerable information, but

Figure 1. Tabular ground ice developed in lacustrine silt, Copper River Basin, Alaska. Photo by D. R. Nichols, June 14, 1955.

many drillers do not have the experience to interpret correctly the cuttings. Holes drilled into or through perennially frozen ground may be used to obtain valuable permafrost temperature data through the use of thermistor or thermocouple cables.

Soils in Permafrost Regions

In permafrost regions foundation materials are even more important to highway construction and maintenance than in non-permafrost regions. Unfortunately the soils in most permafrost regions contain large quantities of relatively permeable silt with little or no clay (Taber, 1943b, p. 248, Tsytovitch and Sumgin, 1937, p. 217-218, Kellogg and Nygard, 1951, p. 21). Black (1951, p. 89) believes that at one time a thin silt of eolian origin mantled more than half of Alaska with only the higher parts of mountain ranges and local areas in lowlands being free. Some of these eolian deposits subsequently have been reworked to form thick fills of organic silt in upland valleys, particularly in non-glaciated areas such as Fairbanks (Péwé, 1955, p. 703-706). Thin blankets of eolian silt also cover large parts of glaciated areas in Alaska (Trainer, 1960, p. 22-23, Miller and Dobrovolny, 1959, p. 73, 74, and 83, Péwé, 1951). Silt of glacial, fluvial, lacustrine, and marine origins is even more widespread locally. Silt of residual origin, however, is restricted largely to non-glaciated areas that are underlain by rock types which can be reduced readily by mechanical disintegration. This type of weathering is caused by repeated freezing and thawing and does not produce clay-rich soils (Taber, 1943a, p. 1463, 1464). Chemical decomposition is of little importance in arctic areas.

The absence of appreciable quantities of clay means that the silt can contain considerable moisture which is conducive to the formation of segregated ice crystals (Taber, 1943b, p. 248). Beskow's studies (1933, p. 73) suggest that in silts and sands containing from 5 to 30 percent clay, the clay particles form a colloidal covering on the sand grains and permeability is retained, but porosity is reduced. In this way moisture is allowed to enter but it has little space to expand upon freezing, with the result that ice masses tend to grow and heaving results (fig. 2). When the clay content

Figure 2. Frost-heaved bench mark near Milepost 106, Richardson Highway, Alaska. Heaving of about 8 inches apparently took place largely after vegetation was stripped preparatory to installation of power line. Photo by D. R. Nichols, June 24, 1958.

reaches 40 percent or more, clay fills the sand or silt pores, and the system becomes practically impermeable and no appreciable seasonal or permanent ice heave occurs. Heave, and consequently settlement of the ground, is slight only if the moisture or ice content is less than two-thirds of the water-holding capacity (Liverovsky and Morozov, 1941, p. 8). Taber (1943a, p. 1457) has indicated why this condition is relatively uncommon in permafrost regions, "The thin layer of thawed soil on perennially frozen ground is commonly saturated with water when seasonal freezing takes place . . . since runoff is minimized by vegetation, and the frozen subsoil prevents subdrainage. Freezing tends to concentrate water as ice in the upper part of the surface layer at the expense of water in the lower part, and, where this water is replaced by percolation from a higher elevation, the total water content may be greatly increased locally as the result of the freezing." Taber emphasizes that heaving "results from the growth of ice crystals rather than from an increase in volume that accompanies the freezing of water" (1943b, p. 248).

General Construction Techniques

Highway construction on permafrost in Alaska has, for many years, been based on the assumption that permafrost always must be preserved. To this end considerable time and effort have been expended to retain the natural insulation between the foundation and fill or base course (Ghiglione, 1954, p. 6). These methods, while temporarily successful, commonly do not prevent ultimate differential settlement. Peat and vegetation are compressed gradually by traffic, and instead of insulators, they become good conductors. The compressed mat may be broken and, when foundation materials are fine-grained and saturated, this semifluid mass is pumped into the base course where it tends to cause additional frost heaving. Breaking of the mat also may inaugurate a new drainage pattern which may thaw the underlying permafrost differentially and cause settlement.

The placing of fill material may furnish a temporary period of ground stability. However, in many instances, the large amount of fill needed for stabilization is not economically feasible to obtain. The fill also may cause thawing of the ground because fill materials generally are placed in the summer during maximum thaw and consist of a warm mass that prevents freezing of the ground to the normal depth in the winter. Consequently, the thermal regime is changed and thaw of the permafrost beneath the fill often results, producing differential subsidence in the fill. This problem may be eliminated by placing the fill in winter or early summer. In subsequent years, however, the thermal regime swings to the other extreme wherever the height of the fill exceeds the depth of summer thaw. The permafrost table rises into the fill and may impede surface and subsurface drainage. The use of numerous culverts or very coarse material in the fill will facilitate drainage but may incur problems of frost heave and differential thaw of permafrost under drainage lines.

Thus, it may be seen that most attempts to construct highways on permafrost without disturbing the thermal regime are doomed to failure. The problem of construction on frozen ground must then be resolved in one of two manners. The thermal equilibrium may be altered in such a way as to control degradation and to minimize maintenance costs and disruption of traffic. Or, permafrost may be completely eliminated or degraded by artificial means to a depth where further thaw is unlikely to influence the road surface. This stabilization of the road surface should be achieved before final paving.

Location of Highways

The preliminary location of highways and borrow sites can be done effectively from aerial photographs on which areas of ice-rich frozen ground can be broadly delineated. However, interpretations should be made with care by those trained in the natural sciences and with considerable field experience in permafrost areas (Hopkins and Karlstrom, 1955, p. 143). Photo interpretation should be followed by detailed ground investigations, preferably utilizing a power auger and other means to provide detailed information on soil, ice, and temperature conditions to a depth of 10 to 20 feet beneath the base of the projected road section.

Wherever destination and routing requirements permit, highways should be located, in order of preference, 1) on permafrost-free foundations, 2) on coarse-grained, ice-free frozen materials, 3) in areas underlain by sand and a low permafrost table, 4) on coarse-grained materials with a high ice content (this condition is relatively rare), 5) on fine-grained deposits with permafrost at depth or width, little ice content, and 6) on fine-grained materials containing permafrost at shallow depths or with high ice content. As in nonpermafrost areas, well-drained locations are desirable, not only from the standpoint of increased ground stability and fewer seasonal frost problems, but also to reduce the possibility of differential settlement due to thaw of permafrost (fig. 3).

Sidehill cuts in permanently frozen ground may produce icing conditions or excessive slumping of cut slopes. Deep V-cuts may expose large ice masses that are difficult to impossible to remove and impracticable on which to build. Even V-cuts in ground free of large ice masses but containing

Figure 3. Differential subsidence of roadbed of the standard gauge Copper River and Northwestern Railway near Strelna, Alaska. The drainage and thermal equilibrium of the fine-grained foundation materials were disrupted by the construction in 1911. Maintenance and use of the roadbed were discontinued in 1938 but subsidence continues. Lateral movement of ballast by creep is as significant locally as vertical movement. Photo by L. A. Yehle, Sept. 13, 1960.

small segregated ice crystals may be followed by excessive slumping, gullying, frost heave, and differential settlement. Excavations in permafrost also are very costly and time consuming, in many cases approaching the cost of similar operations in solid rock in temperate regions (Liverovsky and Morozov, 1941, p. 200). Highway excavations generally are made by 1) natural or artificially induced thaw, 2) percussion tools, or 3) explosives. A complicating factor is the difficulty of removing fine-grained materials once they are thawed because they become mushy and will not support the weight of power equipment. A common practice in road building and the development of borrow pits has been to excavate the unfrozen materials over an area to be thawed in the spring and to allow natural radiation to thaw the ground. For maximum effectiveness, the newly thawed ground must be removed continually to permit maximum transfer of heat to the frozen ground.

Maintenance Problems

Almost all highway maintenance difficulties, whether in permafrost regions or not, can be traced either directly or indirectly to improper drainage conditions. These difficulties run the gamut of drainage problems from differential settlement of foundations because of unequal thaw of permafrost (Nichols, 1956, p. 10) to icings which are masses "of surface ice formed during winter by successive freezing of sheets of water (seeping) from the ground, a river, or spring" (Muller, 1945, p. 218). Icings, of course, are known in non-permafrost regions where surface drainage freezes and fills culverts and, in blocking them, forces water across the road surface where it freezes. Equally familiar to most cold areas is the freezing of ground water at points where it issues from sidehill cuts. Icings are particularly prevalent in permafrost regions where deep-penetrating seasonal frost may emerge with an impervious permafrost table under a roadbed and form an effective barrier to downslope subsurface drainage. Hydrostatic pressure builds up to the point where water is forced to the surface, usually along ditches, and freezes in successive flows to form mounds, occasionally as much as 25 feet high (Taber, 1943b, p. 250).

A related drainage problem is that of solifluction which is the slow gravitational flowing of masses of saturated surficial material on the upper surface of permafrost (Liverovsky and Morozov, 1941, p. 27). The end result is similar to that of "creep" and the "viscous flow" of Sigafoos and Hopkins (1952, p. 178-180). Taber (1943a, p. 1458) describes this process as depending on interstitial water, which "adds to the weight of the soil, and acts as a lubricant, thus decreasing stability and facilitating both slow and sudden downhill movements." If this movement is rapid and occurs along a glide plane without internal flow, it becomes a landslide. Landslide movement may be initiated by any change in conditions which upset the temporary stability (Bean, 1950, p. 186). Highway slump-outs and landslides commonly are caused by overloading of a fill or by the addition of excess moisture to unstable materials. Both of these conditions were encountered at Simpson Hill near Milepost 112 of the Richardson Highway in Alaska. This particular section of road crosses a small gully in a long descending sidehill cut. The road crosses the gully on a thick fill. In the spring of 1954, during construction of a new telephone line upslope and parallel to the road, a wide swath of the vegetation cover was stripped through the spruce forest. This caused rapid degradation of the permafrost table during the summer months; and large amounts of moisture, previously locked in the ground as ice, were

released. ...
slumps in the outer part. Early in September of that same year, a maintenance crew with a bulldozer made cuts in the bluff immediately uphill and downhill from the fill and dumped the cut material on the outer part of the road to bring it back to grade. Overnight, a 15-foot wide section of the paved road and shoulder slumped downward through a vertical distance of 10 feet, apparently along a concave glide plane. To repair this major damage, large amounts of silt were excavated along the cut and dumped into the slump area. Almost a year later, to the day, the same portion of the road again subsided 10 feet (fig. 4) following attempts to level minor slumps by

Figure 4. Slump block of fine-grained highway fill material near Milepost 112, Richardson Highway, Alaska. Centerline of road formerly lay over exposed portion of sheared culvert. Bulldozer tracks on slump block surface are a continuation of those at road level on right side of picture. Photo by D. R. Nichols, Sept. 1955.

the addition of a small amount of fill. Subsequently, deeper cuts have been made into the hill in order to obtain the required road width rather than again replacing the slumped material with fill. Inasmuch as landslides, creep, solifluction, sidehill icings, and frost heaving are so characteristic of maintenance problems of roads built on slopes, it is questionable that these sites should receive favorable consideration for highway construction as suggested by Sigafoos and Hopkins, (1952, p. 191) if relatively good drainage can be provided along hill crests or in valley bottoms.

Whenever the growth and thaw of ground ice masses result in uniform heaving or settlement, highway surfaces may be little affected. However, due to differences in soil material and texture, water supply, and rate of freezing and thawing, differential growth and thaw of ground ice are common and may result in severe damage to highways (Taber, 1943b, p. 249). The degradation of permafrost under roadways may be caused by an acceleration of thaw as a result of the road itself. As Black (1950, p. 261) states, "Black-top pavements are good conductors and heat absorbers in summers and can destroy permafrost" or cause its rapid degradation. However, an even greater factor in depressing the permafrost under roadways has been suggested recently by Greene and others (1960, p. B143-B144). These authors contend that the roadways are ". . . sensitive to random climatic variations from year to year and hence the deep thaw is accentuated during an anomalously warm season. If the excess water formed by melting ice in the surficial permafrost layers can drain off, the thickened active layer will be drier and more easily thawed in subsequent years.

"This, of course, will result in a settling of the roadway at the point where this progressive deep thawing occurs. The water would be expected to migrate in the thawed trough beneath the roadway until it is trapped in a basin, or escapes by exterior drainage. When it is trapped in a basin, as when the road crosses a swale, or a large culvert, the water is ultimately refrozen and some heaving might be expected."

Because of this trapped moisture reserve beneath the highways, engineers should continue temperature and moisture state research during periods of spring thaw and should consider the extension of these studies to include periods of fall freezeup. The restriction of traffic loads during break up, when subgrades are saturated and susceptible to pumping action, is an established procedure for flexible pavements in Minnesota (Lawrence, 1952, p. 284), Alaska, and elsewhere. To aid in the prediction of critical periods of reduced load-carrying ability, the Bureau of Public Roads in Alaska and locally in the smaller 49 states (Turner and Jumikis, 1956, p. 24) has utilized soil moisture and temperature data obtained from fiberglass soil moisture units installed in roadbeds. It is suggested here that wherever the water table may lie close to the road surface and especially in permafrost regions, moisture movements in the subgrade, brought on by heavy loads during freezeup, may cause differential freezing and heaving.

Conclusions

"The structural integrity and endurance of almost every engineering work is jeopardized by the action of water" (Hveem, 1952, p. 110). If possible, this statement is even more appropriate when the water is in a frozen state and when applied to permafrost rather than to non-permafrost regions. Much of Alaska and other Arctic areas are underlain by silts of high permeability and relatively low porosity, and ice masses of both seasonal and permanent nature are common. When the ice becomes thawed, the character of the soil may change, in some cases almost overnight, from a state similar to bedrock to one resembling syrup. As a result, construction practices normal to more temperate areas require drastic revision in permafrost regions.

With almost 2,000 miles of primary connecting road now in existence in Alaska and an additional 2,000 miles projected for the not too distant future (Gruening, 1959, p. 5, 22-25), it is becoming more and more apparent that construction and maintenance problems in permafrost regions be recognized perhaps not entirely unique to roads in permafrost regions, but certainly of primary concern, are: severe frost heave, differential heave and settlement due to growth and disappearance of ground ice masses, icings, and solifluction, slump-outs, or landslide phenomena. These problems result largely from the disruption of the thermal equilibrium of frozen ground by construction activity, the effects of highway traffic, or improper maintenance practices. This disruption, whether intended or inadvertent, most often causes changes in the surface and subsurface drainage characteristics of the construction site.

Immediate solutions to these problems concentrate on improvement of drainage conditions. Long term remedies are based on proper initial highway location and construction procedures. The following measures are suggested:

1) Choose non-permafrost foundations for construction wherever possible.

2) When necessary to construct on permafrost, ice-rich areas should be delineated and generally avoided.

3) Construction methods should be employed that result in only a minor disturbance of the permafrost or that remove it entirely before the final surfacing is applied. The choice between these alternatives should be based on detailed studies of the distribution, temperature, and character of the permafrost, as well as the foundation materials and drainage conditions along a projected route.

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HIGHWAY SALVAGE ARCHEOLOGY

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The various Highway Salvage Programs, of which highway salvage archeology is a part, are some of the newer additions to the well-established governmental policies directed toward conservation of our national resources. The resources involved include the abandoned forts, trading posts, and settlements of early pioneers, the more ancient dwelling sites and religious structures of prehistoric Indians, and the still older remains of extinct animals and plants. These resources are of major importance to early American history; they are the *only* resources of prehistoric archeology and paleontology.

These remains have no immediate industrial use and are not comparable to such resources as iron ore, mineral deposits, and forests in this respect. They are, however, the basis of such useable things as our state and national park systems, our museums, and they frequently serve as attractions for tourists and visitors. Finally, they provide the raw material for learning about our colonial past, about our human predecessors on this continent, and about the animals and plants of the more ancient past. As such, these resources have proved to be a matter of interest not only to academic specialists but also to large sectors of the public as a whole.

It is of particular importance to remember that the resources of which I speak *are* comparable to ore and mineral deposits in that they can be exploited but once and never replenished. They are not comparable, on the other hand, to such things as forests, which can, with proper planning, be supplemented and replenished. If historic, archeologic, and paleontologic sites are excavated by competent specialists and the remains are preserved, restored, analyzed, described in published reports, and properly exhibited to the interested public, then—and only then—can we feel that we have used these resources responsibly. If, however, the sites are destroyed, whether by individuals for the sake of personal curiosity or by responsible organizations engaged in projects that are vital to the public welfare, the historic, archeologic, and paleontologic resources have not been properly used and are irrevocably lost.

During the past twenty years there has been a tremendous increase in major construction projects. Dams are being built throughout the United States for purposes of power production, flood control, and irrigation. Our highway system is being improved and enlarged. A vast network of pipelines is being installed in the western states by the petroleum industry. These projects, basic to our national welfare, are of necessity destroying not dozens, not hundreds, but literally thousands of archeologic sites. In recognition of this situation, the agencies involved have instituted a number of programs designed to salvage such information as is possible without hindering the construction projects involved.

Programs to salvage historic, archeologic, and paleontologic information are not new. They were, in fact, instituted by the federal government in the early 1930's. One of the first major projects of this nature was conducted here in eastern Tennessee, in conjunction with the construction of Norris Dam.

In the Federal-Aid Highway and Federal Revenue Acts of 1956, Congress authorized the use of moneys from the 1½% "planning fund" for the excavation, preservation, and removal of historic, archeologic, and paleontologic remains from within road right-of-ways in any state "to the extent approved as necessary by the highway department of that state." This policy was set forth in Section 120 of that act (presently in Section 305, Title 23 of the United States Code) and further supported by the Bureau of Public Roads Policy and Procedure Memorandum 20-7, October 19, 1956. Subsequently, provisions were made to use funds from the 1½% money to finance advance archeological surveys along projected right-of-ways (*cf.* Office Memorandum 26-01, E. L. Armstrong, Commissioner, Bureau of Public Roads, dated April 15, 1959).

It must be emphasized that, although use of 1½% funds for these purposes has been authorized by federal authorities, the initiation of these programs must almost always come at the state level—specifically, an agreement must be reached between the State Highway Commission and the Chief Highway Engineer on the one hand and a responsible and competent state institution such as a university, historical society, or museum on the other. (Highway projects financed entirely by federal funds are the major exception.)

I would like to turn now to a few more specific considerations. First of all, there is the matter of just how highway projects endanger these remains. The most obvious and most frequent situation involves historic and archeologic sites located within highway right-of-ways; they are unavoidably destroyed by heavy equipment preparing the road bed. In other instances, borrow pits and various operations to obtain fill-dirt frequently result in the destruction of sites not located within the actual right-of-way.

In Nebraska, for example, the most suitable fill-dirt deposit for highway crews is precisely that which was most suitable for the establishment of the large Pawnee villages and other prehistoric settlements. This unfortunate situation is not mere coincidence, and I can offer no proof that highway engineers, as a group, are in collusion to eradicate Pawnee villages from the face of the earth. It is simply a practical matter—the Pawnee needed good dirt for farming and the engineers need it for building roads; both groups have found the best dirt available.

In the southeastern United States a rather different situation occurs from time to time. Construction engineers in need of fill-dirt have occasionally noticed that someone has obligingly collected large piles, or mounds, of dirt for them. The pile may be as much as 50 feet high, or it may be only 3 or 4 feet high. A pile of dirt is a pile of dirt, and frequently it is carried away and incorporated into our national road system. A few project engineers (but rarely the ones in question) may know that these mounds are actually religious structures of prehistoric Indians, but such knowledge is entirely fortuitious and not pertinent to their competence as construction men. For this reason, a very great number of mounds have been lost.

Paleontological deposits are encountered occasionally during normal roadbed construction, but are most frequently exposed in making road cuts. Making road cuts is, of necessity, a rather violent affair, and when fossilized bones are found in this manner they are usually removed from their original matrix or blasted into useless fragments. The intimate connections between geology and paleontology are appreciated better by my audience than myself. These relationships are invaluable in understanding the ecology and environment of all past geologic periods; they are of particular importance to the rapidly developing field of Pleistocene Geology.

The operation of a highway salvage program is a straightforward affair. The recently approved advance-surveys of rights-of-way provide planning departments with knowledge of major sites that may be in their way. Frequently, plans can be altered slightly to avoid disturbing the site, and the matter ends there. If and when it is necessary to destroy sites, the archeologist or paleontologist moves in ahead of the construction gang with a few laborers—usually 5 to 10. He salvages whatever information he can, but *in no case* does he hold up road construction.

It is most important, from our standpoint, that project engineers be assured that they will never suffer costly delays in reward for cooperating with the archeologist. Archeologists and paleontologists fit their excavation techniques to suit the construction schedule. This is not ideal in many cases, of course, but it is absolutely necessary.

Salvage programs have been or are being initiated in many states—Arizona, California, Florida, Illinois, New Mexico, Oklahoma, Pennsylvania, South Dakota, Washington, and West Virginia are examples. A great deal of information has been obtained from these programs and this has resulted in considerable satisfaction to all parties concerned. It should be noted that these programs have also generated a great deal of good-will, not only between academic specialists and project engineers, but also between the interested public and their state highway commissions.

In closing, I shall not make a plea for all of the highway engineers in the audience to rush out and begin salvage projects in their respective states. These programs must be set up with considerable deliberation, and only when the highway authorities can be assured that, should such a project be started, they will be able to obtain the services of an archeologist and/or paleontologist when and where they want him. The shortage of trained academic personnel, rather than lack of cooperation between highway commissions and universities, is the major difficulty in establishing such programs at present.

I would of course, like for you to be aware of the importance and value of these relatively inexpensive programs, should an opportunity to begin one exist. I would also like to solicit your cooperation in reporting any peculiar bones or mounds of dirt that you may encounter in your work to your state university or other responsible agency, even though they may be almost completely destroyed by the time they come to your attention. Such cooperation by big-time earth movers will be greatly appreciated by small-time earth movers like myself.

USE OF CONTINUOUS SEISMIC PROFILER (SPARKER) IN GEOLOGIC INVESTIGATIONS FOR VEHICULAR TUNNEL AND BRIDGE CROSSINGS

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Houston, Texas

The Continuous Seismic Profiler (Sparker) has been used in the past three years for a number of engineering geological surveys in water-covered areas. The general principles of operation are shown in Figure 1. The method of operation is similar to that for a conventional fathometer. The record shows a continuous profile of the water bottom and of the geologic strata beneath the bottom. The sound source is the underwater detonation of a 12,000 volt spark; its sound spectrum is similar to that of a small blasting cap. Reflections are obtained from geological strata down to depths of 600-800 ft. beneath the bottom. The record produced is a geologic cross section, the horizontal scale being horizontal distance along the survey boat's track and the vertical scale being depth.

A number of articles have been written on the Sparker. These include *Deep Sea Research,* vol. 4, pp. 36-44 (1946); *Geophysics,* vol. 24, pp. 749-760

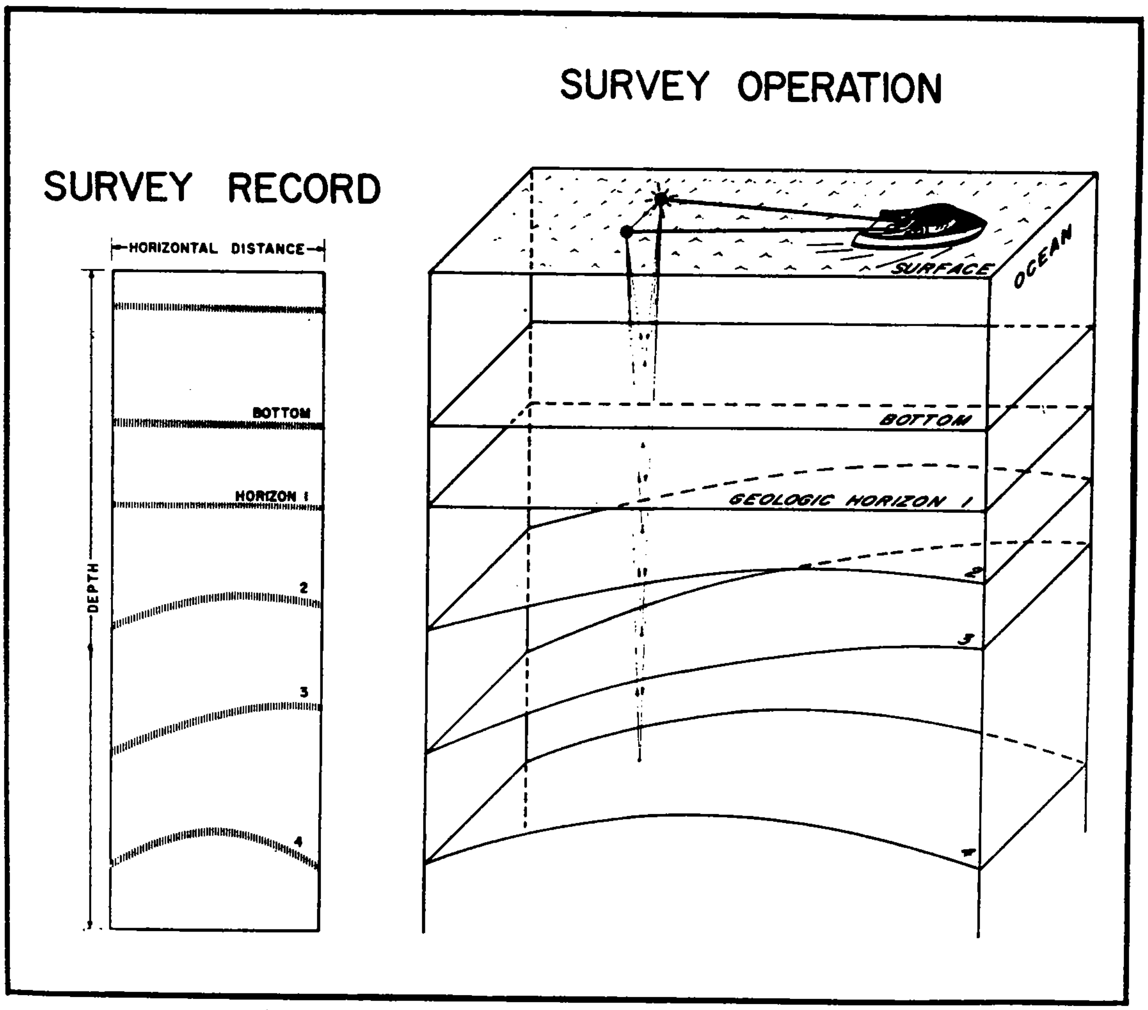

Figure 1. Principles of Sparker Operation.

(1959; *Petroleum Week*, March 13, 1959; *World Oil*, April, 1959; *Offshore*, July, 1959; *Oil Week*, November 6, 1959; *Petroleum Week*, June 17, 1960; and *Offshore*, March issue, 1961. In view of this the discussion here will be limited to a description of some sample Sparker records. These records have been chosen from various areas with different underlying geological conditions.

Of interest for highway problems have been a survey for the proposed tunnel beneath the English Channel, surveys for a bridge and tunnels across Tsugaru Straits and Akashi Straits, and a survey for a bridge in Hong Kong.

Figure 2A shows the record from offshore Louisiana and *Figure 2B* shows the record from Lake Erie. The corresponding soil boring is shown adjacent to each record. The offshore Louisiana record shows clearly the water bottom, the bottom of the soft clay, and a sand lens within the stiff silty clay. In addition the reflection from the silty sand layer within the stiff clay is shown weakly. The Lake Erie record shows clearly the reflection from the water bottom and from the bottom of the glacial till. In general the strength of the reflections from various strata beneath the bottom will depend on the difference between the mechanical properties and density of the material above and below the particular reflecting horizon.

Figure 3 was taken near the edge of the continental shelf offshore California. It shows the ocean bottom increasing in depth from 200 ft. to 340 ft. from left to right on the record. Beneath the water bottom is a reflection from the unconformity surface between the overlying overburden of recently deposited material and the underlying sedimentary rock of Miocene age. The reader will note that the overburden of recent material thins toward the edge of the continental shelf. Beneath this unconformity surface are reflections from the Miocene strata dipping moderately steeply to the right.

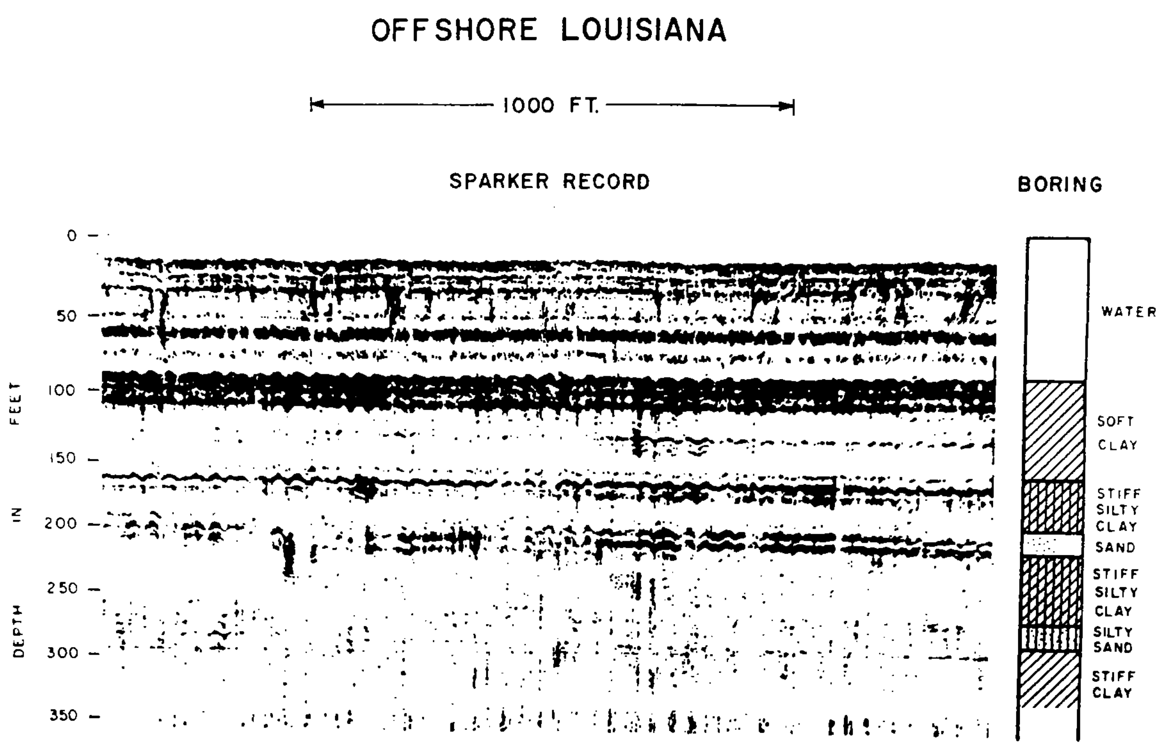

Figure 2A

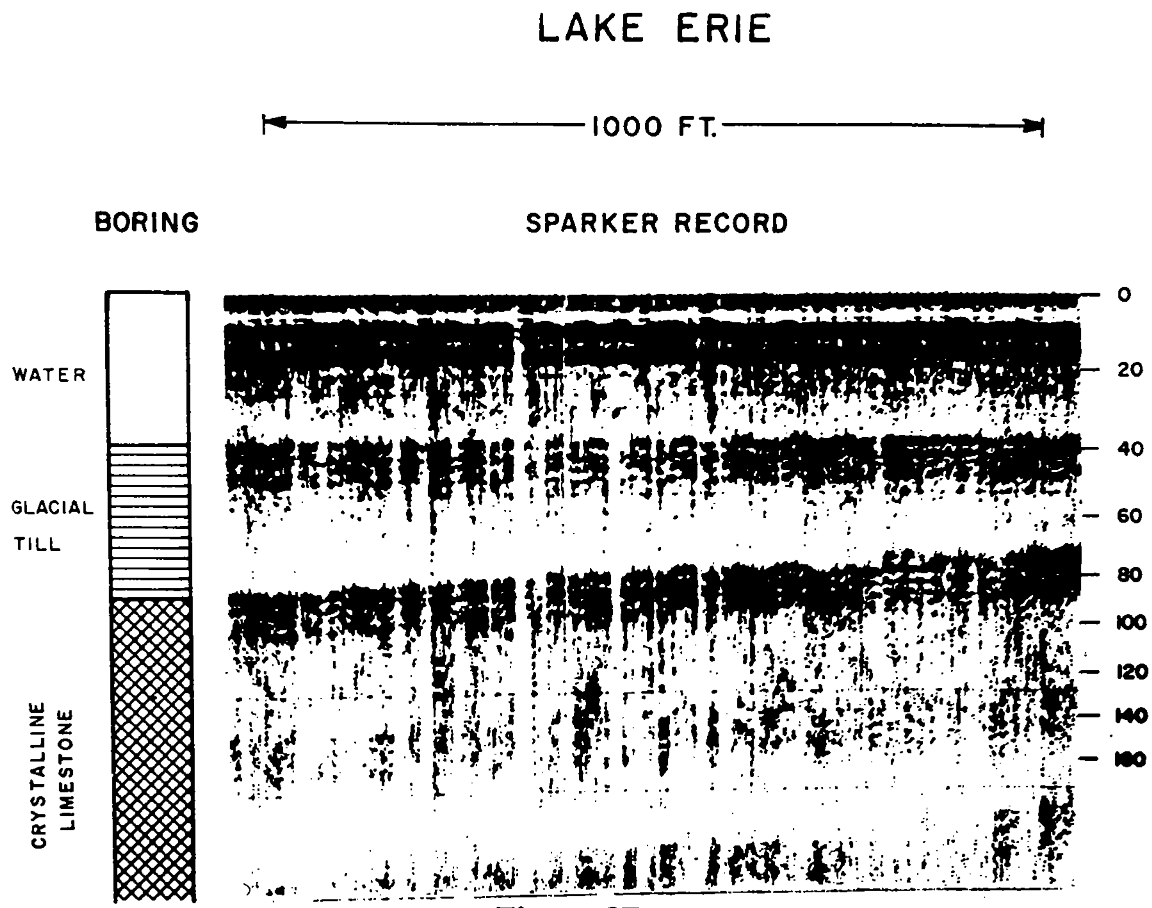

Figure 2B

SURVEY OFFSHORE CALIFORNIA

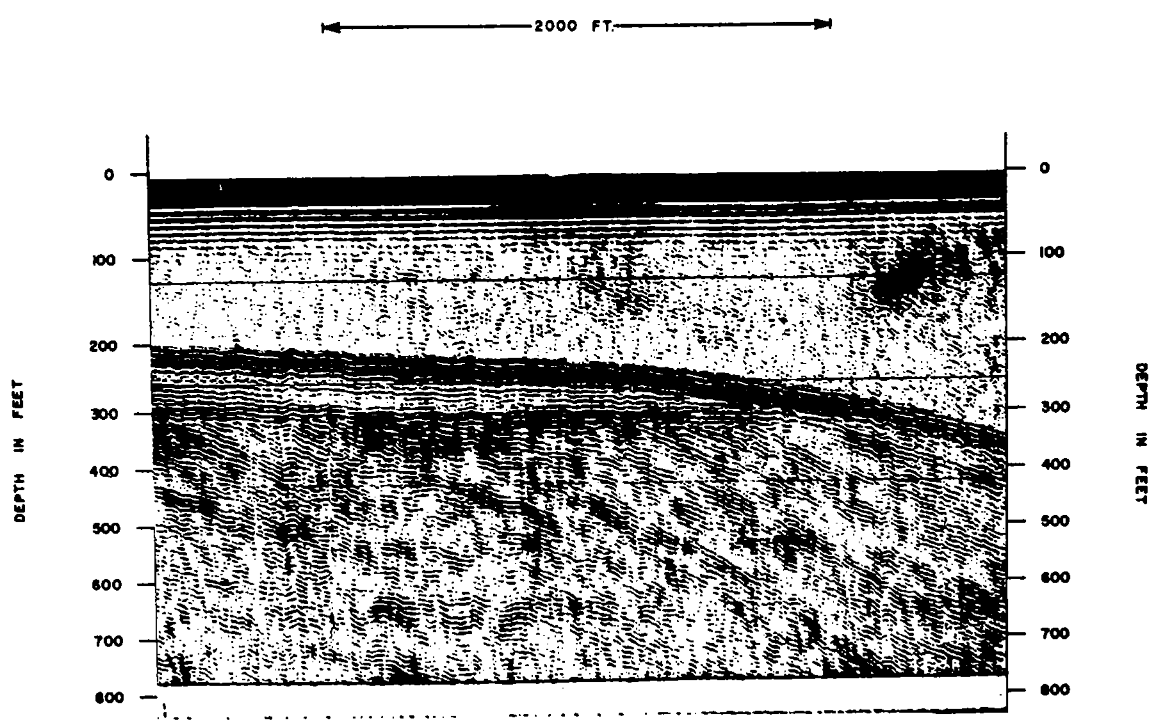

Figure 3. This is a Continuous Seismic Profiler (Sparker) record offshore California. The water bottom is at a depth of 200 ft. at the left hand edge of the record and dips offshore to the right. The bottom of the overburden is at 280 ft. at the left end of the record the overburden thins and disappears to the right. The reflections beneath the bottom of the overburden are from geologic strata of Miocene age.

SPARKER RECORD, JAPAN

AKASHI STRAITS

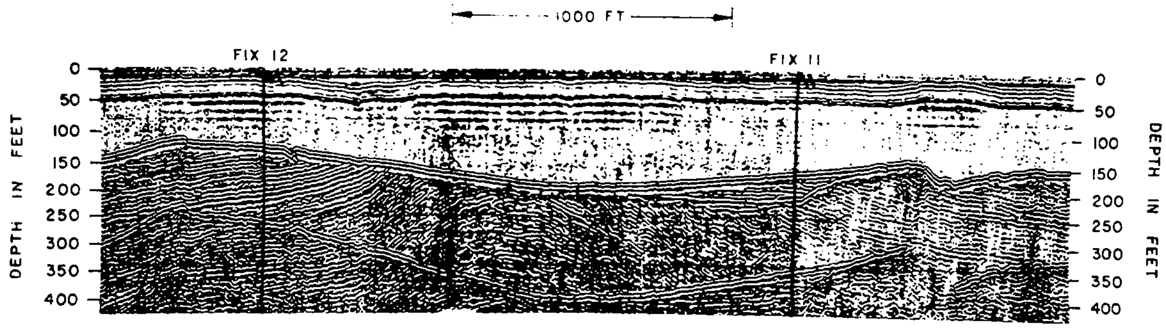

Figure 4

Figure 4 was taken offshore Japan. It shows fairly complicated geology. The water bottom is the first reflection shown on the record. Further down the record at twice the depth interval of the bottom reflection is a multiple reflection from the bottom; this latter reflection does not represent underlying geology. Near location line 11 is a small pocket of channel fill. Beneath the water bottom and the channel fill the strata dip to the left on the left hand portion of the record and to the right on the right hand portion defining a distinct, sharp crested anticline. Just to the right of fix 11 is a fault. Also about two-thirds of the distance between location point 11 and location point 12 is a second fault, shown by steeply dipping strata to the right and more gently dipping strata to the left.

Figure 5 was taken offshore Trinidad in the Gulf of Paria. It was also taken over a relatively complicated piece of geology. The water bottom

SURVEY IN THE GULF OF PARIA

SPARKER PROFILE ACROSS A MAJOR FAULT ZONE IN THE GULF OF PARIA

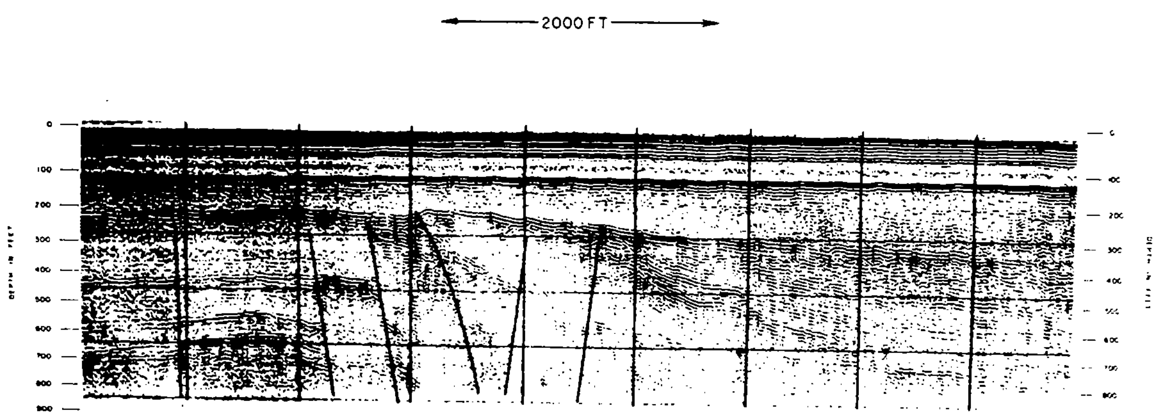

Figure 5. Direct wave is at 20 ft. on the scales, the water bottom reflection is at 110 ft., the mud bottom reflection is at approximately 220 ft., the geologic reflections are beneath the mud bottom reflection. The strong faults are indicated on the record. Note that one fault shows a displacement of the mud bottom also note the increase in mud thickness in the down dropped section to the right.

remains at a depth of about 110 ft. across the record. Beneath the water bottom is a reflection from the bottom of the overburden increasing in depth from about 220 ft. on the left hand side of the record to 330 ft. on the right hand side. The geologic strata beneath the overburden have been severely faulted as shown by the six faults indicated on the record. The reader will note that one of the faults must have had relatively recent movement as shown by the change in level of the reflection from the bottom of the overburden.

Geophysical surveys made with the Sparker are usually straightforward and require small amount of operating time. The equipment can be mounted on any boat suitable for the area of operation. Installation takes a matter of two to four hours. The survey personnel consist of two people. Surveys are made with the boat under way at a speed of three to four miles per hour; survey records are obtained on a continuous basis and profile in the form of a geologic cross section the geology underlying the survey ship's track. For a given engineering survey of a water crossing a total of two to five survey days are usually adequate to obtain a detailed map of the area.

MADISON RIVER-HEBGEN LAKE EARTHQUAKE AND HIGHWAY PROBLEMS[1]

REED W. BAILEY

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In the Rocky Mountains, a geologically and topographically young region, earth movements such as faulting are, in geologic terms of reference, common phenomena. Earthquakes have occurred in the past and will again occur in the future. In this region main highways cannot avoid fault zones, and to traverse the country they must pass over geologically active fissures. The characteristically north-south mountain ranges are generally delineated by faults, and gaps or passes carved by streams, through which roads commonly run, are often associated with them.

Just as it is impossible to avoid potential earthquake areas in locating highways, it also appears to be impractical or even impossible, because of the magnitude of the forces released in faulting, to construct highways that could not be damaged or destroyed. However, highway people must cope with repair, relocation, and reconstruction of highways damaged by earthquakes; therefore they should be interested in the earthquake phenomena and effects that faulting, which produces earthquakes, has on the surface features of the earth.

The Madison River-Hebgen Lake earthquake that occurred in August 1959 should be of particular interest to geologists and highway people because of the extent of studies that have been made and are being made of this earthquake area, and because a main highway traversing the area was badly damaged. This quake was unusually severe and produced many spectacular phenomena. It has been studied by men of widely diverse interests—geologists, seismologists, hydrologists, engineers, and foresters. Publications on numerous aspects of this quake have been issued. Studies are continuing, and one can expect that valuable geological and seismological contributions will be made to our knowledge of earth processes.

Widespread interest in the earthquake was reflected by the coverage given it by newspapers and magazines in this country and abroad. Many things contributed to this interest. Approximately 500 persons were trapped in the area by destruction and damage to the highway, and upwards of 28 lives were thought to have been lost. The rescue of the severly injured by helicopters and the parachuting into the area by Forest Service personnel equipped with first aid and emergency supplies were dramatic and caught the imagination of people everywhere.

The Madison River and Hebgen Lake area in the Gallatin National Forest (fig. 1) constitutes one of the finest fishing and camping opportunities in the national-forest system. The beautiful mountain scenery, forested slopes, and clear running streams brought many people to this area annually. Then, too, Yellowstone National Park is close, and on the night of the quake was heavily populated with tourists who experienced some of the effects of the shock. Most geysers and hot springs were altered, highways were blocked

[1] The author has borrowed liberally from available publications dealing with the quake, and has talked with geologists and engineers familiar with the area. A list of references is appended.

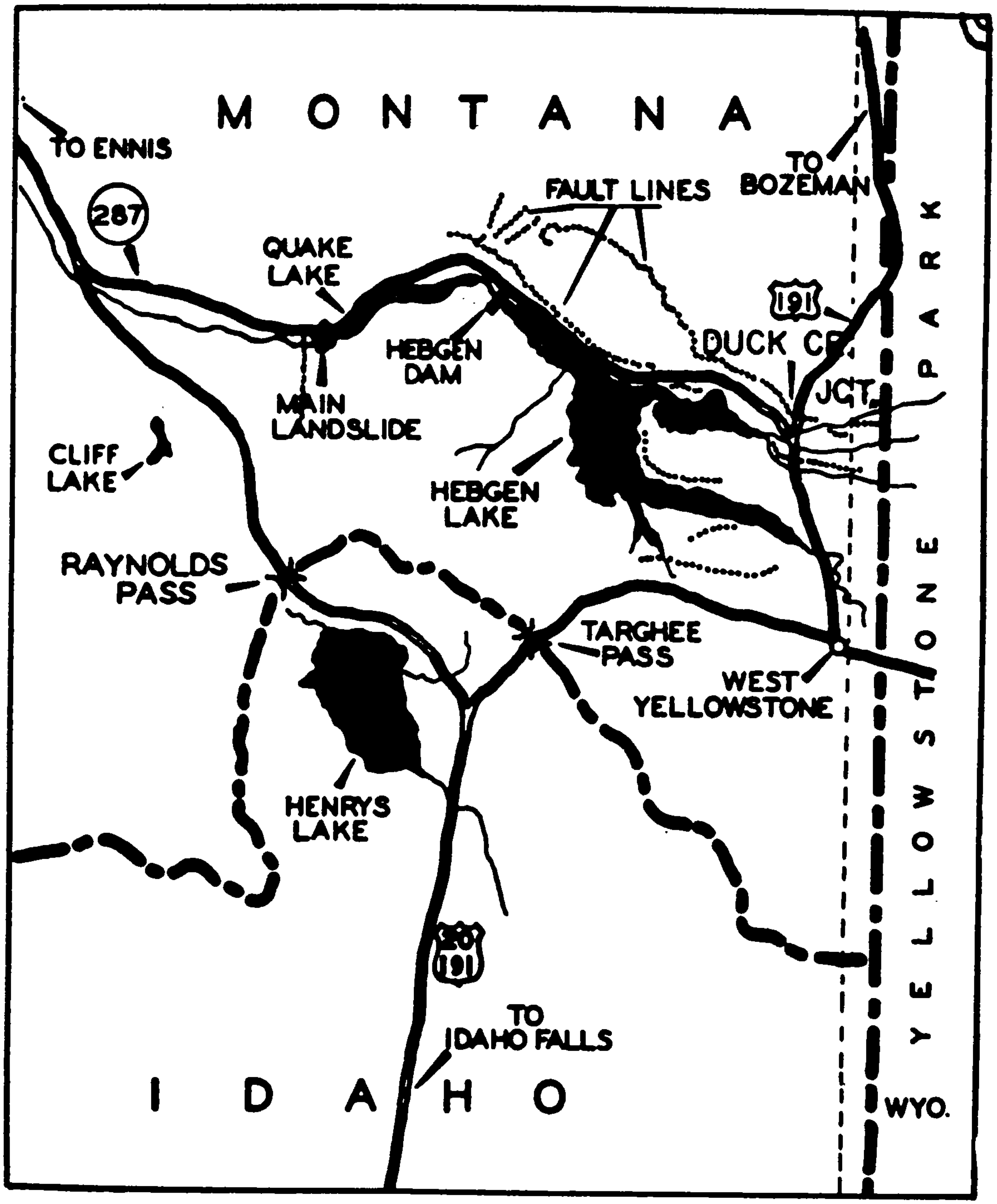

Figure 1. Hebgen Lake earthquake area. Dotted lines show location of recent active faulting. (Montana Power Company map.)

by falling rocks, and buildings were damaged. Under these circumstances the quake became of great general interest, and the phenomena produced by the faulting gave the scientists and engineers a rare opportunity to study earthquakes and their manifestations.

"Geologic Area," Gallatin National Forest

Because of the location of this earthquake area, the spectacular evidence of earth movements, and the anticipated interest of the traveling public, the U. S. Forest Service has set aside 37,800 acres as a geologic area. It will be provided with campsites, roads and trails, and guide service to enable tourists to see and learn at first hand the effects of a geological event and the processes involved in the shaping of the landscape of the Madison Range and adjacent valleys. Last summer, even with no through traffic, 370,000 people visited the quake area, presaging the heavy use this area will receive in the future.

The Earthquake and Its Effects

The initial and heaviest earthquake shock occurred at 11:37 p.m., August 17, 1959, and continued for about 40 seconds. Subsequent shocks, whose severity ranged between 5.5 and 6.5 recurred during the next several days. Minor shocks continued to be recorded at Bozeman, Montana, for several months following the major shock.

The epicenter of the quake was calculated to be at a point near Duck Creek Junction of U. S. Highway 191 and State 287, approximately 7½ miles north of the town of West Yellowstone. Seismograph station at Pasadena, California, reported the magnitude of 7.1 on the Richter scale, which was only 1.2 of a point less than that of the San Francisco earthquake of 1906. It is interesting to note that the Tokyo earthquakes of 1923 were 8.2 and 7.7 respectively. Recent Chilean earthquakes ranged from 7.25 to 8.5. The Hebgen Lake earthquake along the Madison River will be recorded as a major earthquake. The shock of this quake was felt over an area in excess of 500,000 square miles.

The Hebgen earthquake area, which includes Yellowstone Park, is in an active seismic belt extending from north of Helena, Montana, through southeastern Idaho, to southern Utah (fig. 2). More than 70 earthquakes of medium or greater intensity have been recorded for this region from 1852 to 1959; some of these were of high intensity. Damage reported has not been great compared to that from equally violent quakes in densely populated centers. However, the potential for damage is high, and past frequency indicates that earthquakes in this elongated belt can be expected in the future.

The area most affected by the Hebgen Lake earthquake constitutes a narrow belt adjacent to the Madison River, Hebgen Lake Dam, and the mouth of the Madison River Canyon. State Highway 287 traverses this area, was affected greatly by the faulting, and is adjacent to many of the most spectacular features of the earthquake.

The appearance of new faults begins near the junction of U. S. Highway 191 and State 287 near Duck Creek; fault scarps are pronounced features in the landscape as far as the Cabin Creek area, a distance of 14 miles. Many breaks in the earth's surface appear along this sector, and fault scarps showing vertical displacement of as much as 22 feet occur (fig. 3). Red Canyon and Hebgen faults which are the most conspicuous, begin near the east end of the deformed belt in a zone of faulting. These two scarps become more distinct and separate as they extend northwestward. They show a maximum divergence of about 3 miles, and end in another zone of faulting, which is made up of a large number of displacements near the Hebgen Dam and Cabin Creek (fig. 1).

In addition to vertical displacement along the faults, the area suffered subsidence. Levels run by the Coast and Geodetic Survey before and after the earthquake showed Hebgen Lake basin to have been warped and tilted with a maximum subsidence at midlake of 18.8 feet, and bedrock supporting

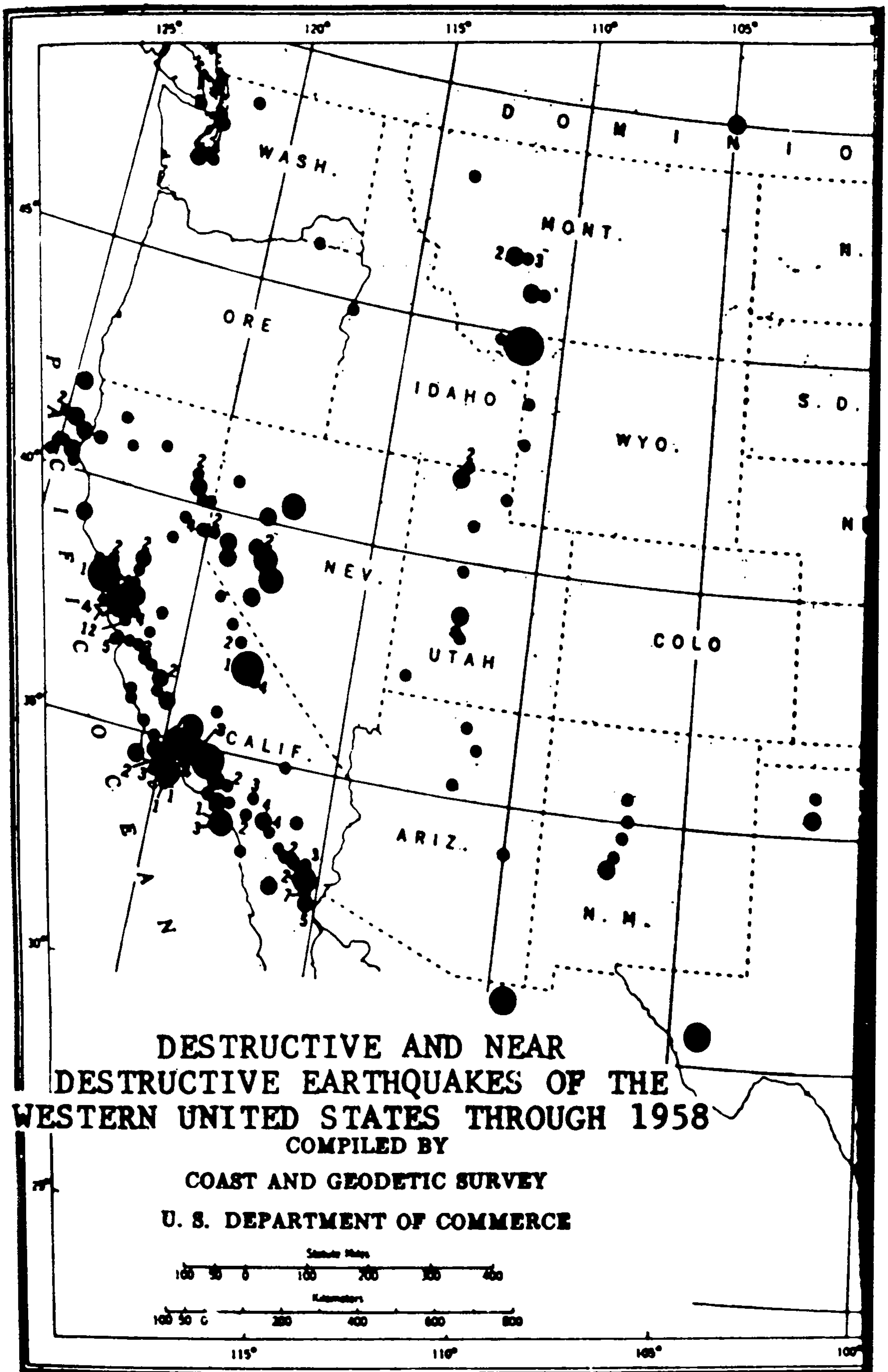

Figure 2. Seismic belt extending from Canadian border to Mexico and passing through the Hebgen Lake earthquake area. Size of dots reflects the frequency of destructive or near destructive earthquakes through 1959.

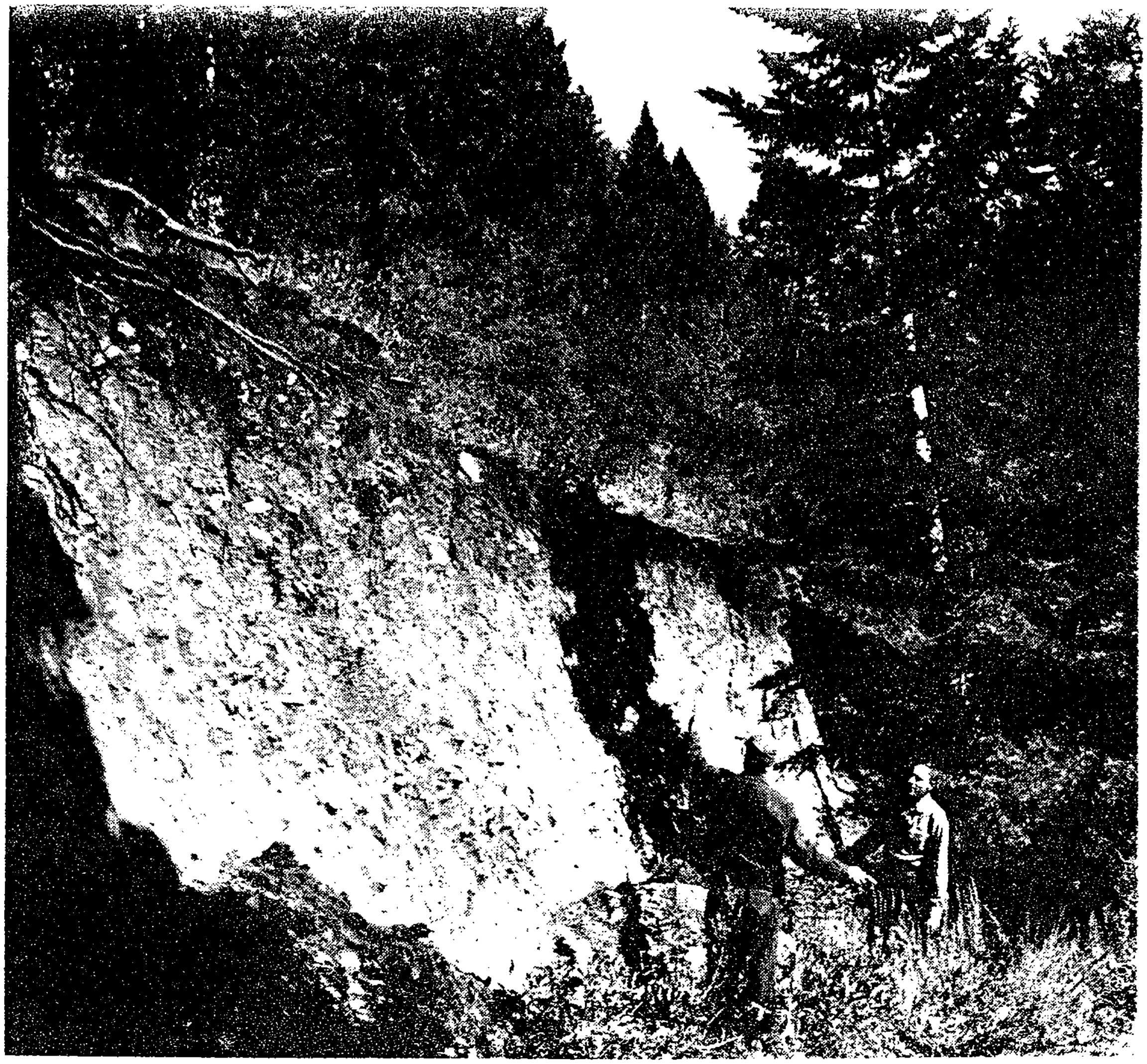

Figure 3. Twenty-one foot vertical displacement along fault paralleling Highway 287 near Hebgen Lake. Where similar displacements occur in colluvial material on steep slopes cracks and slumping above and below scarp often pronounced. (U. S. Forest Service photo.)

the dam dropped 10 feet. The lake basin tilted to the north so that the area along the north shore formerly above water level was inundated, while stretches of beach on the south side were raised above the lake level. Dropping and warping of Hebgen Lake basin set up huge waves of oscillations whose period was 17 minutes; these oscillations lasted 11½ hours. Initially waves were large enough to overtop the dam, and they damaged the downstream fill. The shattering, slumping, and twisting effect of the deformation and the pounding of the waves did extensive damage to the dam. However, it held and was later repaired.

Highway 287 begins at Duck Creek Junction (fig. 1), where it leads off from U.S. 191; it extends northwest and west down drainage, passes along the north shoreline of Hebgen Lake and Dam, through a narrow, twisting canyon that opens out into Madison Valley. After leaving the narrow canyon—a cut through the Madison Range—this highway runs north to Ennis, where it joins transcontinental highways. From Duck Creek Junction to the mouth of the canyon, a distance of 20 miles, this highway passes through a beautiful and heavily used recreational area of the Gallatin National For-

Figure 4. One of a number of breaks with considerable vertical displacement that severely damaged highway. (U. S. Forest Service photo.)

est. It is also an important approach to the Yellowstone National Park from the Missoula-Butte-Helena centers of population.

This highway suffered considerable damage, and in several places was completely destroyed (fig. 4). There was vertical warping, and in places the downwarp was sufficient to cause the road to be inundated by Hebgen Lake. The highway bridge across Duck Creek dropped 6 feet in elevation. Longitudinal cracks from a few inches to 2 feet in width opened up along sections of the highway, and shattering of the surface was pronounced. The road near several of the bridge approaches settled so much that bridge decks were 1 to 2 feet above the surface of the road. In four places the road slipped into the lake; one of these left a slide scar 1,200 feet long and 350 feet wide, and the shear plane face was 80 feet from the water to the natural ground. One hundred ten thousand cubic yards of colluvial material slid into the lake. Another slide that destroyed the road was 400 feet long, 150 feet wide, and had a slip face 85 feet above the water level. Faults cut the road and produced scarps up to 6 feet in height. Rocks, many of which weighed several tons, broke loose from ledges and rolled onto the highway and into campgrounds with devastating effects (fig. 5).

Madison Canyon Slide

The most spectacular effect of the earthquake was the great slide that occurred near the mouth of the Madison River Canyon, 7 miles below Hebgen Dam (fig. 6). This slide dammed the Madison River, covered the high-

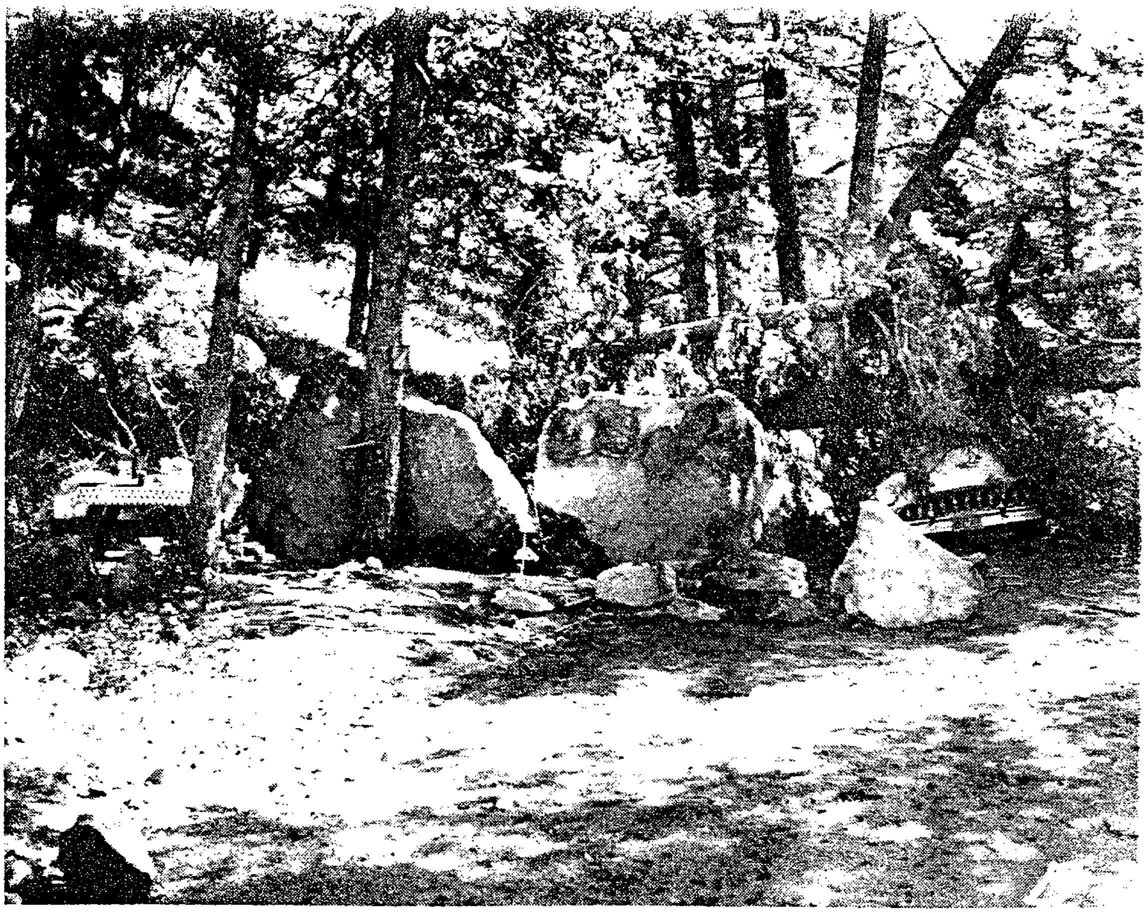

Figure 5. Boulder shaken loose from ledge up slope by the quake cascaded into this campground, killing two people. Roads were blocked, trees destroyed, and campgrounds obliterated by such rock falls. (U. S. Forest Service photo.)

way to great depth, and formed a lake that inundated 6 miles of the highway.

The first shock of the earthquake triggered the movement of a large mass of rock with an overburden of soil and trees from the south slopes of the canyon. This slide material swept into the canyon and up the north slope, where its vertical depth was 420 feet above the highway. As the slide reached the bottom of the canyon it spread out both up and down the canyon for a distance of about three-quarters of a mile. The depth of the slide material immediately above the stream bed was 200 feet. The total volume of the material in the slide has been estimated to be 40 million cubic yards. It was at the campground in the bottom of the canyon covered by the slide that 18 persons are thought to have been buried.

The slide material was made up of pre-Cambrian dolomite, schist, and gneiss. Foliation planes of the rocks dipped toward the canyon at about a 40-degree angle. A massive outcrop of dolomite at the base of the slope before the slide occurred acted as a support or buttress to less stable schist above. Apparently the quake shattered this dolomite dike, and schists broke along cleavage planes and slid into the canyon. Material came to rest in the same relative position that it occupied before the slide took place, indicating that there was little churning or rolling of the mass. The top of

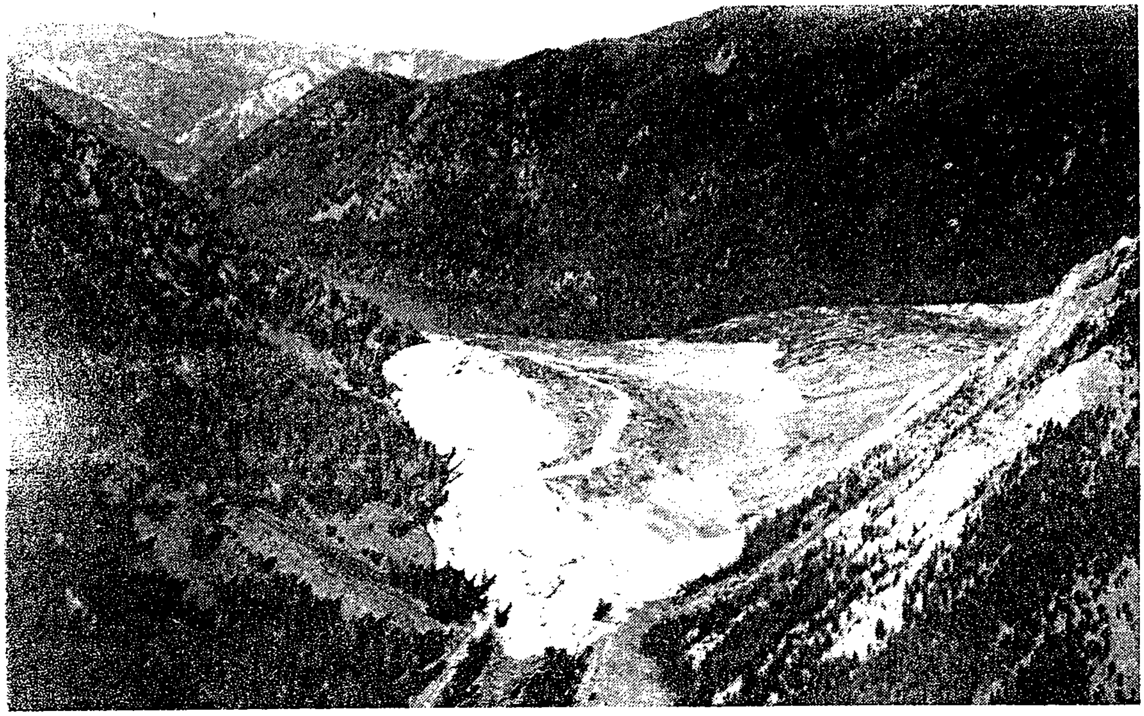

Figure 6. Looking east at great slide near mouth of Madison River Canyon which buried Highway 287. Quake Lake forming back of slide dam. Forty million cubic yards of schist, gneiss, and dolomite rock slid from the south wall of the canyon. Problems of highway relocation begin here. (U. S. Forest Service photo.)

the slide scar is 1,400 feet above canyon bottom, and the break was beyond the crest of the ridge.

As the lake, now named Quake Lake, began to form back of the dam, it became obvious immediately that a dangerous flood threat was in the making. If a flood occurred, it would spread disaster downstream to the town of Ennis and other small communities along the river. This threat resulted from the damage already done to Hebgen Dam, from the continuing quakes, and the creation of a new lake that already was rising rapidly and would eventually overtop the slide.

The Corps of Engineers of the U.S. Army was requested to take over the task of alleviating the flood danger. The Corps immediately built roads to the top of the slide dam and began constructing a spillway over it. Upon completion of the spillway, which was ready to receive the first overflow from the impounded water, it became evident that the gradient of the outlet was too steep to assure stability of the channel; so, with the help of the running water and dragline and bulldozer equipment the new spillway was lowered 50 feet. The following tabulation published by General Keith R. Barney of the U.S. Army lists the operation of the newly formed dam and the amount of material excavated.

Excavation for spillway in the dry (initial)	600,000 cu. yds.
Spillway armoring in the dry (initial)	160,000 cu. yds.
Random fill in spillway bulldozed in from south side	100,000 cu. yds.
Rock hauled and placed in spillway 12 September through 9 October)	640,000 cu. yds.
Rock excavated from spillway during cutdown (50-foot lowering)	700,000 cu. yds.

and deposited in delta	1,750,000 cu. yds.
Construction of access and haul roads	20 miles

At completion of the new spillway by the Engineers on October 28, 1959, the lake level was lowered to an elevation of 6,400 feet and the lake contained 36,000 acre feet of water. This compared with a peak at elevation 6,453 feet and a maximum lake storage of 80,000 acre feet. The Army Engineers inspected this spillway in June 1960 and reported that no substantial structural changes had taken place during the winter.

Landslides and Steep Slopes

Other landslides, of various sizes and forms, occurred in the Madison Range at the time of the Hebgen earthquake. These movements ranged in character from sliding talus, soil mantle slips, and slow-moving mud flows, to displacement of bedrock down slope. Although landslides are a characteristic earthquake phenomenon, they are occurring and have occurred in many places without the shaking effects of faulting, and in steep mountain country they are often a prominent degradational process (fig. 7). Landslides are not new features in the landscape of the Madison River mountains. Old scars and deposits varying in age, as evidenced by the extent of modification caused by weathering and erosion, are to be seen throughout the range.

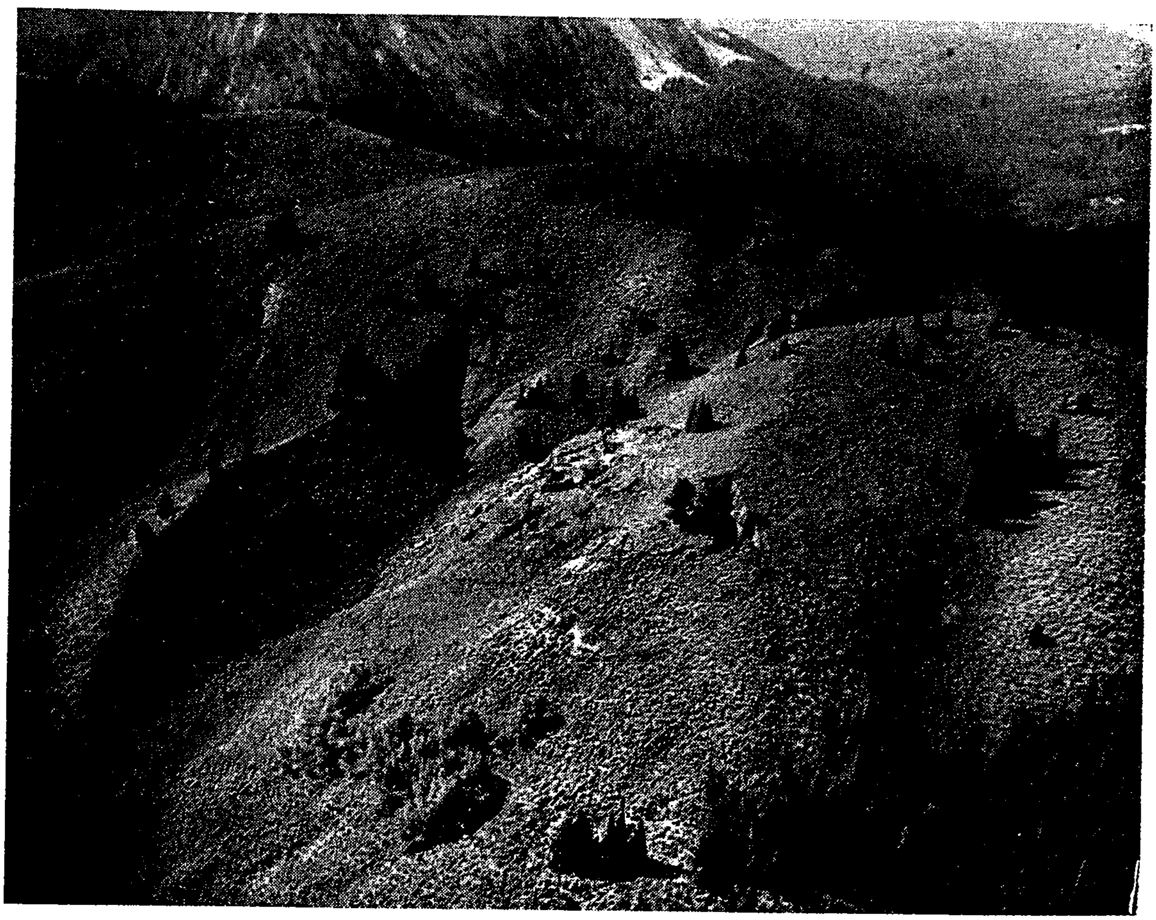

Figure 7. Small slips and mantle slump near Hebgen Lake which occurred at time of earthquake. Note old landslide topography in background. (U. S Forest Service photo.)

Steepness of slope, character of the mantle, and structure and composition of bedrock play important roles in causing landslides. Steep slopes are a common feature in the Rocky Mountain region, and many problems of forestry and grazing, and especially road building, are associated with such slopes. An examination of the origin of slopes is instructive. A common topographical feature apparent in the West—and it occurs in the Madison River Range—is a nonsymmetrical profile across east-west canyons. South-facing exposures are often less steep than the north ones; they have less soil and vegetation on them, and the evidence of rain wash and surface runoff erosion is more pronounced. These contrasts are no doubt due to differences in exposure to the sun. The south-facing slopes receive more energy per unit-area than the north-facing slopes; this gives rise to differences in local climate and, over the ages, to differences in relief, soil, and plant cover. Because weathering and erosion have been more active on the north side, alluvial deposits are deeper and more extensive on that side of the canyon, and the stream is often crowded over against the south wall, causing under-cutting of the north-facing slope. It is not uncommon for the profile on the south side to be convex upward, especially in the lower part of the slope, as contrasted with a generally concave profile on the north side.

Angle of slope of a mature topographic landscape is determined not only by stream erosion, the underlying rock and its structure, and accompanying weathering, but it is also due to processes involved in the formation of the soil mantle. For instance, it has been observed that in areas where raw talus slopes normally repose at an angle of 35 degrees, vegetated slopes underlain by fine-texteured soils with well-developed profiles may have angles of slope beyond 45 degrees—even at angles as great as 60 degrees. Soil formation on such slopes must have been accompanied by the stabilizing influence of plant cover, and it remains in place today by virtue of the binding and hydrologic functions that the vegetation performs. These slopes, because they are beyond the angle of repose, are in critical balance; and any disturbance of them through modification of the plant cover or undercutting by road building or by shaking of the earth's crust by earthquakes can result in displacement of the mantle through slippage or erosion. Earthquakes, steep mantle-covered slopes, and undercutting of banks by streams are features common to young geologic regions such as those found jn the Madison River Range and present difficult problems to the road locator and builder.

Problems of Road Location and Construction

All this brings into focus some roadbuilding problems now being faced. Earthquake damage to Highway 287 was so severe that traffic through the canyon cannot be resumed until sometime next year. At present, traffic from the northwest is routed over a semi-improved road from the mouth of the Madison Canyon over Raynolds Pass through the Henry Lake Valley, where it joins Highway 191. A new highway being constructed along that general route will bypass the Madison Canyon area, but it cannot avoid geologically recent faults. This new location closely parallels some of these faults and crosses over a fault zone between the Madison and Henry Lake Valleys.

The Bureau of Public Roads will rebuild the forest Highway 287 through Madison River Canyon, past Hebgen Lake and Quake Lake to Highway 191.

The main purpose of this road will be to service the important recreational areas of Madison River Canyon. Topographic changes wrought by the earthquake, such as the great slide, slide scarps along Hebgen Lake, Quake Lake covering parts of the old highway, and the pronounced fault scarps that prompted the Forest Service to designate this as a Geologic area, will greatly increase the number of tourist visits and will demand access by well-located and safe highways.

This will require new location for certain critical sections of the highway (figs. 8 and 9); it will present many difficult and costly problems in construction and maintenance that had not been associated with the original highway. The original road was built on the alluvial fill on the south side of the stream through the narrow part of the canyon and then on colluvial material past the lake to the junction with Highway 191. A new road must be built over the 400-foot slide and along steep and often rocky slopes of the south side of the canyon above the level of Quake Lake. Because the new highway must in some sections be constructed on the slopes above the bottom of the canyon, crossing side drainages will require grades and bridges that were not necessary in building the original highway. The new road will have to cross new fault scarps, and in at least two areas the ground between the mountain and the river has been so extensively disturbed by

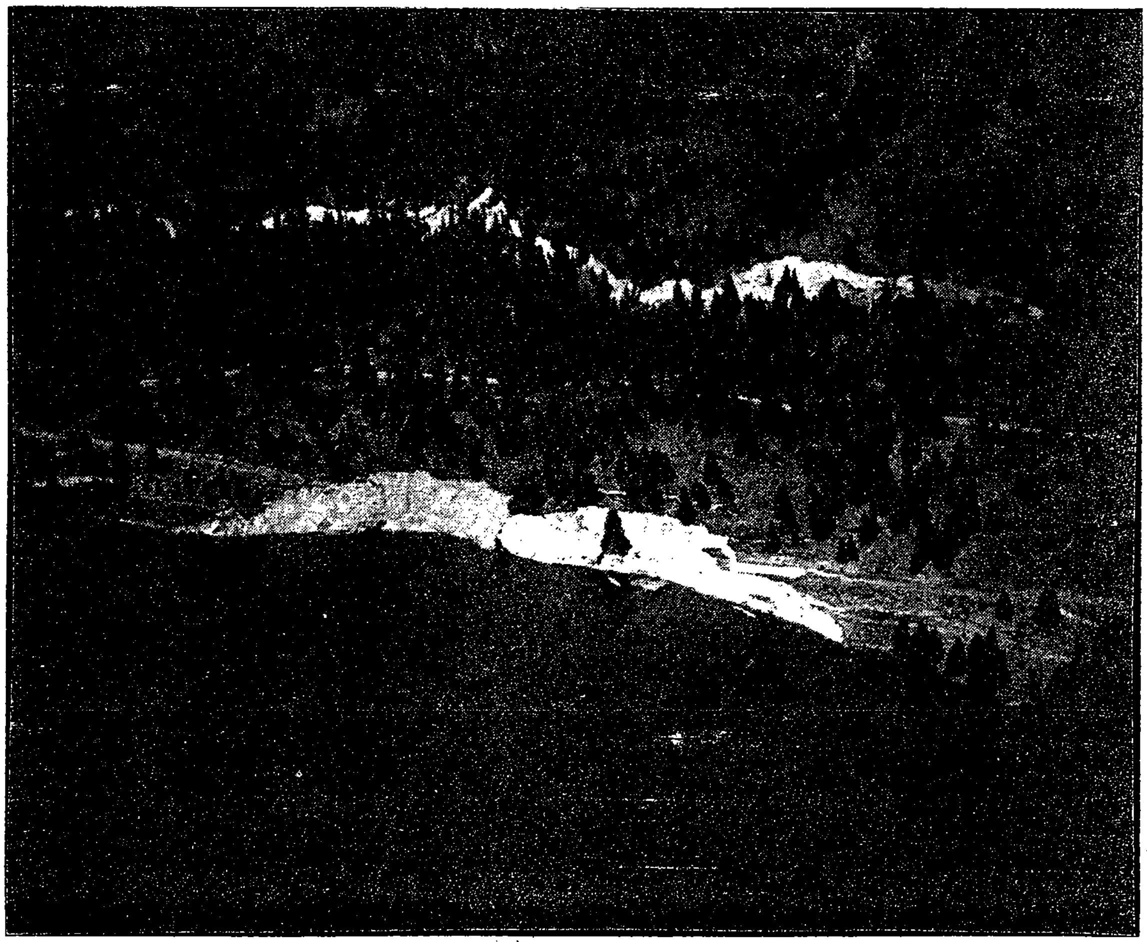

Figure 8. Section of highway slid into the lake. Up slope are two new scarps, one probably due to mantle slumping but another by faulting showing a displacement of 15 feet. The situation here represents a difficulty of relocation and construction facing the highway engineer. (U. S. Forest Service photo.)

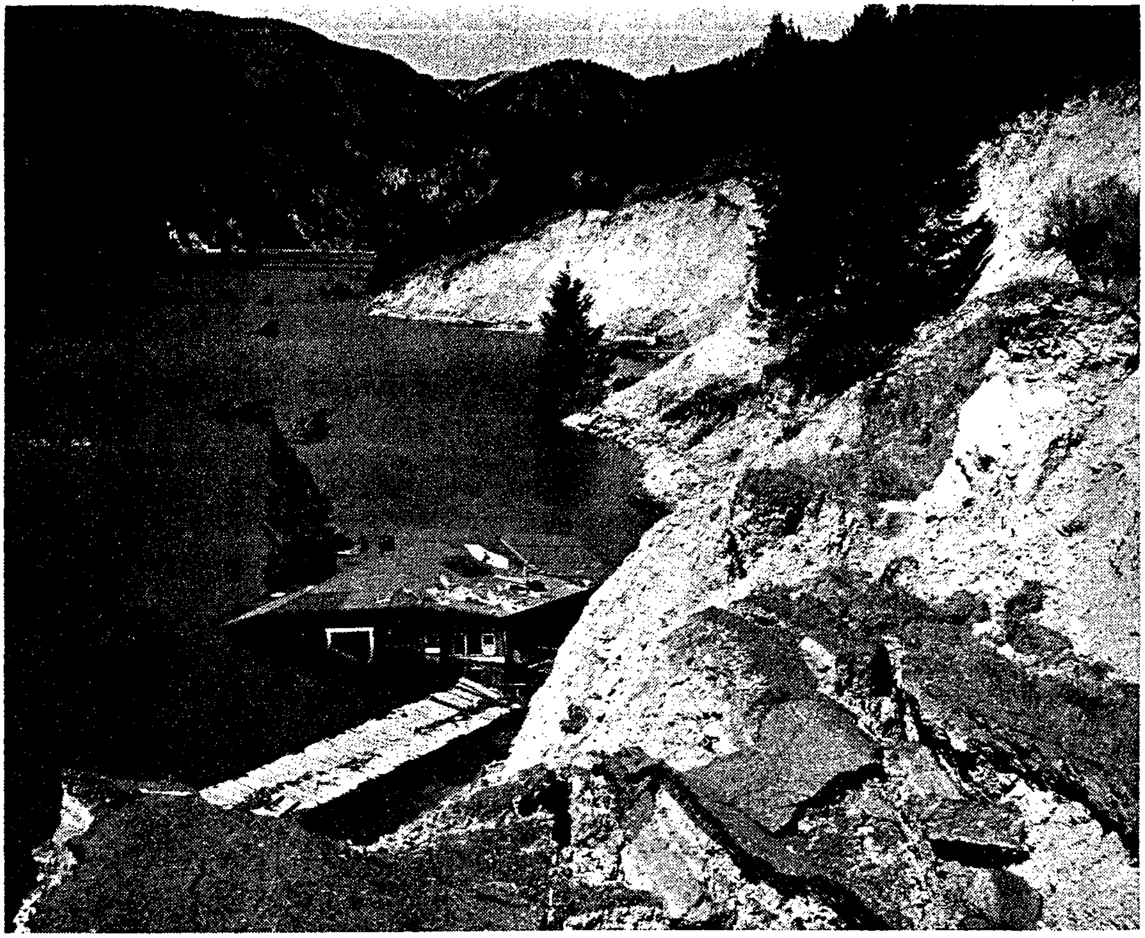

Figure 9. Same slide as shown in figure 8. The slide is 1, 200 feet long, 350 feet wide. One hundred ten thousand cubic yards of colluvial material slipped into the lake. Difficulty of highway relocation and construction apparent. Breaks with displacement up to 15 feet occur up slope from this scar. (U. S. Forest Service photo.)

a series of displacements that the area is at present unstable and may continue to be so for some time in the future.

Locating and constructing a highway past slides that carried parts of the old highway into the lake present special problems. Because of the steep slip scarps the stability of the material above the break is in question, and certainly to build across the new slide scar would require expensive and extensive stabilizing structures to support the highway. Problems of cuts and fills, of subsequent erosion and slumping, landslides, and drainage will confront the engineers and will require the utmost skill in road location, design, and construction.

Conclusion

In view of the fact that new Highway 287 and the new road will cross both new and old fault lines and will traverse an area considered diastrophically active, the question naturally rises, "When might an earthquake be expected in this area again?" By way of answer we can say only that the recent faults represent a release of stresses that probably had been accumulating over a long period of time. The 1959 earthquake was the manifestation of the release of stresses, and presumably it will require a long time for equal or similar stresses to build up again to the point where the earth's crust will again break causing faults and quakes. It is not unreasonable to expect the immediate future to be a period of relative quiescence. In any event, the mass of geologic, seismologic, and hydrographic information that has been derived and that remains to be derived from the Hebgen Lake earthquake may provide, in addition to enriching our understanding of earth processes, clues or techniques, and even principles, for locating highways, utility lines, etc. to minimize damage that might result from future earthquakes.

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GEOLOGICAL CONDITIONS COMPLICATING HIGHWAY AND RAILROAD RELOCATIONS IN THE NORTHWEST

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Introduction

The Corps of Engineers, U. S. Army is involved in the relocation of highways and railroads as a result of its civil works activities in construction of reservoirs for multiple purpose projects. The natural location for both railroads and highways is the river valleys in the rugged topography which characterizes much of the Northwest, and as a result, reservoir projects in the Northwest require the relocation of these facilities before reservoirs can be put into operation. In many places along the Columbia River and some portions of the Snake River there are railroads and highways on both sides of the river. For example, John Day Reservoir which is now under construction has railroads on both sides of the river totaling 136 miles and highways on both sides of the river totaling 81 miles in the reservoir area. For this one project alone, about 30,000,000 cubic yards of embankment and 73,000,000 cubic yards of excavation, including 35,000,000 cubic yards of hard rock, will be required. Relocations required by projects constructed and under construction since 1950 include over 400 miles of railroad and about 250 miles of highway at a total cost of about $400,000,000. This work is located in the states of Oregon, Washington, Idaho, Western Montana and a small portion of Western Wyoming as shown on Figure 1. The climate of the area is characterized by extremes. Annual precipitation ranges from less than 10 inches in the Columbia Plateau to well over 100 inches in some of the mountainous areas. Temperature in the Eastern portion of the area ranges from —40°F. to well above 100°F., while in the area west of the Cascades the range is generally between 35°F. and 80°F.

Geological Setting

As shown by the land forms map, Figure 1, the topography of the Northwest is characterized by rugged mountains, low relief plateaus such as the Columbia Plateau which ranges from elevation 1,000 feet at its western edge to about 4,000 feet near the eastern boundary. The Rocky Mountains which form the east boundary of the region range between 10,000 and 12,000 feet in elevation, and the Cascades near the western boundary of the region range between 6,000 and 8,000 feet with occasional peaks which exceed 11,000 feet. The Puget Trough forms a relatively flat valley between the Cascades and Coast Ranges. Bedrock of the area consists of sandstones, shales and limestones and their metamorphic equivalents in the Rocky Mountain areas; granites with some lava flows in the Idaho bathylith area; and great depths of lava flows in the Columbia Plateau area; volcanics in the Cascade and Coast Ranges, as well as shales and sandstones along the Pacific Coast. The Rocky Mountain area has been highly folded and faulted and the Cascades have been distorted by local intrusive bodies and regional faulting with a resulting complex system of fractures and faults. Deep and erratic weathering has occurred in the granites of the Idaho bathylith and

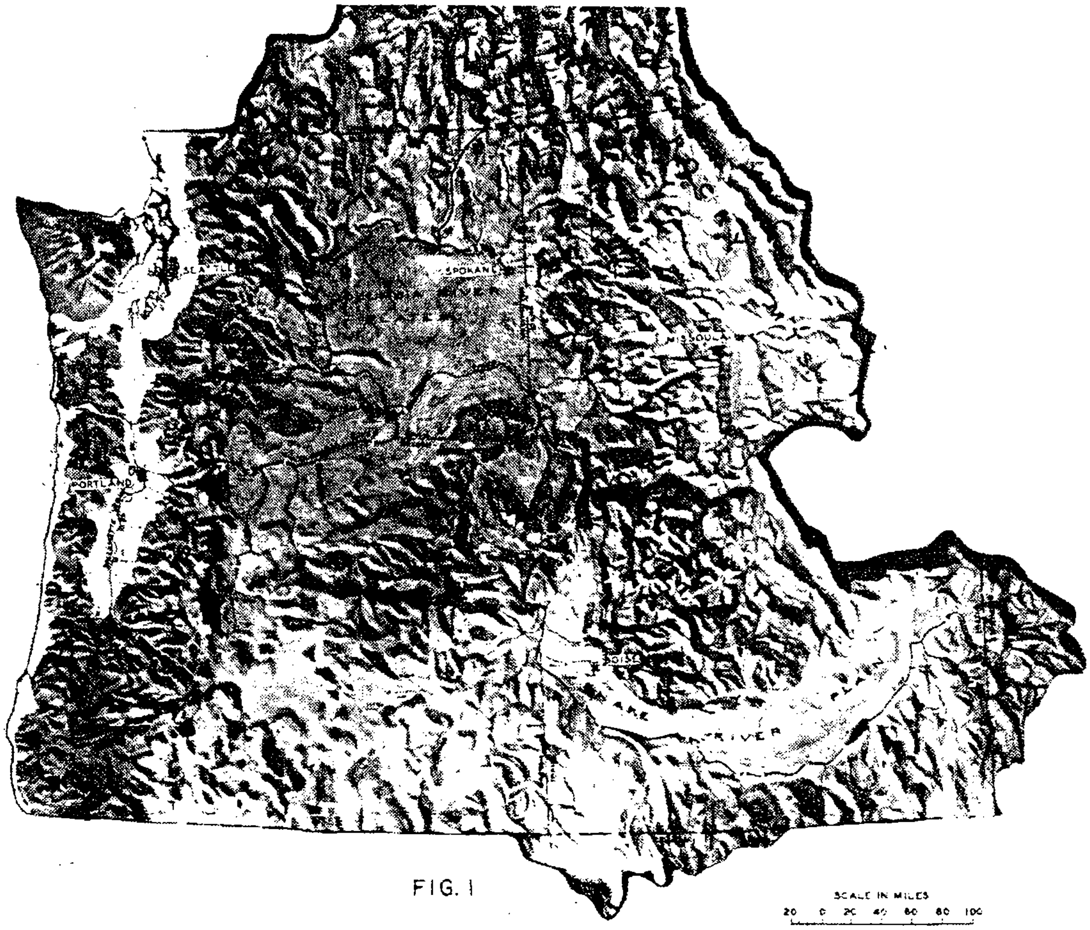


FIG. 1

Figure 1.

in the volcanics of the Cascade and Coast Ranges. Glacier action has profoundly altered the valleys of the Northern and Eastern sections and extensive glacial deposits remain in certain areas. Ancient glacial valleys now masked by glacial deposits or slide materials further complicate the geological picture.

Special Considerations in Relocating Highways and Railroads around Reservoirs

As stated previously, railroads and highways in a rugged country naturally follow the drainage systems as near river level as possible. The construction of a reservoir forces the location of railroad and highway grades near the maximum reservoir level. Constuction at these locations 100 to 600 feet above the river is complicated by the following:

a. The limiting grades and curvatures required by main line high-class railroads and the present-day highways. Although the limitations on highways are not as severe as on railroads, the requirements of the national defense highways and high-speed truck haul roads for logging operations make construction of both railroads and highways at the dictated locations difficult and expensive.

b. The topography at required elevations is usually characterized by scalloped contour lines with the spur ridges and valleys causing numerous and sharp reversals in direction. The slopes are steep and barely stable and at some locations continue to much higher elevations at these steep slopes so that benching into the slope may disturb the stability. In the Columbia River Gorge the alignment may traverse talus slopes which are as much as 1,000 feet high.

c. The depth of weathering is usually greater at this height above the river, the materials generally wet and of low strength especially in the western Cascade Mountains. Ancient and historically recent slides in these deeply weathered and weak materials cannot be bypassed.

d. Fills that extend into the resorvior pool must be founded on a sound foundation and be constructed with stable materials and slopes to preclude failure during fluctuations of the pool.

e. Difficulties of obtaining accurate geological information in geologically complex and heavily wooded areas with deep overburden such as in the Western Cascades.

f. The unfavorable attitude of joints, fractures and sometimes faults with respect to the excavation makes it very difficult to establish a stable backslope within a reasonable cost where the steep slope may continue more than 1,000 feet above the proposed grade.

g. Because of the traffic and the expense involved in any interruption of service either on highways or railroads, and at some locations the lack of alternate routes, the relocations must be designed and constructed in such a way that they can be placed in continuous operation once the old grade has been abandoned. No time is available for seasoning of the new cuts and fills as these are replacements of existing facilities and will be expected to carry a full load as soon as they are put into service.

Geological Problems

To illustrate the geological conditions which complicate the construction of railroad and highway relocations in reservoir areas, several specific problems will be discussed.

a. *Slope failure in decomposing granite.* Figure 2 is an idealized section

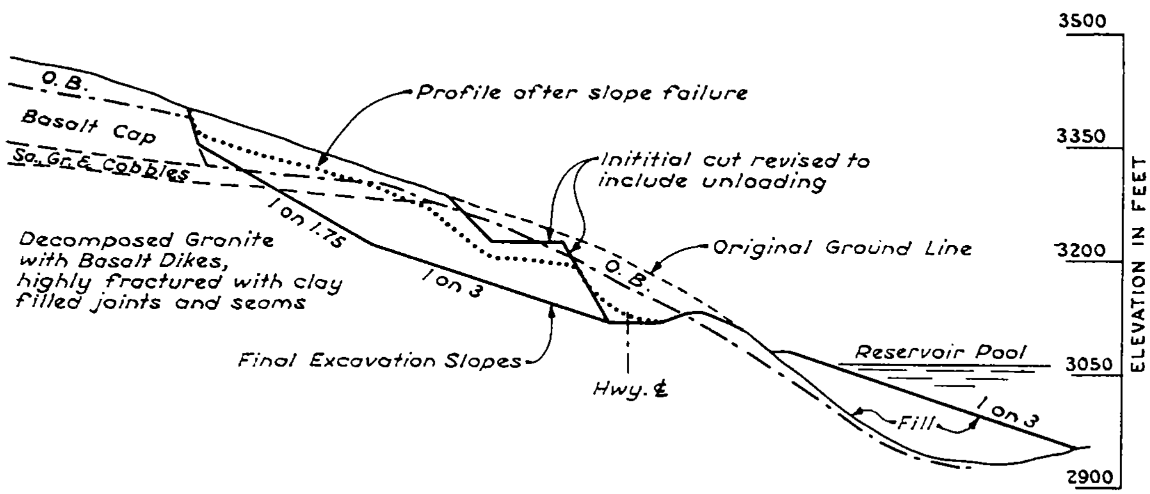


Figure 2. Slope failure in decomposed granite, Lucky Peak Project, Idaho State Highway No. 21.

of a road relocation in the Lucky Peak Reservoir near Boise, Idaho. The bedrock is granite, decomposed, highly fractured with clay filled joints and seams near the surface. The granite has been intruded with basalt dikes which are also altered, and the ridge at this location is capped with a

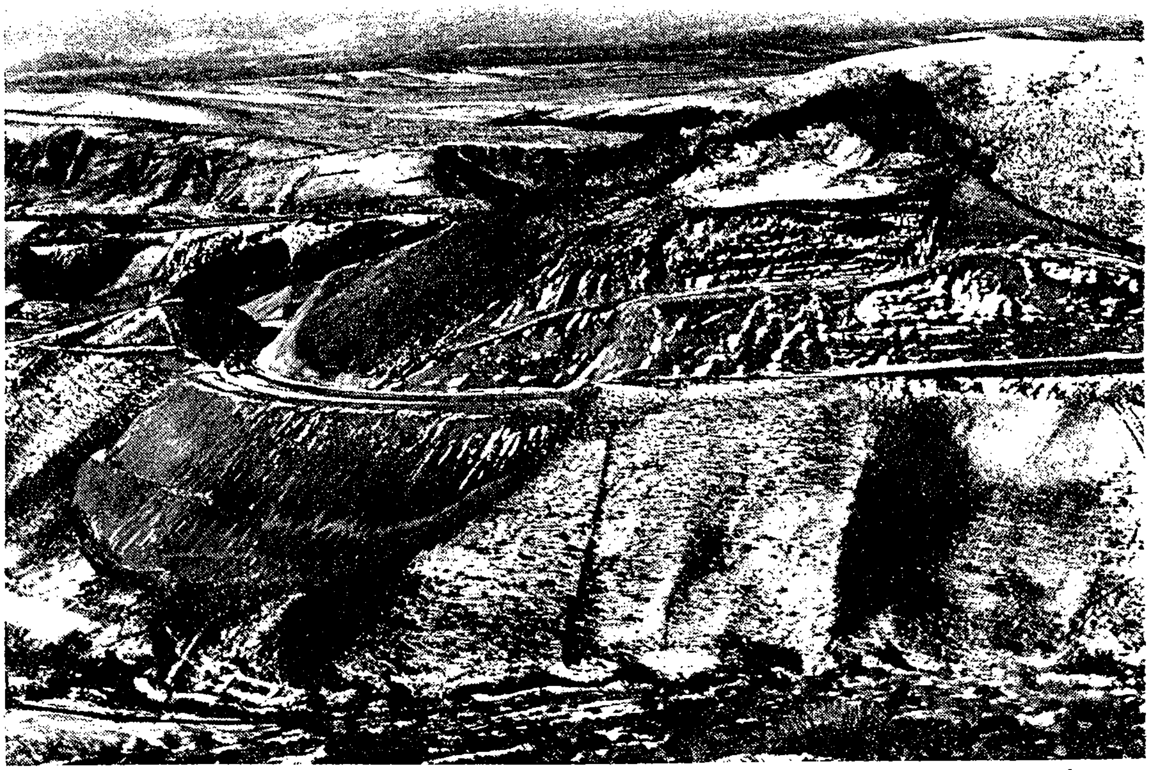


Figure 3. Slope failure in decomposed granite, Lucky Peak Reservior.

columnar basalt flow, with sand, gravel and cobbles occurring between the basalt flow and the granite. During construction failure occurred as shown by Figure 3. The displacement at the top of the slide was about 35 feet. The movement was arrested by removal of some of the slide material and forming a bench some 100 feet above the roadway grade. The slide dropped approximately 35 feet in the first year; the next six months it dropped about 2 feet; and then about .6 foot for the next six months. Because the movement had occurred above the resorvoir and the movement had practically stopped within two years, it was decided that no further work was required for correction before filling of the reservoir. Correction could be made after filling of the reservoir, if necessary. However, there was some concern about the possibility of the slide extending below the reservoir elevation, and a waste fill area was reshaped to stabilize the slope below pool elevation. Movement continued after filling of the reservoir and the excavation above the roadway grade was resloped as shown by the "final excavation slopes" of Figure 2 to preclude the possibility of future slides. This reservoir has been in operation for five years now and no futher difficulties have been experienced.

b. *Weak bedrock horizon in high rock cut.* The thick lava flow deposits along the Columbia and Snake rivers contain interbeds between some of the flows over limited areas. Weak horizons of bow breccia and cinder beds are sometimes encountered at unfavorable locations. When these horizons occur near roadway level below rock cuts, failure of the weak horizon undermines the otherwise stable basalt, with the result that large blocks of basalt may fall onto the roadway. There have been several instances in the past years where rockfalls from these high basalt cuts and bluffs have resulted in fatalities by rock falling on an automobile or falling in front of an automobile and causing a fatal accident. It is difficult to locate all of these weak horizons during the planning stage because they are usually buried under talus slopes. Subsurface explorations would locate some of the weak horizons but not all because they are sometimes lenticular and extend for only a short distance. When they are encountered during construction, correction consisting of unloading is accomplished by flattening the slope in the area of the weak horizon, establishing a bench above them and setting the slope back in the competent basalt as shown by Figure 4.

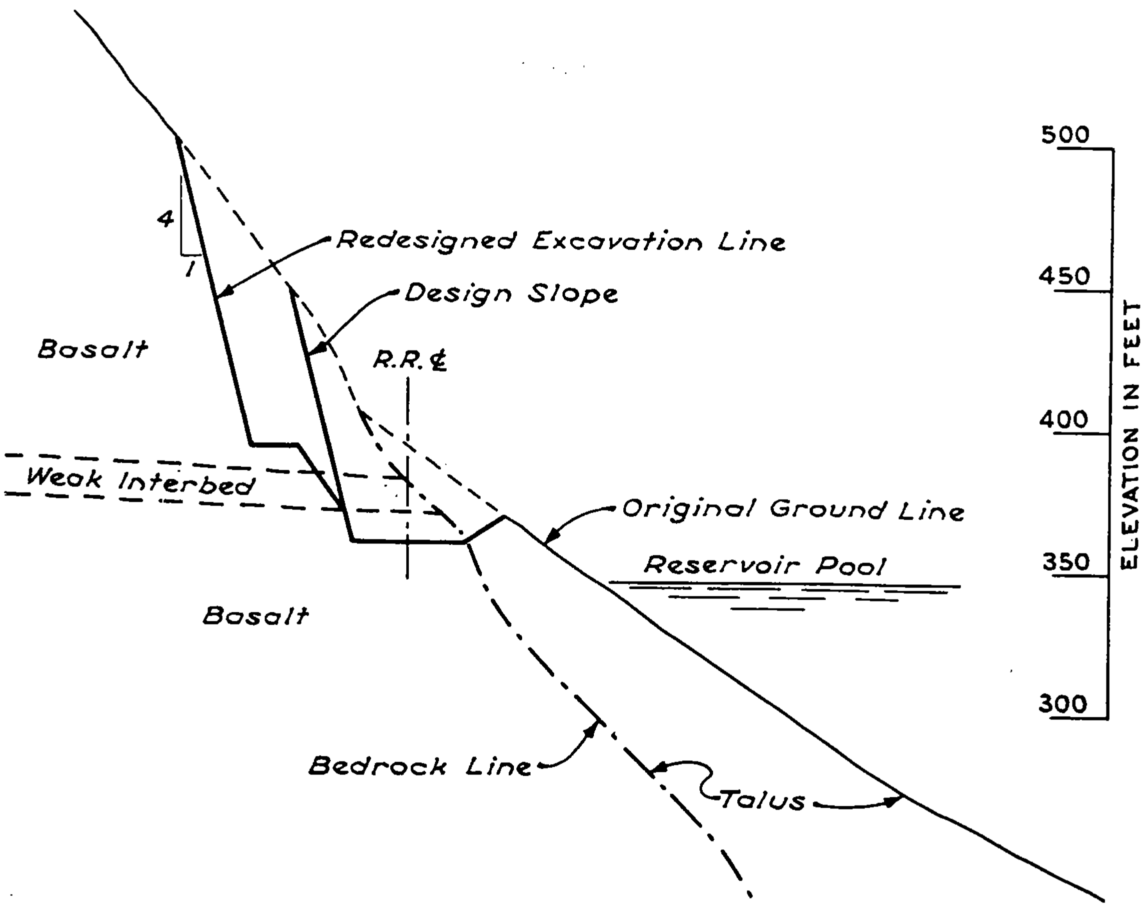


Figure 4. Typical section showing influence of weak horizon on slope stability.

c. *High talus slopes and rock bluffs.* Figure 5 is a photo of high talus slopes and rock bluffs in the John Day Reservoir along the Columbia River. The relief from the river to the top of the hill in this picture is about 1,000 feet. The area for relocation of the railroad and highway is very limited and will require an alignment in the talus slope. The talus slopes are of a heterogenous composition containing silts, blow sands, ash and fragments of basalt. Because of the serrated face of the bluff the top elevation of the talus slope varies considerably. In order to establish a stable and safe roadway in this location it is necessary to cut the slope in the talus flat enough to insure its stability and in addition provide berms and collection ditches to prevent spalling rock from the high bluffs from reaching the roadway.

d. *Unfavorable joint and fracture patterns.* Figure 6 is a section showing modifications of a slope above a log haul road and a railroad relocation in the Howard A. Hanson Reservoir. The initial excavation with a ¼ to 1

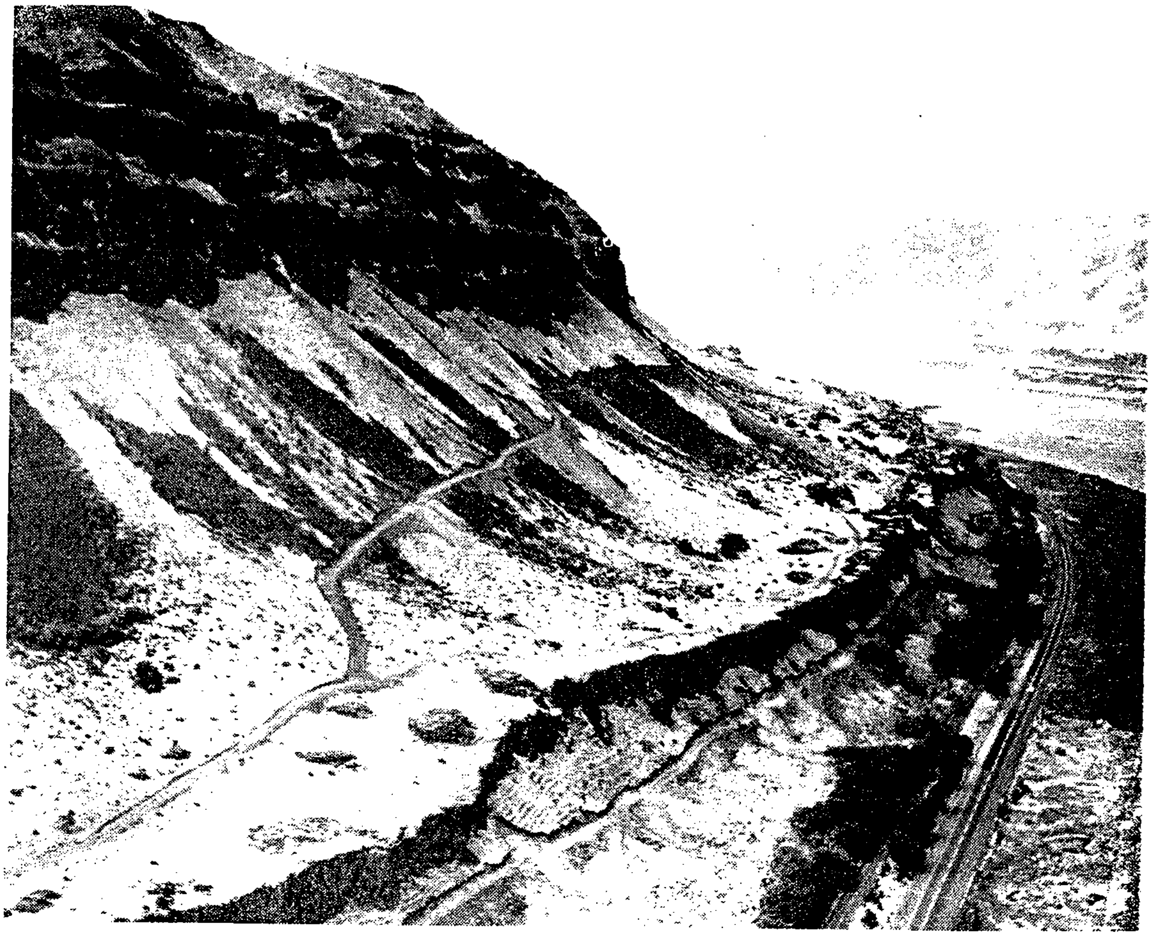


Figure 5. High talus slopes and rock bluffs, John Day Reservoir, Columbia River.

backslope is indicated in the triangle identified as '56. After original excavation in 1956 the slope was modified by additional excavation as indicated by the '57 and '58 portions. The rock involved was an andesite which was in some places highly fractured and jointed with some joints sloping towards the cut and joints intersecting in such a way that some of the segments appear to be at the point of failure. After completion of the modifications in 1958 and removal of all unsafe appearing rock the cut now appears to be stabilized. Figure 7 is a photograph of the essentially completed railroad and log haul road in this section. The section shown in Figure 6 was taken through the cleared area in the upper center of the picture.

e. *Slide in unconsolidated materials.* Many slides have been experienced in the overburden and highly weathered rock of the Western Cascades. Sometimes these slides occur in ancient slide areas. Evidences of previous failures are indicated by slickensided surfaces and fault planes filled with soft clayey gouge. The residual clays usually have many seams and fractures. Such a slide is illustrated by Figure 8, a profile of the Voss Slide in the Lookout Point Reservoir. The reservoir for this project was filled for the first time in 1955 to elevation 925. It was drawn down to elevation 825 that fall. Slight movements of the railroad grade were observed during the drawdowns of 1955 and 1956, and in the 1957 drawdown a horizontal movement of about 2.5 feet occurred. During the 1958 drawdown the slide accelerated rapidly until it was moving at a rate of about 3 inches a day. Total movement of a control hub riverword of the railroad was 26 feet

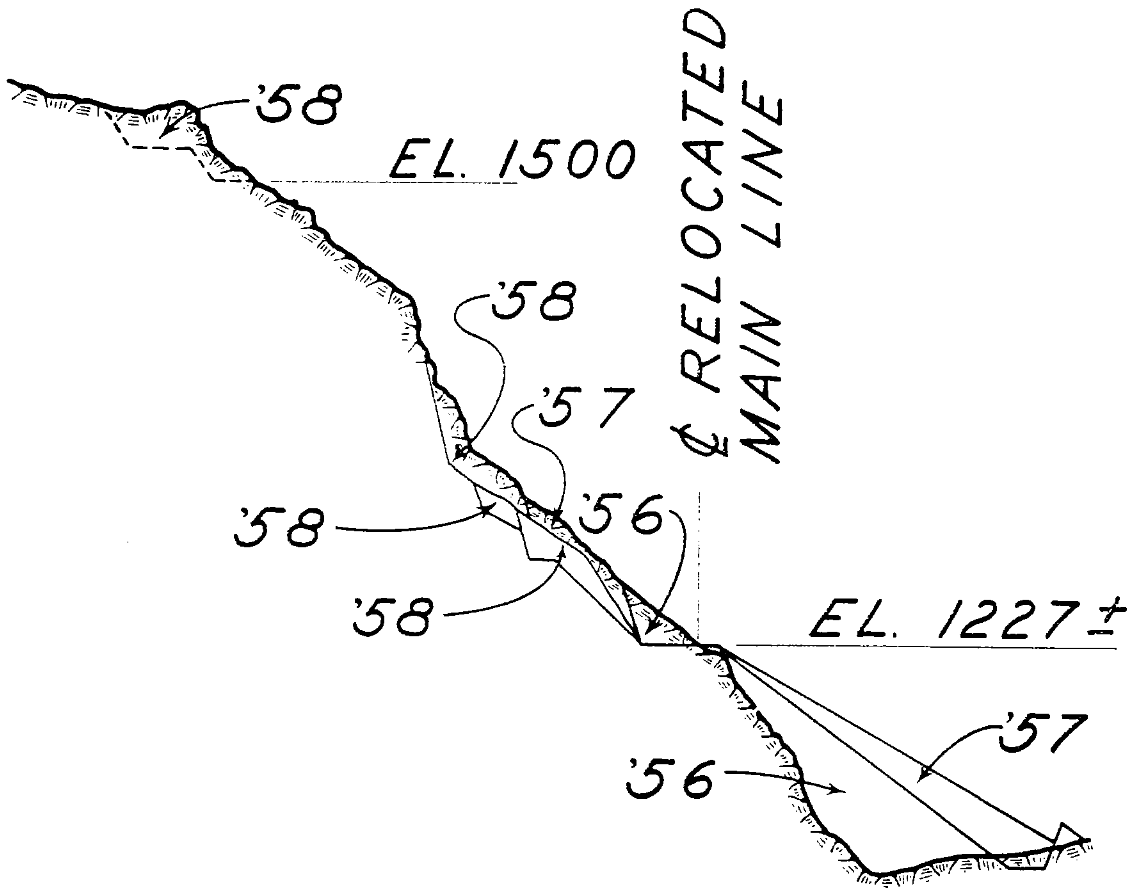


Figure 6. Section showing modifications of slope caused by faulting, unfavorable joint pattern, and intense fracturing of rock—Howard A. Hanson Reservoir.

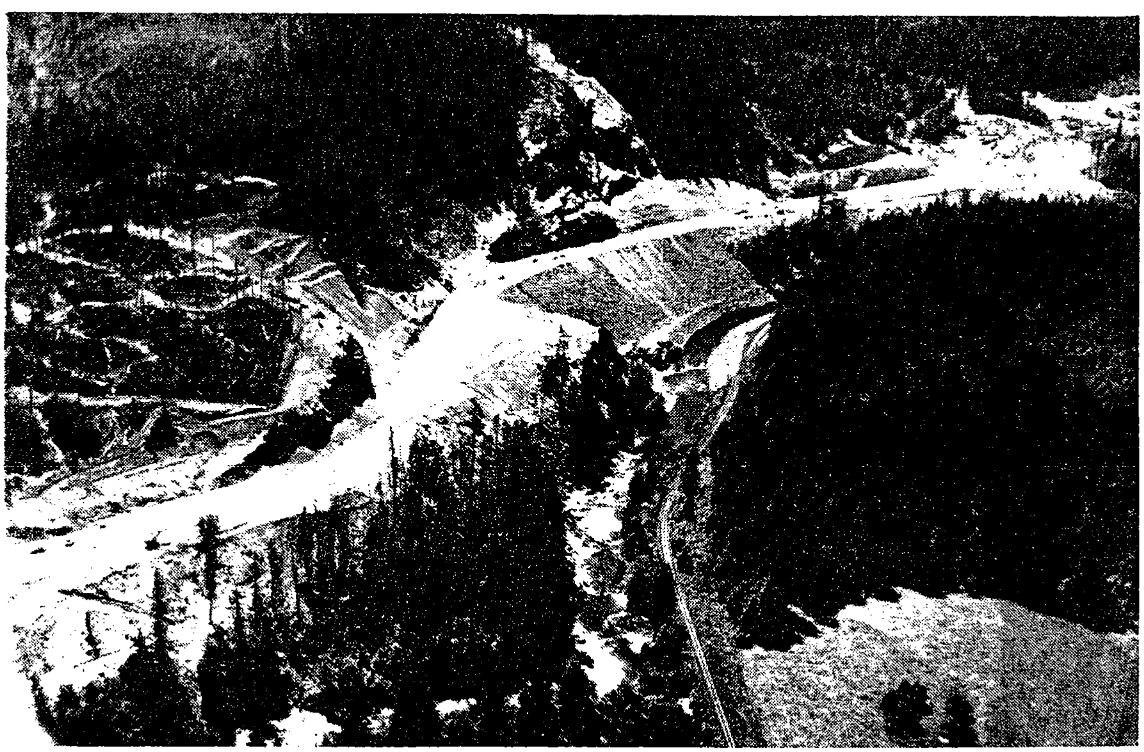
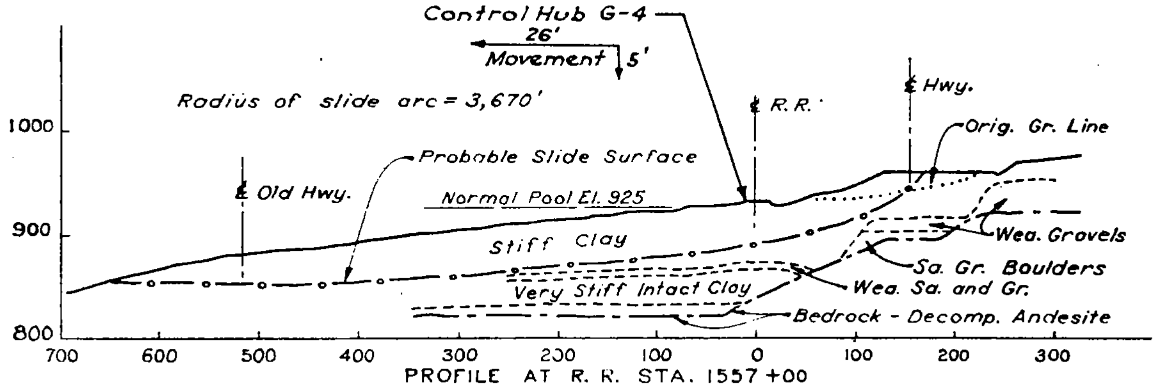


Figure 7. Photo shows difficult and rugged terrain through which railroad was constructed. Clearing in upper center of picture represents limits of excavation required to remove unstable rock to about 600 feet above the railroad grade.



Slide surface determined by depth of kinks in piezometer pipes and exposure of slip plane in hand dug test pit.

Figure 8. Profile of Voss Slide.

horizontally towards the reservoir and 5 feet vertically. At the highway, up the hill from the railroad, the displacement was 14 feet. The toe of the slide mass was approximately 75 feet below full pool elevation. You will note on the profile that the slide occurred in the stiff clay. This clay contained fractures and slickensided surfaces. The slide plane was filled with soft, fat clay gouge. Railroad traffic was maintained during the period of movement by operating under slow orders and daily realignment of the track. The slide problem was corrected by realigning both the highway and the railroad grade, up the hill where stable foundations could be obtained on the weathered gravels.

f. *Oversteepened cutslopes in materials of questionable stability.* Because of the steepness of the natural slopes and the vertical distance from

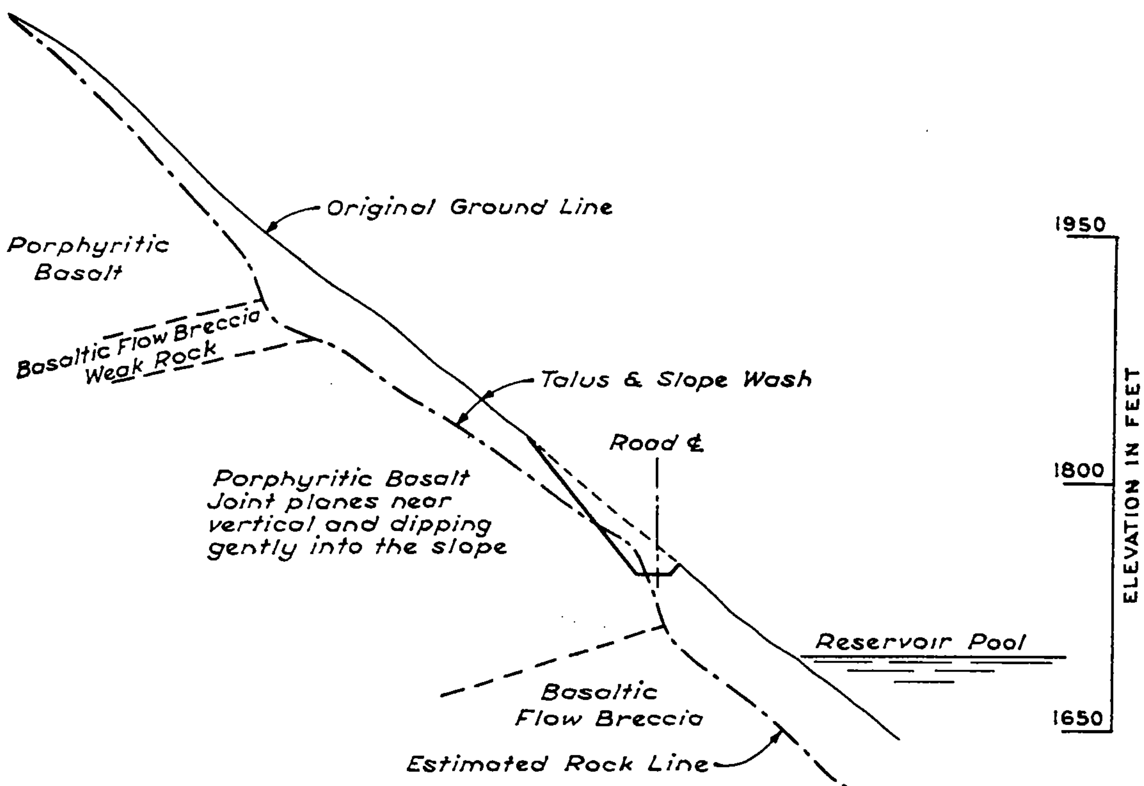


Figure 9. Typical section of roadway cut into steep slope and unconsolidated materials.

the reservoir to the tops of the hills it is sometimes impracticable to construct back slopes on what normally would be considered a conservative slope for the materials involved. Figure 9 illustrates a typical section of this situation. The natural slope is approximately that which would be required for a stable slope in the materials involved. Benching a roadway into this slope some 500 feet above the river and possibly 300 to 1,000 feet below the top of the hill requires an over-steepening of the back slope unless excavation starts at the top of the hill and parallels the natural slope all the way to the roadway grade. This would involve the removal of a tremendous volume of materials. In order to keep the excavation quantities reasonable, a slope is selected which will catch on the natural slope within a reasonable distance and will provide reasonable safety against major slides. This slope is generally the equivalent of the steepest natural slopes

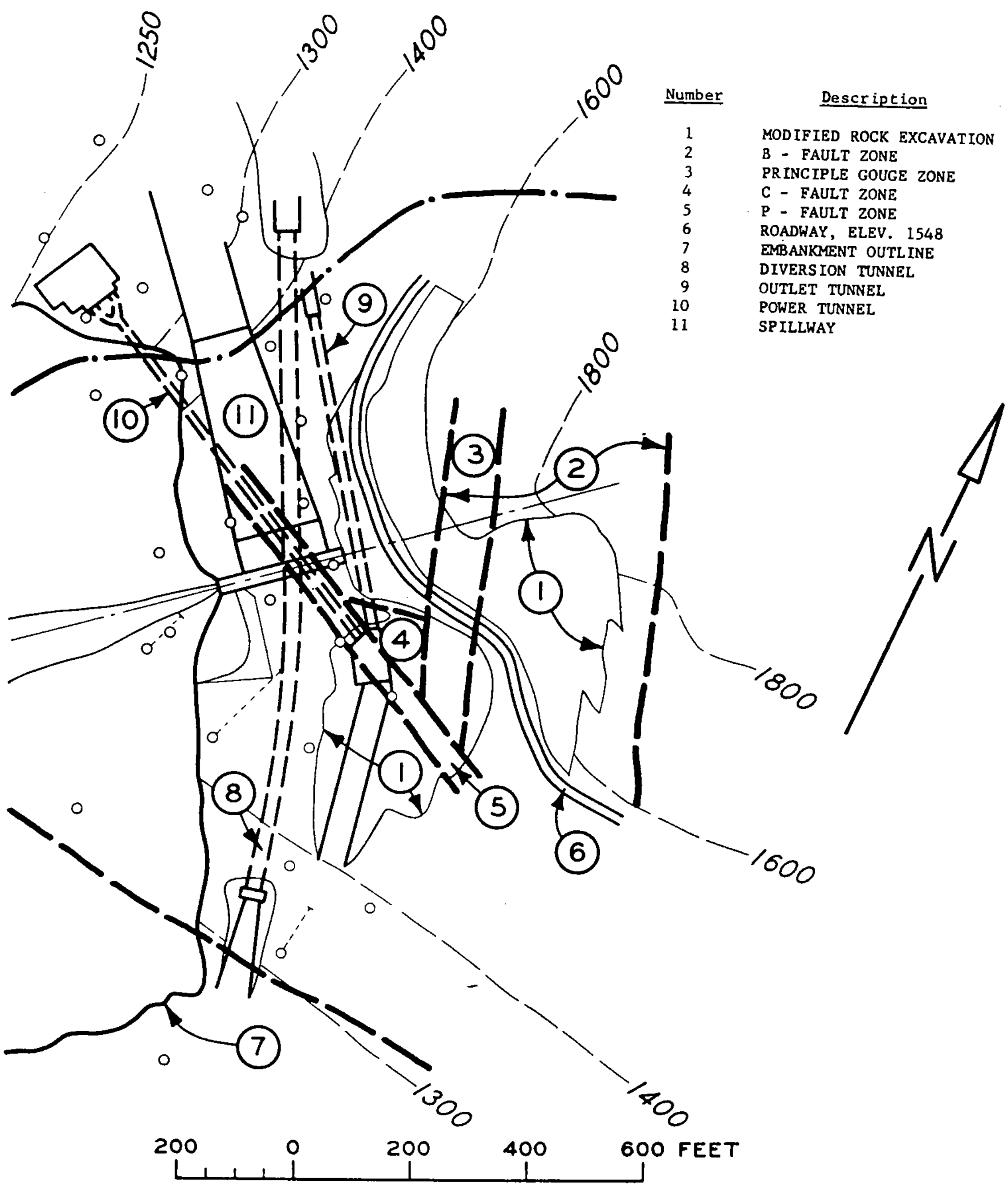


Figure 10. Plan of Hills Creek Dam, right abutment.

found in the area and is a compromise between a slope designed conservatively to preclude sliding and a slope which would be completely instable. A calculated risk is being accepted in this type of design and some slides may occur. However, the slope will eventually stabilize itself and only minor maintenance will be required to keep the road in operating condition. This practice can be followed only on the secondary roads such as those used for access to timber lands.

g. *Faulting.* Figures 10 and 11 illustrate the geological structure of the right abutment at Hills Creek Dam. The natural ground surface is shown by the upper dashed line on Figure 11 and the originally designed slopes are shown by the dashed lines identified by the number 8. The roadway level is at elevation 1548 and excavation was required to some 160 feet below the roadway for construction of an intake channel and intakes for the outlet works and power penstock of the Hills Creek project. The faulting had disturbed the rock over a wide zone with gouge zones and fault planes at an unfavorable dip to the design excavation slopes. Soon after excavation was initiated and before the roadway elevation was reached, failures along the fault planes and gouge zones indicated the necessity for a modification of the slopes where these zones intersected the cut slopes. Several attempts were made to stabilize the slopes by a slight flattening of the slopes in the fault zone but after excavation had progressed down below the roadway level into the intake channel area major slides occurred and it was determined that a complete redesign of the slopes must be accomplished if a stable excavation was to be obtained. Not only were the slopes of the

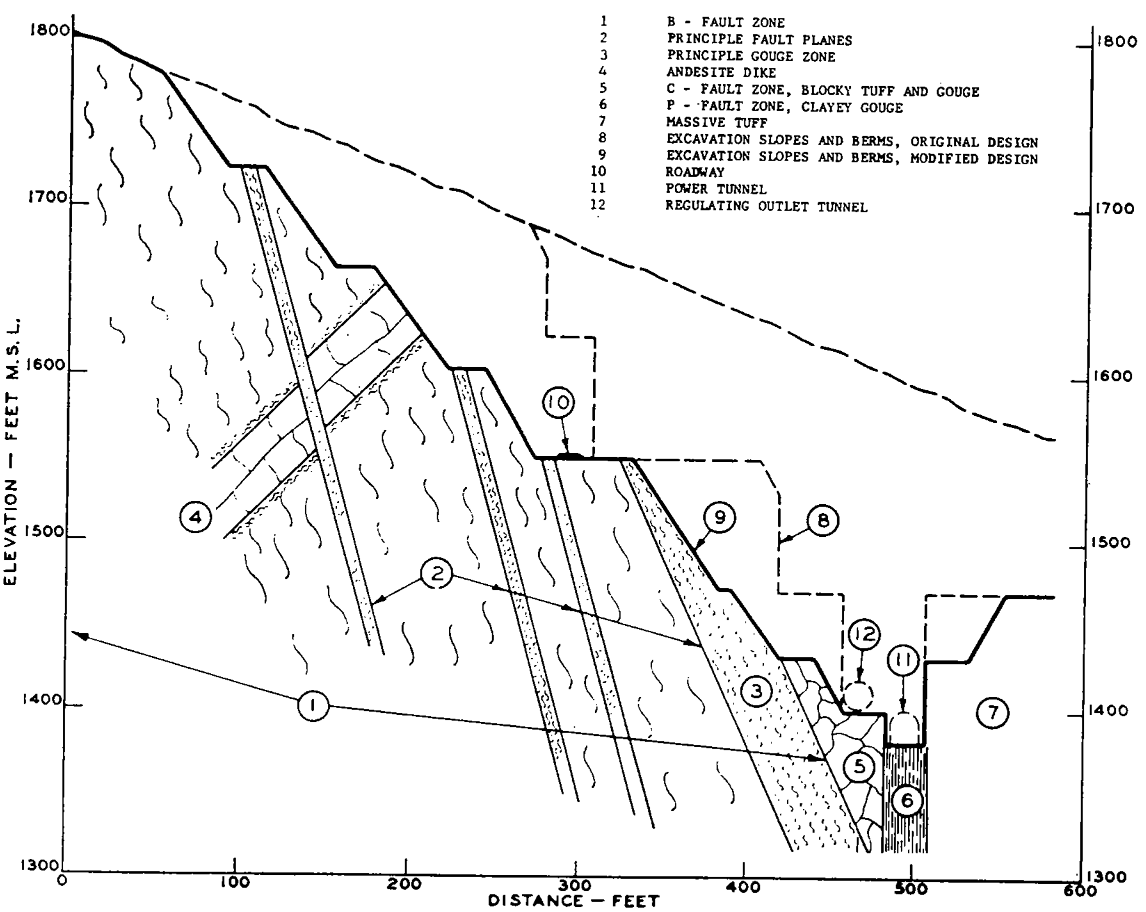


Figure 11. Section through right abutment cut, Hills Creek Dam.

of the proposed excavation slopes, but the P fault zone which was a wide zone of clayey gouge with an almost vertical attitude intersected the B fault at roadway level in the very highest portion of the cut. Thus a triangle of material in this area was completely separated from the mass of the rock hill and when support was removed as the excavation progressed additional slides continued. Finally the entire excavation was modified, the road alignment changed and stable slopes obtained as shown by the line identified by the number 9 on Figure 11. Figure 12 is a photo showing the completed excavation and the relationship of the faults to the cut slopes. The area identified by number 3 is a massive rock mass which was excavated on the originally proposed slopes and is stable. The relationship of the B fault zone and the P fault zone is indicated by the outlines numbered 1 and 2.

Conclusions

As brought out by the illustrations, the geological problems are complicated by the fact that the relocations for railroads and highways in connection with reservoir projects are seriously restricted in location. In certain portions of the Northwest geological conditions are such that they can be relatively accurately predicted and with nominal exploraions and proper interpretation fairly accurate designs can be prepared. In other areas such as the Idaho bathylith where the granites have been intruded, faulted and deeply weathered, it is very difficult to predict in detail all the problems that will be encountered during construction. The volcanic area of the Western Cascades are especially complicated, geologically, and it is

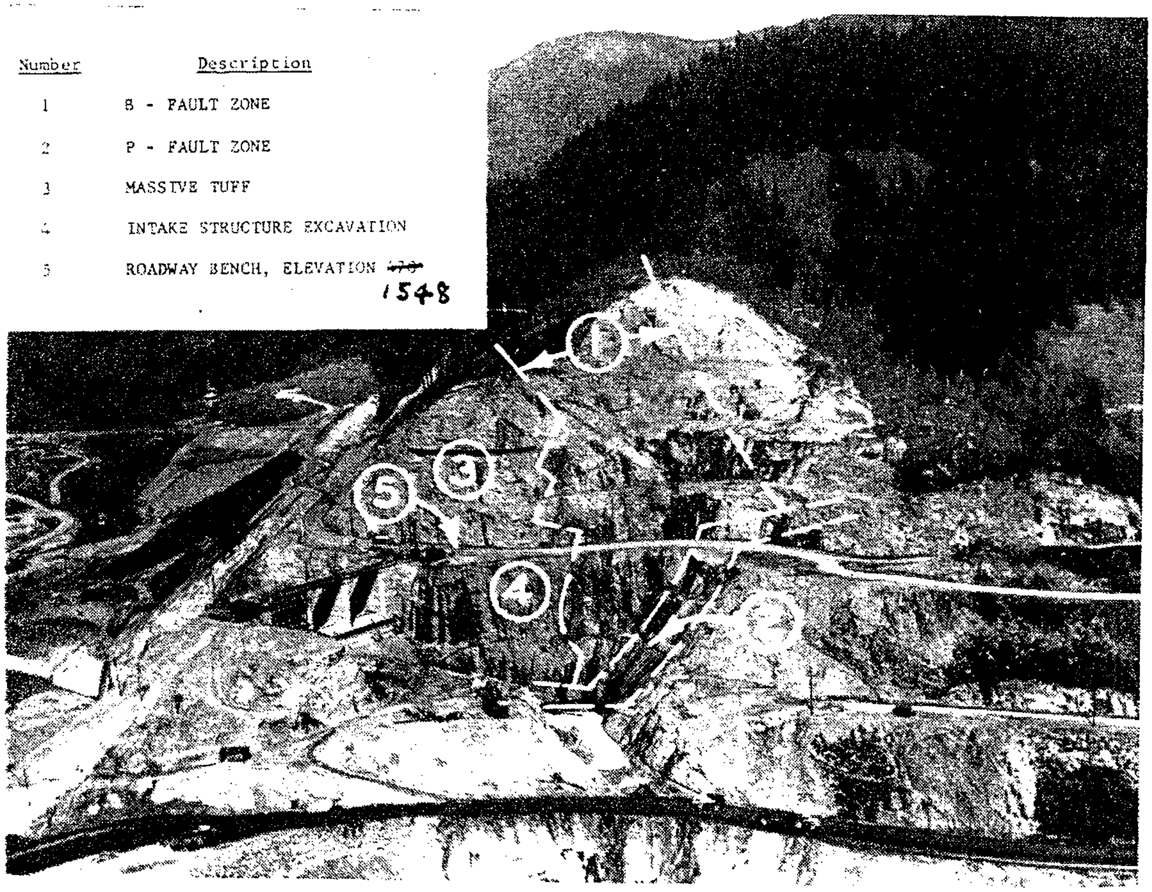


Figure 12. Completed excavation showing relationship of faults and cut slopes.

only the inexperienced geologist who would make recommendations for design in this area without detailed geological reconnaissance and explorations. Deep residual volcanic soils, highly faulted and fractured, pyroclastic and flow rock and modification of the bedrock lines by glacial action make geological interpretation especially difficult. The possibilities for completely accurate geological interpretations in such areas as these are not favorable. However, with experience and proper attention to detail, accuracy is greatly improved. Because of the nature of the geological conditions and the surprises that are always being uncovered by construction, continued use of geology is required during the construction period. The main objective in the geological phase of the planning and preparation of construction plans and specifications is to be able to recognize the possible difficulties that will be encountered during construction, make provisions for these in the contract so modifications can be made during the construction period without delaying the completion of the reservoir project or increasing the cost unreasonably.

OBSERVATIONS ON SUBSURFACE EXPLORATION USING DIRECT PROCEDURES AND GEOPHYSICAL TECHNIQUES

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Any means available for obtaining the quality, quantity and type of subsurface information required should be used in providing design data adequate for the efficient, economical and safe construction of a proposed structure. The geologist and engineer must be ready and willing to make use of as few or as many of the exploration methods available to him as is necessary to obtain a complete record of existing subsurface conditions. These will include geophysical tests as well as the more direct explorations by borings, test pits, etc. In some instances it may be possible to obtain the required information from the results of the geophysical tests without further exploration by borings. It is good policy, however, to make the geophysical tests first and supplement them with borings when the nature of the problem and character of the information sought warrants a more detailed analysis of subsurface conditions. The over-all achievement of exploration tools should be reviewed without condemning or eulogizing a particular procedure for a single failure or success. Further and more regular use of exploration methods, such as the geophysical tests, may result in greater perfection and an unforseen efficiency in analyzing the results obtained in future subsurface studies.

Subsurface Exploration Methods

The divining rod, willow stick, etc., have been mentioned at times in connection with locating water and other natural resources. Without endorsing such procedures, it can be stated that water coursing through a porous medium can produce measurable electric potentials. The reverse of this proven phenomenon is embodied in the drainage of some types of soil by electrical osmosis whereby an applied electric potential causes water to flow through the soil toward a drainage zone. It is possible that such "natural potentials" do occur from some such phenomena at times and likely have a resolving power in disclosing the conditions producing them when they are measured in some fashion and analyzed. This possibility will warrant further consideration and discussion.

Since most geologists and engineers are quite familiar with the various direct exploration procedures used in this country, no attempt will be made to discuss them individually. Also, much has been published concerning the various geophysical techniques used in shallow explorations with regard to details of testing and theoretical background.[2] For this reason, no detailed discussion of these methods has been prepared.

The direct methods of exploring the subsurface used for the deeper explorations include hand or machine-dug pits, hand auger borings, rod soundings, wash borings, power auger borings and power drilling. Shallower

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[2] See the list of references presented at the end of this paper.

tests for density and moisture measurements and other near-surface studies are a part of the subsurface exploration picture, as well.

Perhaps less thoroughly understood are the very practical uses of geophysical methods for various subsurface studies. The procedures involved include the magnetic, gravitational, seismic and electrical methods with some nuclear devices being tested for use in very shallow subsurface engineering studies. Also, a certain amount of information on probable subsurface conditions may be taken from aerial photographs which when combined with some direct control tests may constitute useful design data.

The geophysical methods of exploring the subsurface offer faster, more economical means of obtaining subsurface information than is generally possible when using any of the direct procedures referred to above. It has been established that most soils and rocks possess properties such as density, magnetic attraction, elasticity or electrical resistance which may be measured and used in a study to determine the character and position of the materials involved. Some rocks, by reason of their greater density, produce extraordinary gravitational forces in excess of that of surrounding rocks or materials of lesser density. A grivimeter survey would locate such dense formations. Igneous rocks are generally more magnetic than sedimentary rocks. Thus the presence of an igneous dike would be disclosed by a survey with a magnetometer. However, because of the difficulty of interpretation of the test results in terms of shallow engineering needs, neither the gravimeter nor the magnetometer has proved to be generally useful in shallow explorations.

The refraction seismic test, making use of the elasticity and density of the subsurface formations, measures the velocity of seismic waves which are produced by small explosions at shallow depths or by hammer blows made at the ground surface. Hard rock formations will have wave velocities some ten times the velocities found in unconsolidated soils. It is relatively easy to locate the rock formations beneath such unconsolidated materials. This test is very useful for some subsurface studies. These include the location of solid rock for structure foundations, classifying materials for excavation and slope design purposes, evaluation of quarry sites and determining the rock line above the invert of a proposed tunnel. More recently this test has received much publicity for use in predicting the "rippability" of the softer rocks and weathered materials,[3] enabling those concerned to plan for less expensive excavation operations in some instances.

The earth-resistivity test makes use of the electrolytic properties of the soil and rock formations in the subsurface. Soft, plastic clays, marls, mucks, etc., usually possess relatively low resistance to the passage of an electric current. The more granular soils, sands, gravels and solid rocks usually possess a much higher resistance to current flow. By proper spacing of the four electrodes used in the test, normally in a straight line on the ground surface, it is possible to measure the resistivity of the subsurface materials to successively greater depths and obtain much valuable information regarding their character and the depth at which the various layers occur. However, since many factors may affect the electrical resistance of soils and rocks, calibration or trial tests made over known materials in a given area are essential to good results.

The electrical test, in addition to being effective for obtaining informa-

[3] See reference No. 22 of the list of references given at the end of this paper.

tion concerning foundation conditions at structure sites, is equally good for use in tunnel investigations, slope design studies, quarry site investigations as well as locating materials of construction, testing depth of swampy materials and investigations of landslide conditions. "Rippability" can be established also in areas where exposed road cuts permit calibration tests to be made over material susceptible to ripping. Many grading projects have been investigated by the Bureau for which earth slopes were recommended based on subsurface data obtained by earth-resistivity tests. The "rippability" of the materials involved was well demonstrated by subsequent construction work. Where older road cuts are not available for comparative tests, a single hand-augered hole or a shallow test pit is often sufficient confirmation for a large amount of test data showing earth conditions or "rippable" material.

The detail and nature of the information required for a shallow hole for a utility pole will necessarily differ from that associated with a deeply dug well or a pile foundation problem. Thus the decision to use rod soundings in one area may be as sound for the purpose involved as a diamond drill hole would be in another area involving a much more complicated subsurface problem. At times the only method of exploration immediately available is arbitrarily used regardless of its suitability for the conditions involved. Cost estimates may determine selection of a method without due regard for the accuracy of the information obtained and used. In this regard, it is of particular interest that the concept of cost, particularly the unit cost per foot of auger borings, drill holes, test pits, etc., seldom is mentioned by those preparing technical discussions of explorations made on a given project. Casual mention is made of "100,000 linear feet of borings," perhaps, with no further discussion of unit cost.

In an attempt to avoid such oversight in the current discussion, an effort was made to collect data to show a comparison of costs for various direct exploration methods commonly used. Table 1 presents the results of this rather limited survey. Many factors enter into the overall costs of various exploration projects. The need for dry sampling in augered holes raises the cost for a particular hole. Obtaining undisturbed samples will increase the cost also. Many boring contracts employ "lump sum" charges to defray costs of moving into and out of a work area. Perhaps such factors as these deter individual reporting of unit cost data. Also, some boring contracts are made a part of an overall contract bid for a given structure with no assurance that the quoted unit prices for borings reflect costs entirely concerned with such work. Yet, as new and improved exploration procedures are developed, comparative cost figures assume an increasingly valuable role in a decision reached to employ one procedure or another. Reference to table 1 shows that power auger boring costs ranged from $1.42 to $6.28 per linear foot. Similar costs for core drilling ranged from $2.00 to $11.00. These have involved many of the extra cost items and special conditions referred to above. Perhaps average costs of $3.50 to $4.00 per foot for augered holes and $5.00 to $6.00 for drilled holes could be assumed when considering other procedures.

If, under some favorable geologic conditions, certain subsurface information ordinarily obtained by borings or from drilled holes could be obtained by geophysical tests at two to ten percent of the cost of the former, some consideration should be given to use of geophysics. If these rapidly made

Location	Year	Total cost of project	Total footage involved	Samples taken	Cost per linear foot by various exploration methods: Auger borings, Hand	Auger borings, Power	Wash borings	Core drill	Test pit	Remarks
					Dollars per linear foot					
Harvard 1/	1940	Unknown	Unknown	No sampling	0.75 to 2.00		0.50 to 1.25	3.50 to 35.00	10.00 - up	Core holes 1-1/8 to 36 inches
Ohio Turnpike	1951	$441,785	60,405 earth 8,812 rock	Dry samples, undisturbed and Shelby tube 5/		6.28 3½ in. casing 5.94 2½ in. casing 5.95 mud drilling		7.30		Core holes 2¼ inches
Building cost data (P.B.A.) 2/	1953	Unknown	Unknown	Dry samples when material changes occur		4.25 0 to 50 feet 4.75 50 to 100 feet		11.00		
Woodrow Wilson Bridge (Potomac River)	1955	20,194 3/	Unknown	Split spoon at 5-ft. intervals Undisturbed				3.45 to 5.00 4.65 to 5.95		On land On water
Basic Soils Engineering, p.451 (by B.K. Hough)	1957	Unknown	Unknown	Split spoon at 5-ft. intervals			2.50 to 9.00 6/	4.00 to 12.00		Core holes 1-3/8 inches
Oregon	1958	Unknown	Unknown	No sampling		2.00		2.00		Referred to as "drill and auger borings".
Maryland	1959	Unknown	Unknown	No sampling		4.50		6.00		Contract items, BPR
New York	1960	34,100	5,200 earth 700 rock	Dry samples		3.50 3.50		7.00 7.00		On land - lump sum $150 to $800.00 On water - for barge $250 to $1500.00
West Virginia	1960	Unknown	Unknown	No sampling		3.84				No lump sum charge
North Carolina	1960	12,442	5,460 earth and rock	Dry samples		1.42	2.82	2.82		3110 ft. cased, 2350 ft. uncased
Mississippi	1960	16,857 4/	4,474 earth and rock	No sampling		2.45		4.00		
Tennessee River Bridge (Natchez Trace Parkway)	1960	Unknown	3,500 rock	No sampling				10.48 7/		Bid submitted as part of overall bid for structure

1/ "Exploration of Soil Conditions and Sampling Operations" by H. A. Mohr, 2nd Ed. Rev. 1940. Harvard Univ. Grad. School of Eng., Soil Mechanics Series No. 4

2/ Public Buildings Administration.

3/ Including $2785.00 for taking 207 undisturbed samples.

4/ Includes $5.00 each for 44 rock core units and a $350.00 lump sum for moving.

5/ Undisturbed samples $12.58 each, Shelby tube samples $14.08 each.

6/ Includes penetration resistance records.

7/ For coring rock - 20 bids - minimum unit cost $6.50, maximum unit cost $15.00.

Table 1. Comparison of unit cost for various direct exploration methods.

valuable subsurface deposits not even attempted by direct procedures in some areas because of cost or unfavorable geologic conditions, their use is again warranted. Both of these situations have been encountered in the work done by the Bureau.

Slope Design

Although power auger borings, for the depths that may be attained in a given formation, can be used to advantage in some slope design studies, their use is not too practical in rough country or heavily wooded areas. Such borings are limited to reaching the first rather hard, dense layer and may produce poor results when attempted in talus material or other bouldery formations. In areas such as southwest Missouri where relatively thin but hard layers of chert may exist throughout the limits of a proposed cut area, auger borings can be inconclusive. These thin layers of chert produce refusal depths for the auger borings and require core borings to prove the character of the material to the full depth of the cut. This increases both time and costs for the explorations made. Figure 1 shows two curves for resistivity tests made in Jasper County, Missouri, in 1957. In the graphs shown, as well as in all subsequent graphs involving resistivity-depth tests, the relation of resistivity to depth is shown by the crosses and dashed-line plotted to the "individual value" scale. The left-hand graph shows a change to solid rock at a depth closely approximating that found by the power auger. The boring record is shown along the bottom of the graph. Evidence of solid rock rather than a boulder is found in the continuing trend towards the higher values of resistivity. Briefly, this trend towards higher resistivity suggests a formation change near the depth where the trend first appears. The writer uses the cumulative curve analysis[4] to obtain the depth to the formation change as shown by the solid line curve of the graph. This is essentially a graphical integration of the upper curve (dashed-line) using a three-foot depth interval as the integration constant. The point of intersection of the straight lines in the lower curve (20.0 feet) is taken as the depth to the higher resistivity rock layer.

The right-hand graph of Figure 1 shows data for a test made over a proposed cut where a thin chert layer had stopped the auger making necessary a core-drill hole to prove the existence of the weathered material below. A 10- to 12-minute earth-resistivity test at this location indicated the need for earth slopes. The core-drill record is plotted at the bottom of the graph. The thin chert layer and cherty clay immediately below created a slight rise in resistivity which was quickly overcome by the effect of the underlying red clay. The analysis of the resistivity curve was based on the results of a similar test made over known weathered material in the immediate area.

Figure 2 shows seismic data obtained when investigating a project for slope design data in the vicinity of Washington, D. C. The time-distance graph is plotted from measured time and distances obtained as the test is carried out in the field. Each distinct formation layer will produce a straight-line segment in this graph, from which the velocity for the layer can be determined by taking the reciprocal of its slope. The comparatively low wave velocity of 2,855 feet per second obtained for the second layer, which extended to a depth of 21.5 feet below the surface, was indicative of weath-

[4] See reference No. 8 of the list of references given at the end of this paper.

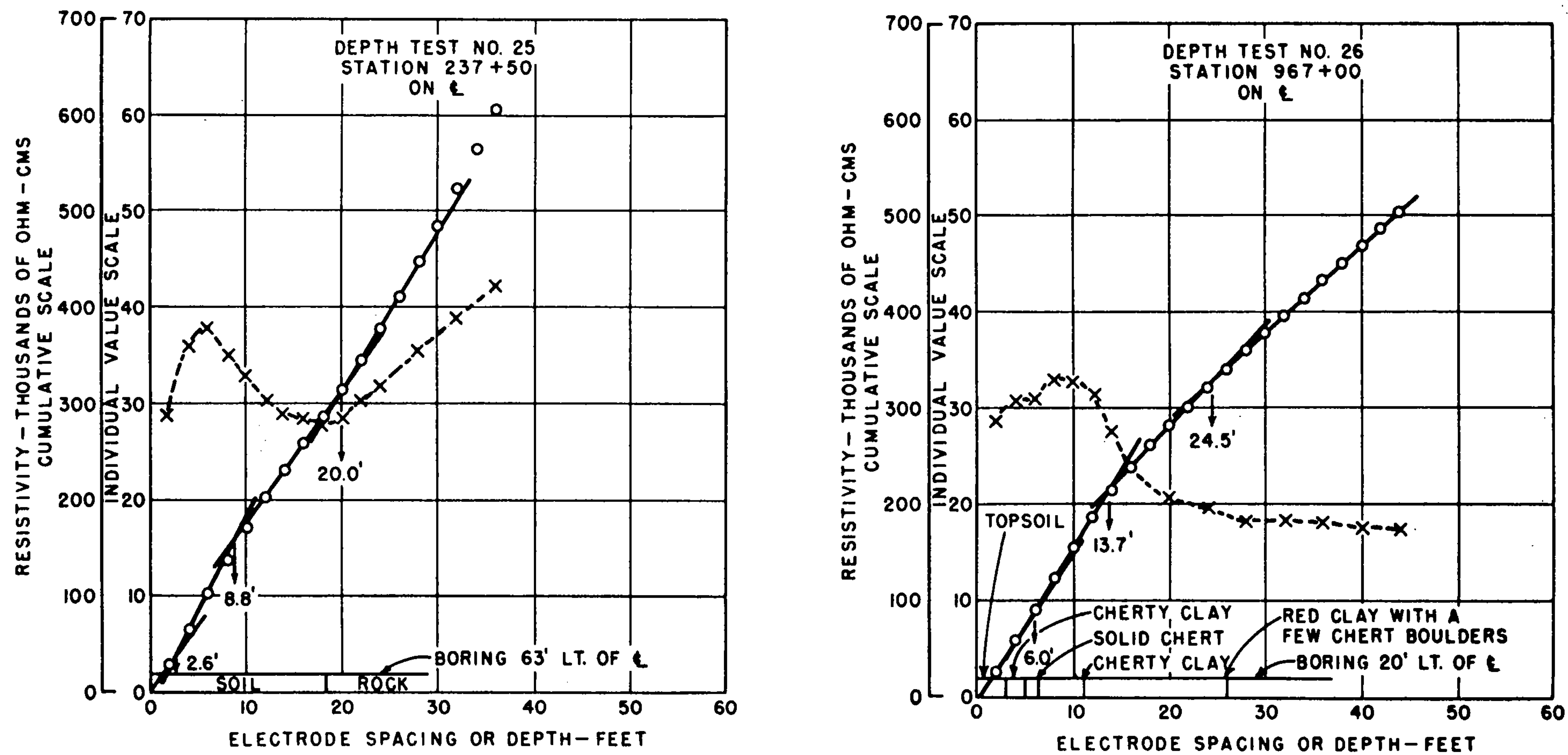

Figure 1. Results of earth resistivity tests compare favorably with boring data obtained to control slope design on Route 166, Jasper County, Missouri.

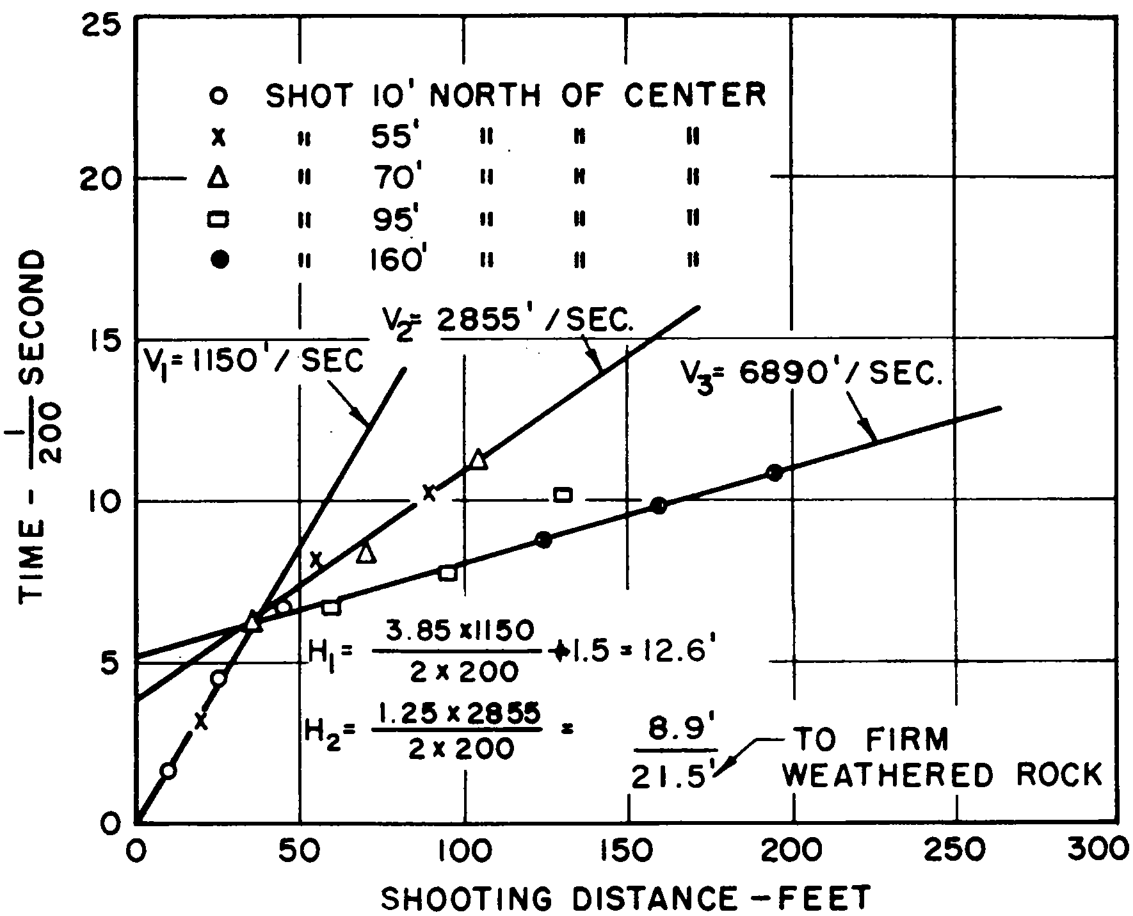

Figure 2. Time-distance graph for refraction seismic test made at Station 136 + 50, Project 1A1, George Washington Memorial Parkway, Arlington County, Virginia.

ered material requiring at least a 1 to 1 slope for stability and which would be easily ripped by modern ripper units.

The method used to compute the thickness of the formation layers is shown in Figure 2, using the relatively simple formulas normally employed by the Bureau. Actually, for a carefully drawn time-distance graph, it is possible to use an even more simple graphical solution as follows: From the point of intersection of the V_2 velocity line with the time axis, draw a horizontal line to intersect the V_1 velocity line. Project this point vertically downward to the distance axis and take one-half of this distance as the layer thickness. The second layer thickness is found in a similar manner by drawing a horizontal line from the point of intersection of the V_3 velocity line with the time axis, to intersect the V_2 velocity line and projecting this point to the distance axis, again taking one-half of the distance. When using buried explosives to produce the seismic wave, one-half of the shot depth should be added to the first layer thickness when it is obtained in this manner. There is little need to resort to more involved formulas or procedures for obtaining the layer depths when the ratio of V_1 to V_2, or V_2 to V_3, etc., is of the order of one-half or less.

Figure 3 shows data obtained by the electrical test on a cut section adjacent to that where the foregoing seismic test was made. The presence of weathered material is again indicated, the smooth-trending portion of the curve being almost identical with curves obtained over weathered rock ex-

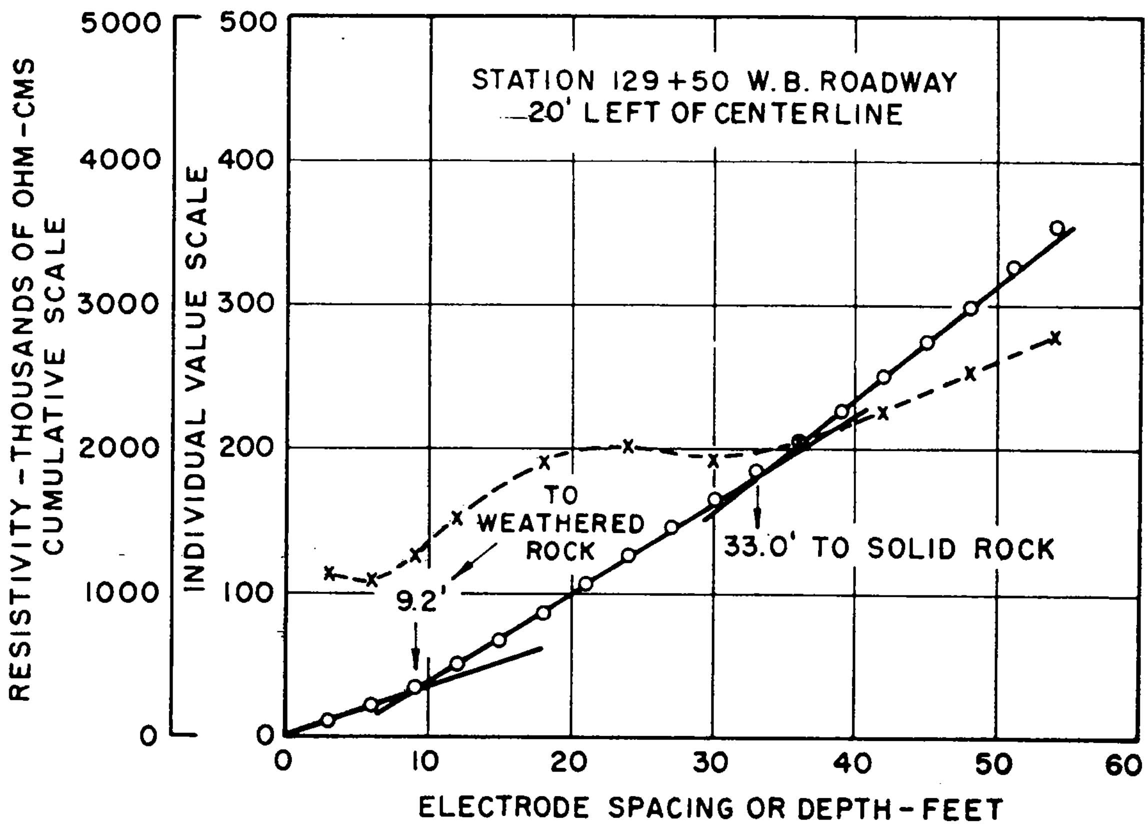

Figure 3. Earth-resistivity test showing changes from soil to weathered rock and to solid rock, Project 1A1, George Washington Memorial Parkway, Arlington County, Virginia.

posed in the local area. This material could be excavated to a depth of 33.0 feet using modern ripper and scraper units. The rise to higher resistivity at this depth signifies a more dense layer below.

Earth-resistivity tests requiring only 10 to 12 minutes for a 60-foot depth can be used in conjunction with relatively inexpensive rod soundings to provide rather convincing evidence of ledge rock in lieu of using a core-drill. Figure 4 shows data obtained in Wisconsin where these two procedures produced good results. The sounding rod, stopped by an "obstruction," is not generally conclusive regarding the existence of solid rock. However, when a check is made with the resistivity test, a strong continuing trend towards higher resistivity such as that in Figure 4 is good evidence of a continuous hard layer at an elevation corresponding closely with that where the sounding rod met refusal.

Perhaps it should be acknowledged at this point that the writer favors the use of the electrical resistivity test in obtaining data for the solution of many subsurface problems. Although some may question the propriety of such an approach, the fact remains that the electrical test has repeatedly produced good results in surveys made for all of the sub-surface problems listed above.

Foundation Studies

Both seismic and electrical resistivity tests have produced excellent results at structure sites. Use of the seismic test to locate suitable foundations for a bridge across a small stream is illustrated in Figure 5. Geologic conditions were excellent for this type of test at this location. The surface

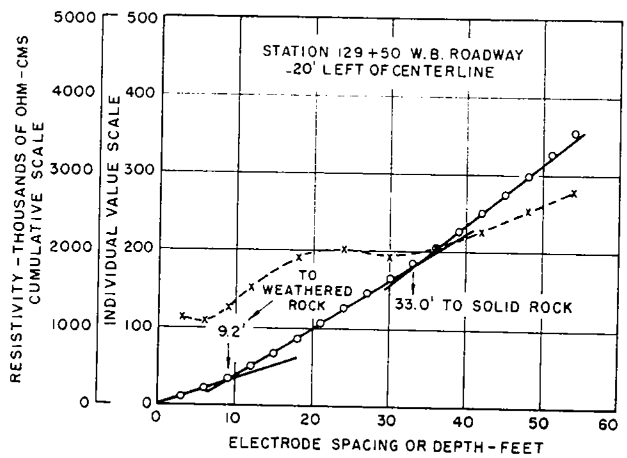

Figure 3. Earth-resistivity test showing changes from soil to weathered rock and to solid rock, Project 1A1, George Washington Memorial Parkway, Arlington County, Virginia.

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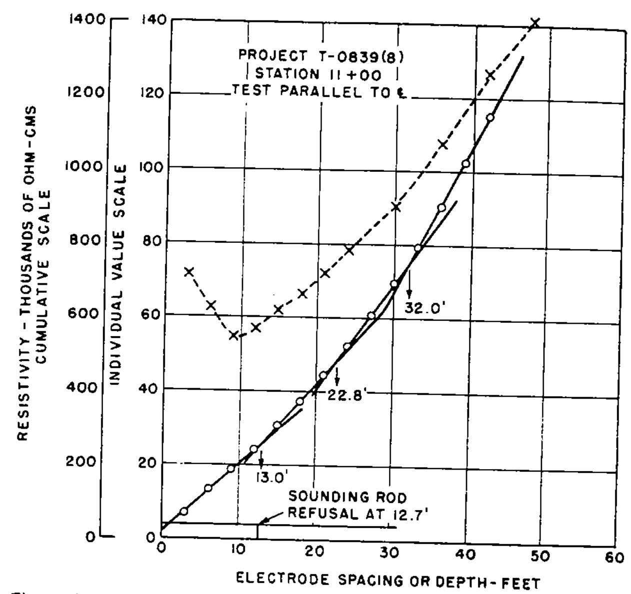

Figure 4. Showing correlation between results of resistivity test and rod sounding on roadway project in Washburn County, Wisconsin.

material was a light-weight volcanic pumice material having relatively low wave velocity with a very dense volcanic rock below possessing a high wave velocity. There is little need for further proving the character of the material possessing a wave velocity of 8.600 feet per second, since such material can be relied upon to be quite firm and possess good supporting power.

Figure 6 shows data for three earth-resistivity tests made at structure sites in Montana and Washington. The curve on the left was obtained in a calibration test made at a boring location on the west bank of the Nookchamps River near Mt. Vernon in northwest Washington. The definite uptrend obtained at a depth of 40.9 feet was produced by the shale bed found by the boring at a depth of 42.0 feet or at an elevation of −10.0. Since the boring made on the east bank found shale at elevation −17.0, 27.0 feet below where it was found on the west bank, some question existed regarding its position at a center pier location. The center curve of Figure 6 shows data for a test made in the middle of the stream where the apparatus was set up on a 6-foot-long sandbar. The men moving the electrodes as the test progressed wore hip-boots as they waded through the shallow stream. The slight but definite and continuing rise in resistivity appearing in the curve at a depth of 46.5 feet is good evidence of the shale bed at an elevation of

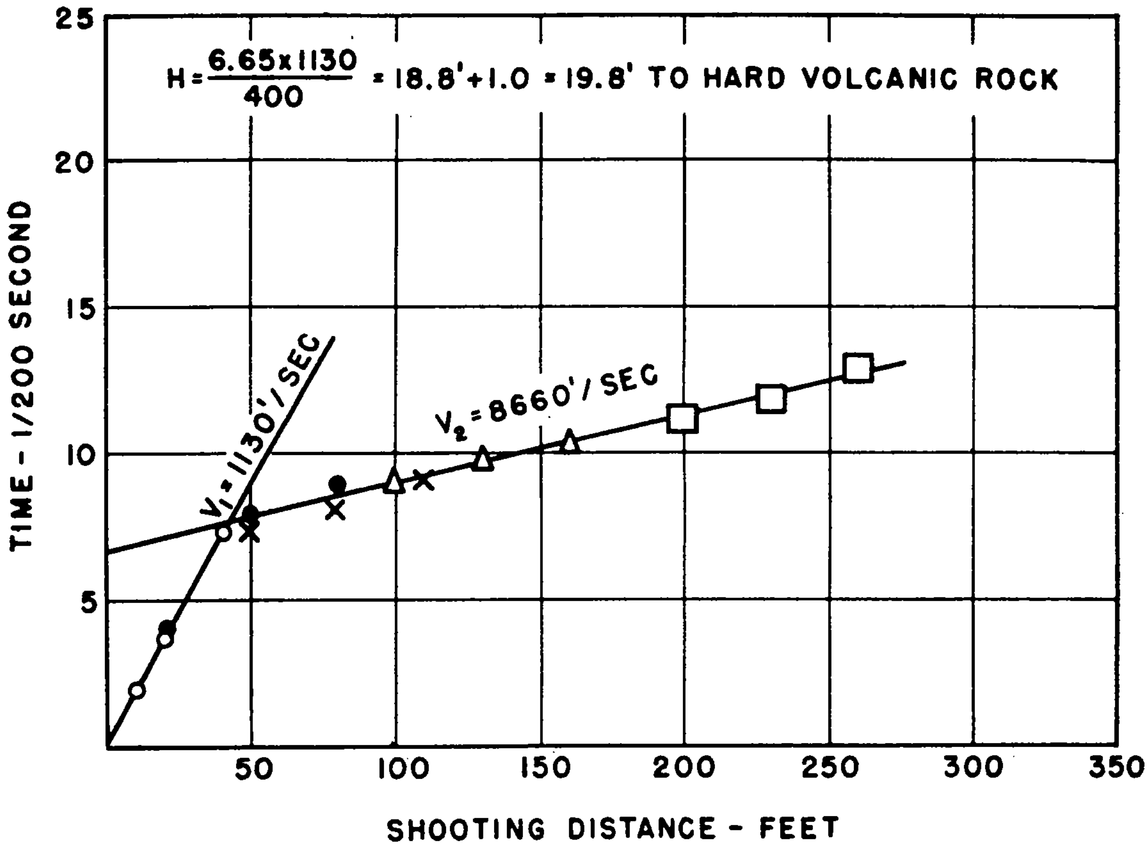

Figure 5. Refraction seismic test on north bank of Short Creek near Diamond Lake, Oregon.

—23.0 feet. This would place the shale beneath the center pier at an elevation about 6 feet below where it was found on the east bank. The right-hand curve of Figure 6 shows the effect of a gravel stratum found by the test at a depth of 8.6 feet. The continuing rise in the resistivity curve indicates a relatively thick layer of gravel or other more dense material capable of furnishing excellent foundation support for the railroad overpass under construction at the time the test was made.

Other Studies

Use of the electrical test to locate and evaluate hidden deposits of granular materials, so widely sought and used in modern-day highway construction, has been of considerable value to many groups in the past. Its use in a study of muck deposits and swampy areas has been well demonstrated also. Both seismic and resistivity test procedures have produced good results when applied to studies of quarry sites and the investigation of proposed tunnel locations. However, since discussion of the results obtained may be found in earlier publications,[5] no further consideration will be given these four rather important applications other than to direct attention to their effectiveness.

Some emphasis should be given to two applications of the electrical test, however, that are of special interest to the Bureau of Public Roads at present. One of these involves resistivity tests made directly upon the water surface when investigating foundation conditions for large structures cross-

[5] See reference Nos. 1, 3, 9, 13, 15, 19, 20 and 21 of the list of references given at the end of this paper.

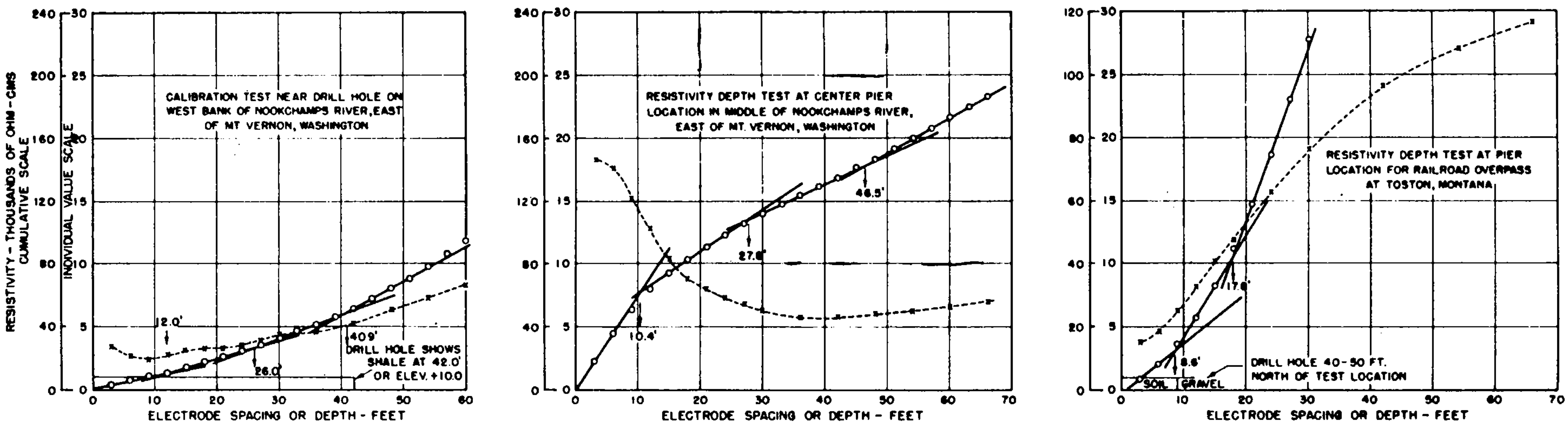

Figure 6. Earth-resistivity tests locate shale bedrock for bridge foundations near Mt. Vernon, Washington and sand and gravel for overpass foundations near Toston, Montana.

ing wide expanses of water. The other application has to do with the investigation of existing and potential landslide areas.

Resistivity Tests on Water Surfaces

With regard to the water tests, the procedure has been demonstrated thus far in Nicaragua, Alaska, Alabama, Louisiana, South Carolina and South Dakota. Only in the latter instance was it possible to make rather thorough use of the test to investigate full-scale river crossings and make a suitable comparison of the results of the electrical tests with boring data.

Figure 7 shows sections prepared from data obtained in resistivity tests made at two crossings of the Missouri River, one at Pierre, the other at Chamberlain, South Dakota. At both locations the resistivity test, which was made by floating the four electrodes directly upon the water when necessary, produced data locating the firm foundation material within a few percent of the elevation at which it had been found by auger borings made through the ice during the preceding winter. In both graphs the rock line found by the resistivity survey is indicated by the circled crosses and a heavy dashed line. The rock line found by the borings is shown bf crosses and a lightly-drawn dashed line.

The upper graph of Figure 7 shows the conditions found at Pierre where

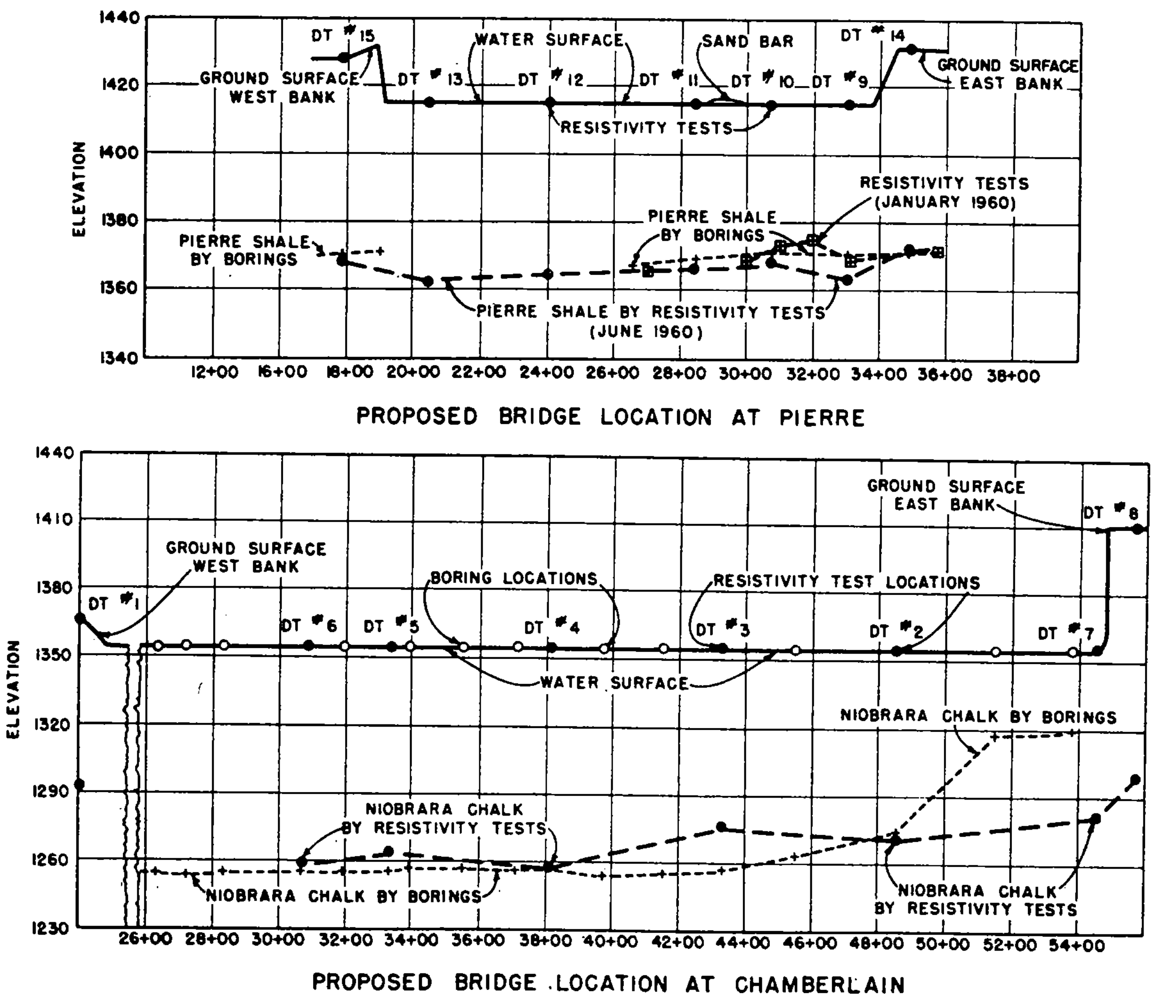

Figure 7. Results of resistivity tests made on water surface compare favorably with auger borings made through ice to locate rock line beneath the Missouri River at Pierre, and at Chamberlain, South Dakota.

the foundation layer was Pierre shale. The firm shale had a resistivity that was considerably lower than that of the river deposits. This produced downtrends in the curves plotted for the several tests. At the Chamberlain location (lower graph) the underlying Niobrara chalk possessed a higher resistivity than that of the unconsolidated river deposits which produced uptrends in the several curves.

Obviously, there is need for control borings or drill holes to determine the exact nature of the several layers producing noticeable trends in the resistivity curves. The cost of core drill operations over water can approach 10 to 15 dollars per linear foot. Any means, such as the resistivity test, that can be used prior to the much more costly direct exploration methods will reduce costs by eliminating all but the minimum number of corroborating borings for determination of subsurface conditions at piers. When it is considered that a well organized crew could perform two to four resistivity tests per 8-hour day over water, carrying them to depths of 130 to 140 feet, the cost per foot of depth for the electrical test becomes rather insignificant in comparison to direct methods.

Attention is directed to the elevation for the rock line established in the lower graph (Chamberlain location) by resistivity tests Nos. 3, 7 and 8. Test No. 3 is obviously in error, a fact that would likely have been evident from comparison with the results of other tests that would have been made at pier locations on either side if the resistivity survey had been made for the entire crossing and prior to use of borings. However, the rock line established by tests Nos. 7 and 8 appears logical. Further borings were to be made to establish the true elevation of the rock between station 48 + 60 and the east shore line.

Figure 8 shows data for a resistivity test carried to a depth of 120 feet below the surface of the Ashley River at Charleston, South Carolina. The three strong trends toward definitely greater resistivity appearing at 41.0 feet, 65.0 feet and 99.0 feet were undoubtedly closely associated with the depth of the water and the thickness of successive layers of marl having greater density as indicated by the effort required to drive the casing for a boring made nearby.

Landslide Investigation

Perhaps there is no greater problem facing the highway engineer than that of stability of fill foundations and cut slopes involved in placing the tremendous fills and making the deep cuts generally associated with some of the parkway, thruway and Interstate construction. The complexity and magnitude of this problem warrants the use of all types of surface and subsurface exploration, including aerial photography and geophysics, to provide a design adequate to prevent occurrence of landslides or to alleviate existing landslide conditions.[6]

One of the special studies now being emphasized by the Bureau to achieve this purpose involves the use of the resistivity test to provide detailed surveys. A limited demonstration of the procedure for such studies has been made in 14 States and in Central America. Except in Ohio,[7] they have involved a few tests made along a line through the center of the sliding area to obtain data permitting a plotting of the probable slip surface and its

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[6] See reference No. 23 of the list of references given at the end of this paper.
[7] See reference No. 21 of list of references given at the end of this paper.

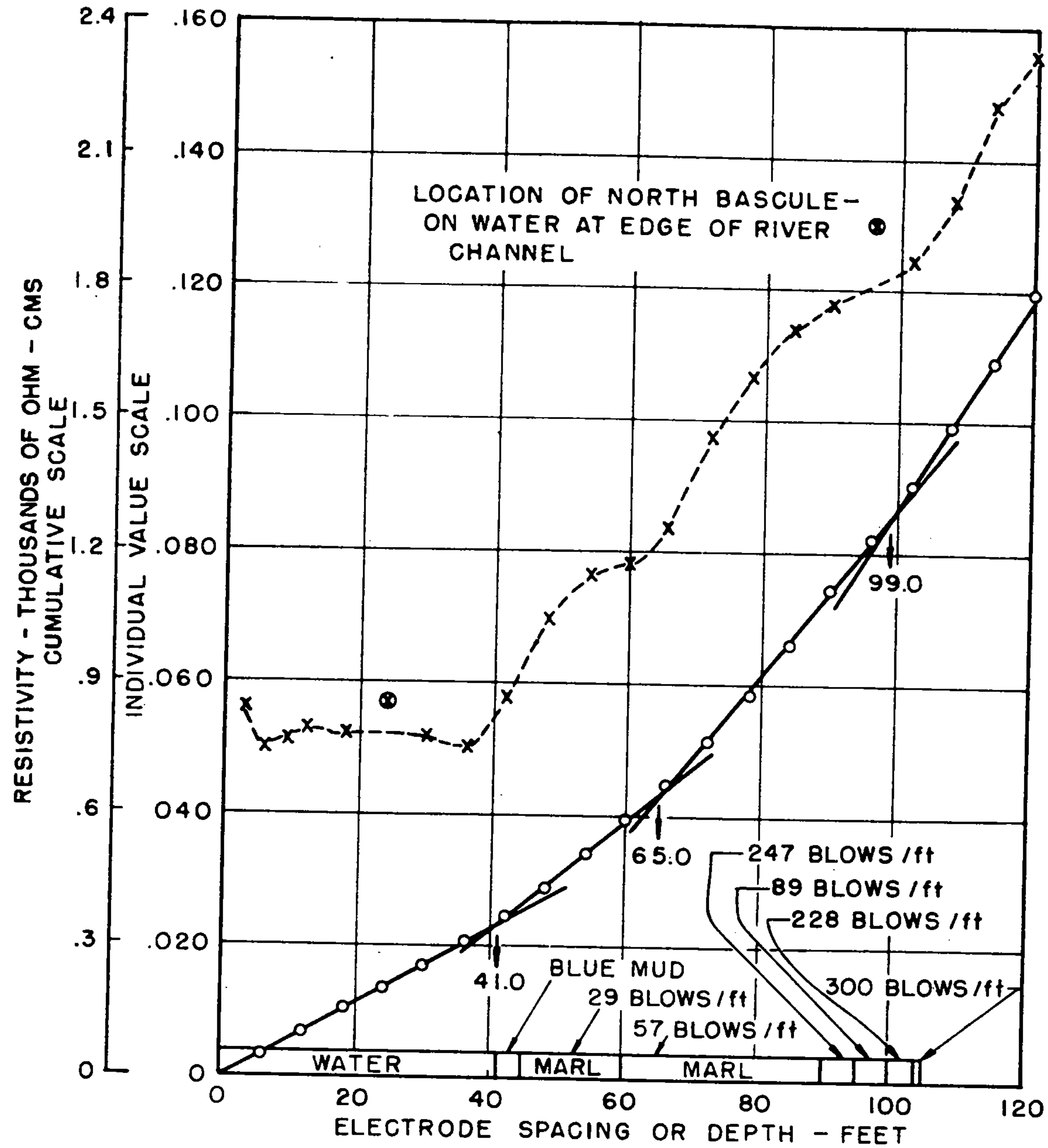

Figure 8. Results of earth resistivity test made on surface of the Ashley River—Charleston, S. C.

position below the slide surface. There is need for a far more comprehensive study of the application of such a procedure to this very interesting problem.

The possibilities of this rapid test procedure are illustrated in Figure 9. Shown is a section prepared from data obtained from 11 tests made across a landslide which developed early in 1959 on a section of the Blue Ridge Parkway in western North Carolina. The heavy dashed line has been drawn to outline the probable surface of an impervious rock formation upon which the comparatively large mass of talus and old slide material moved. Obviously, due to the great volume of material undergoing movement, removal of the sliding mass could be prohibitive in cost. A likely solution is the use of lateral drainage to dry up the slip surface to permit frictional forces to be developed beneath the great weight of material involved. Actually, the first action taken might involve removal of the sliding material on a 50-

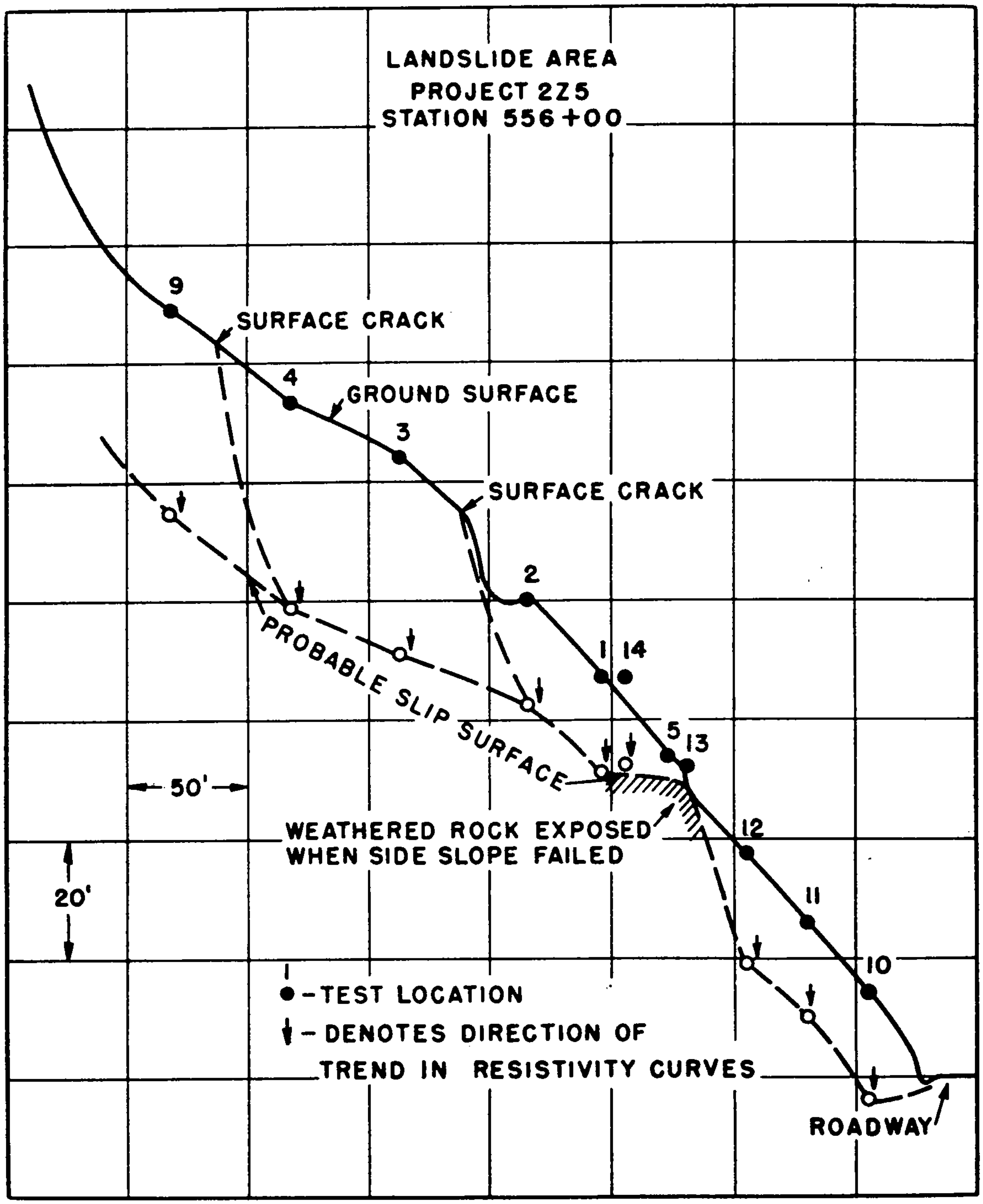

Figure 9. Resistivity tests establish probable slip surface in landslide on Great Smoky Mountains Parkway in western North Carolina.

to 75-foot-wide bench at the top of the cut slope with interceptor trenches placed in the weathered rock bench floor to prevent further water from reaching the cut slope in any quantity. This would permit the slope to be rebuilt under stabilized conditions and the bench could be used as a working area to remove any slide material entering the area from above before it could involve the travelled roadway. Regardless of the solution finally chosen, the analysis of resistivity test data has permitted one to philosophize upon one procedure that might succeed and will permit a considered selection of locations where other needed information might be obtained by use of borings, drill holes or test pits.

Presented in Figure 10 are curves plotted for each of the 11 tests made to obtain data for plotting the section shown in Figure 9. Note the definite, sometimes sharp and again slight, "down-dips" (circled areas) occurring in each of the 9 curves involving some depth of overburden above the suspected slip surface. This would exclude the curves for tests Nos. 5 and 13. These "down-dips" were likely produced by one or both of two possible causes. One might involve the abnormal increase in saturation of the soil mixture just above the impervious slip surface which could, in some cases, produce sudden and significant changes in its resistivity. The other may involve actual electric potentials established by forces operating at the slip surface. As mentioned earlier in this discussion, it is possible for water moving through a porous layer to create an electrical potential that may be partly. if not altogether, responsible for the "stray potentials" often measured but normally eliminated by various means when obtaining and analyzing resistivity data. In fact, such phenomena might well be associated with any distinctive change in moisture content or structure from one layer to the next. Such phenomena (self potentials) are measured regularly in deep drill holes being investigated for possible oil reservoirs or in a search for porous formations containing potable water supplies. A review is being made which involves the re-evaluation of resistivity data obtained over a wide variety of geologic conditions found from Panama to Alaska and Virginia to Hawaii, to determine whether a significant trend does occur that might reveal the existence of such natural potentials in the very shallow studies connected with highway work.

In the study, which is still in progress, an empirical procedure involving the ratio of natural potential to applied potential is being used. The ratio values are arbitrarily plotted against electrode spacing or depth in the same manner as when plotting the resistivity-depth curve. Either a "peak" in the plotted data or a continuing trend towards larger values of the ratio beginning at a particular point was assumed as criteria for "good" correlation of variations in natural potential with changes in resistivity when occurring at comparable depths. Figure 11 shows "ratio" curves plotted for data obtained in Panama, Costa Rica and Arkansas. The arrows show the location and direction of trends obtained in the resistivity curve. The actual subsurface conditions found by borings, or by direct inspection of cut slopes, are shown along the base of each graph. In the upper left-hand graph, the "peak" at 12.0 feet corresponds well with the change found by the boring at 11.0 feet. The strong increase in ratio values beginning at the depth where the boring found shale is also of interest. A distinct downtrend was obtained in the resistivity-depth curve at a depth of 23.5 feet at this location. A similar increase in the ratio values when shale was involved is shown in the upper right-hand graph which was plotted from data obtained in northwest Arkansas. At this location a strong downtrend was obtained in the resistivity-depth curve at a depth of 9.0 feet. Referring to the lower left-hand graph, there is some relation of the "peaks" in the curve to the change from one soil layer to the next and from soil to the underlying rock layer. The fourth graph of Figure 11 shows almost no variation in the ratio values obtained for a test made over a rather uniform sandstone formation in Arkansas. It seems more than coincidental that the peaks or trends in the "ratio" curves should occur so near the depths at which the borings found changes in the subsurface materials and where substantial trends appeared in the original plotting of the resistivity curves.

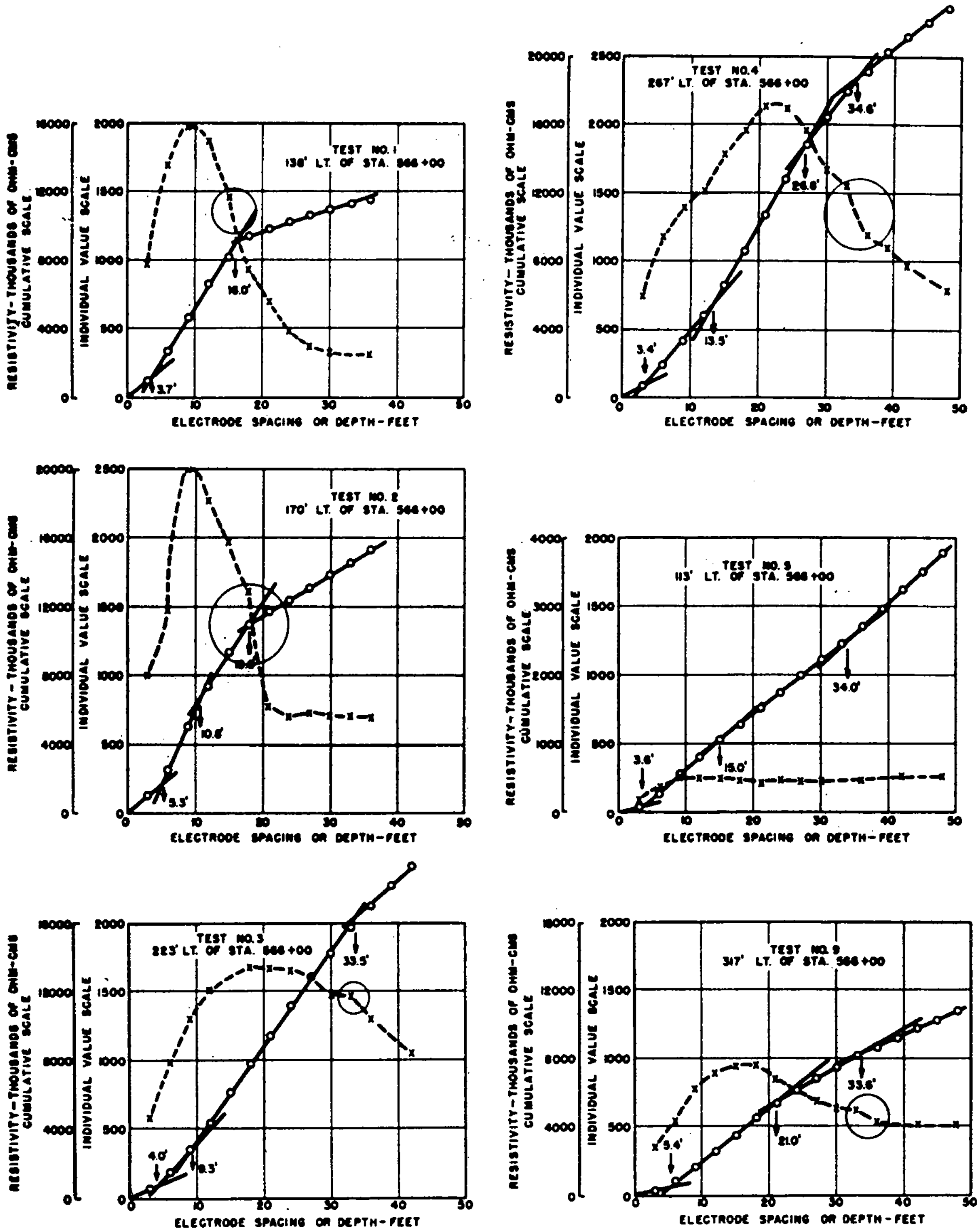

Figure 10. Earth resistivity curves showing increased downward trend in resistivity likely produced by conditions existing at slip surface in landslide area on Project 2Z5, Blue Ridge Parkway, in western North Carolina.

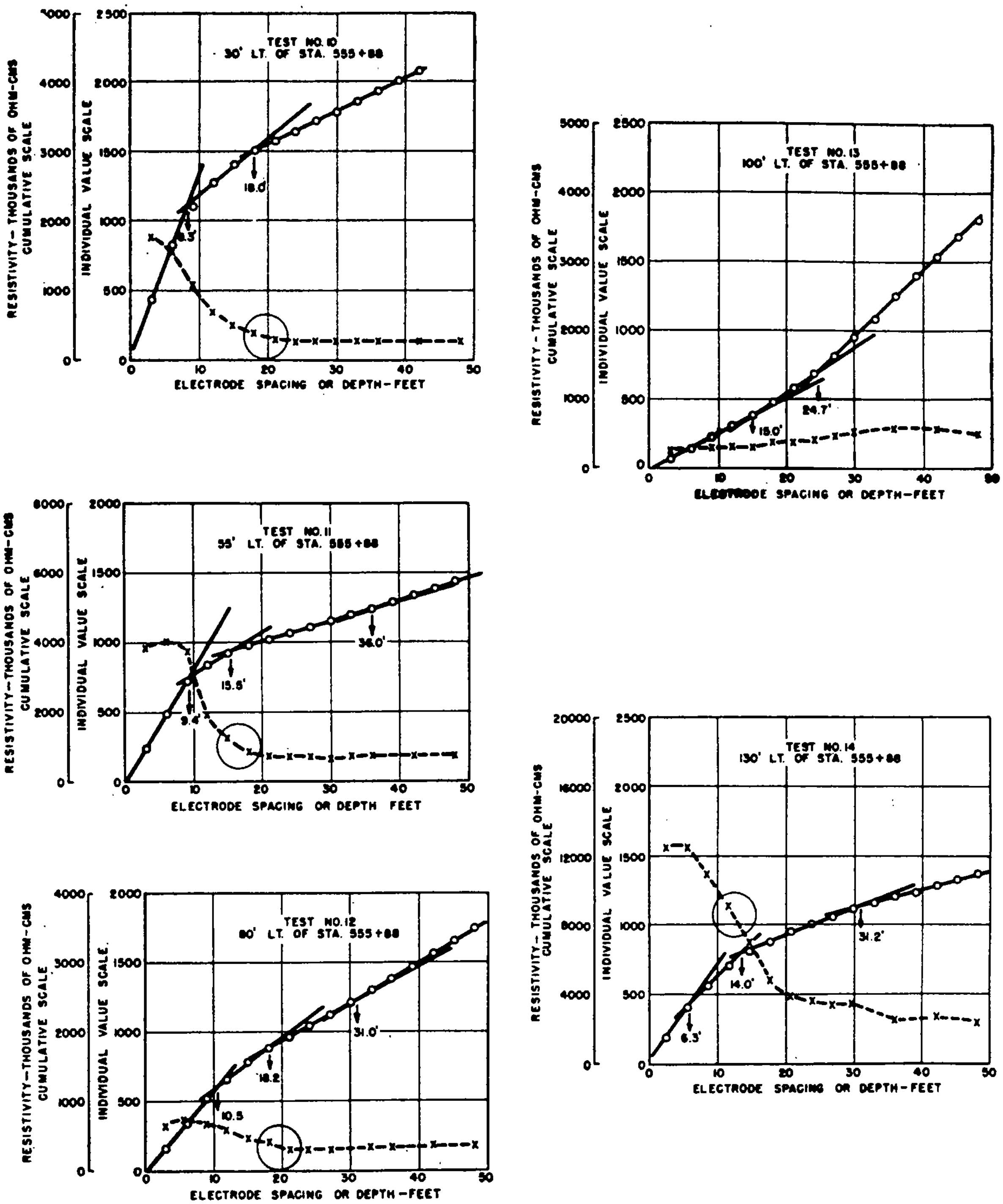

Figure 10 (cont.). Earth resistivity curves showing increased downward trend in resistivity likely produced by conditions existing at slip surface in landslide area on Project 2Z5, Blue Ridge Parkway, in western North Carolina.

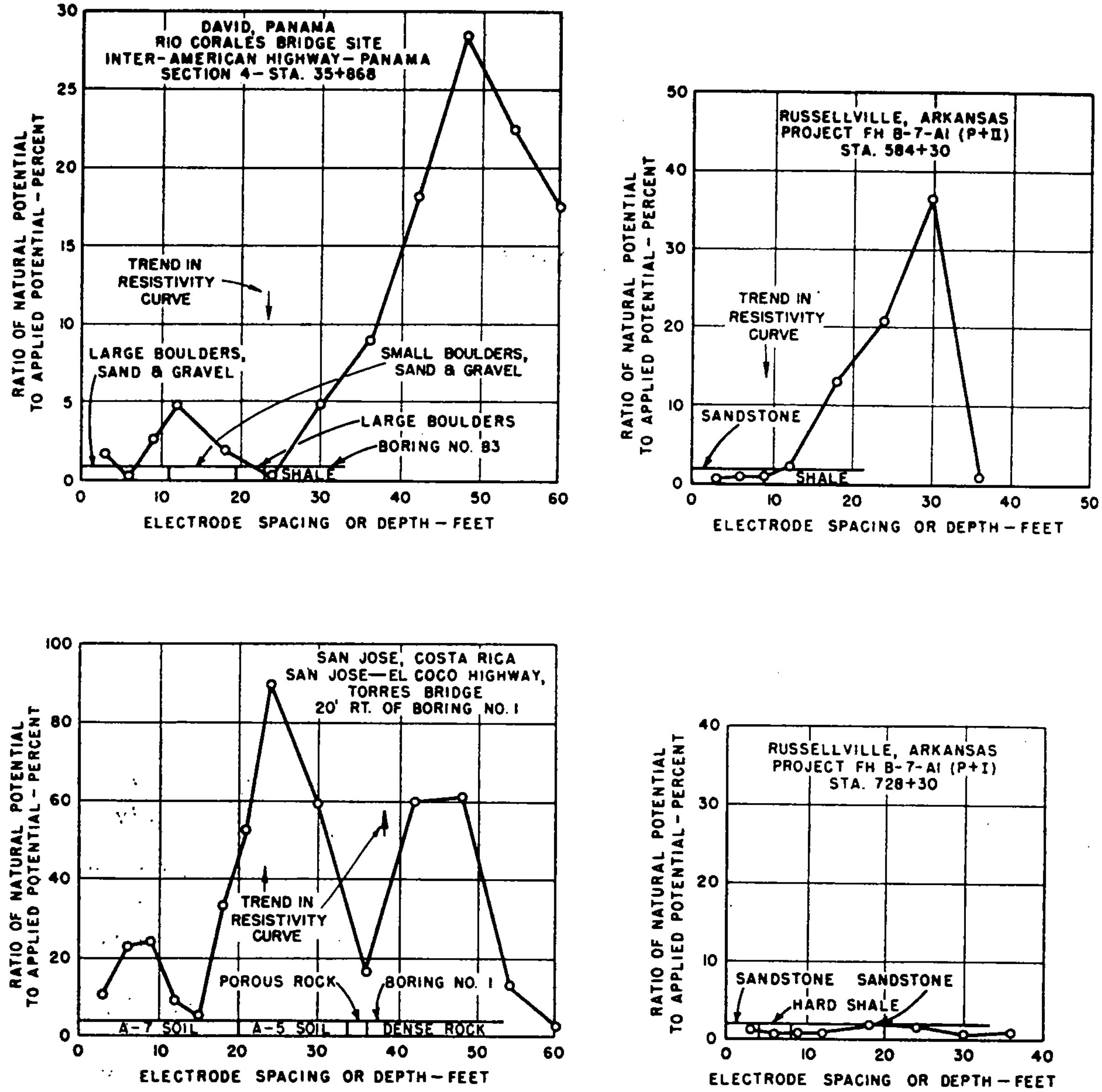

Figure 11. Curves showing possible correlation of natural potentials with moisture, density or formation changes in the subsurface.

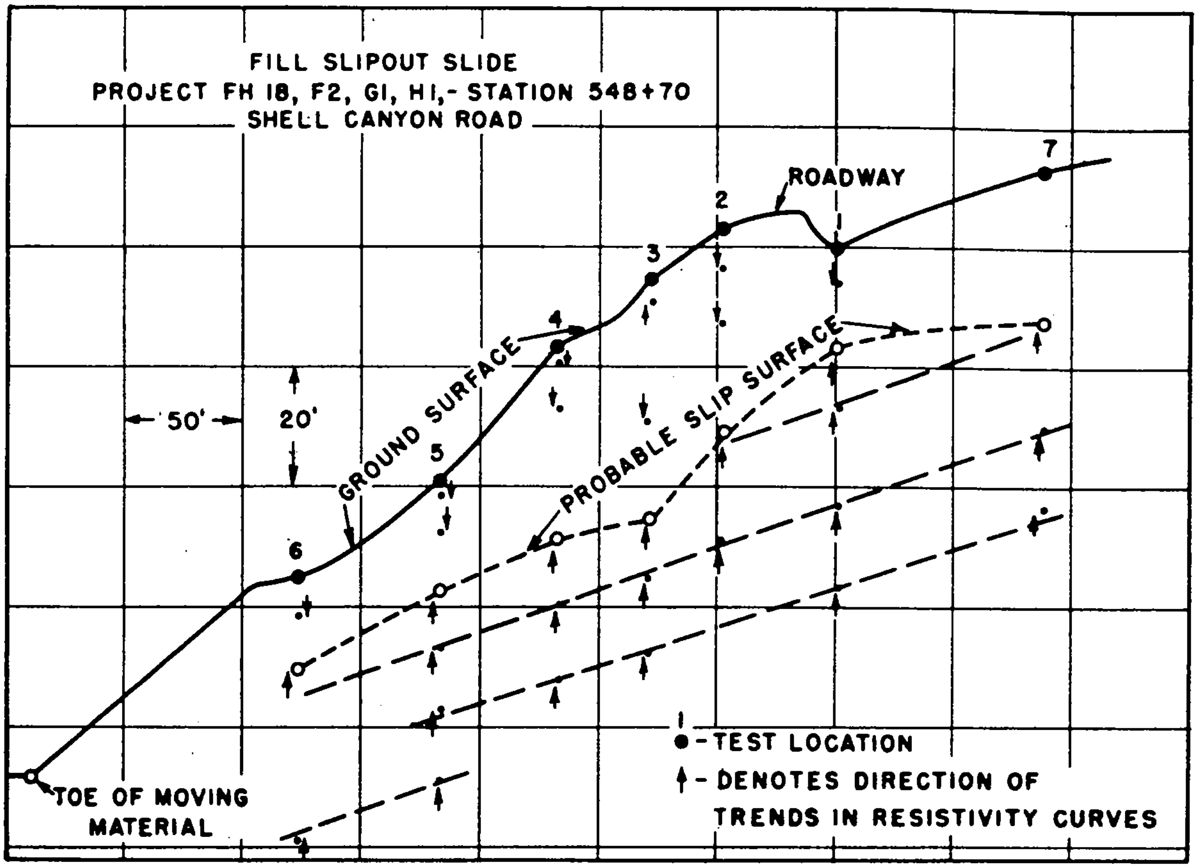

Figure 12. Section showing possible slip surface located by resistivity survey in slide area on Route 14, 60 miles southwest of Sheridan, Wyoming.

Chance is again ruled out when it is considered that similar treatment of the data from about 1500 tests, involving several thousand "changes" in resistivity, has indicated that over 80 percent of the resistivity changes were at depths where ratio "peaks" or "trends" were also found.

Figure 12 shows a section plotted from the results of resistivity tests made in August 1960, when studying a severe slide condition that had developed on the Shell Canyon Road in Wyoming some 60 miles southwest of Sheridan. The probable slip surface is again shown by the heavy dashed line which suggests that relatively large masses of material are undergoing movement. This survey was completed by the writer and two other engineers in a 5- to 6-hour period. The tentative decision to attempt the interception of the water contributing to the slide condition by use of interceptor ditches or lateral drains is possible based upon the data of Figure 12. A boring or test pit carried to a depth of 16 to 18 feet in the upper ditch line could serve to prove the character of the material upon which the slide developed.

Figure 13 shows the curves for the 7 tests made at the Shell Canyon location. The strong uptrends in each curve are ample proof of a layer of high resistivity material existing throughout the length of the slide area tested. The boring referred to above and perhaps one other to prove the absence of any low resistivity material capable of upholding the slide material would be conclusive in validating the suggested conditions shown in Figure 12.

The effort required to reach suitable boring locations on steeply sloping landslide areas can be both time-consuming and costly. Some locations may be unattainable. Any means, such as the resistivity test, that can be used

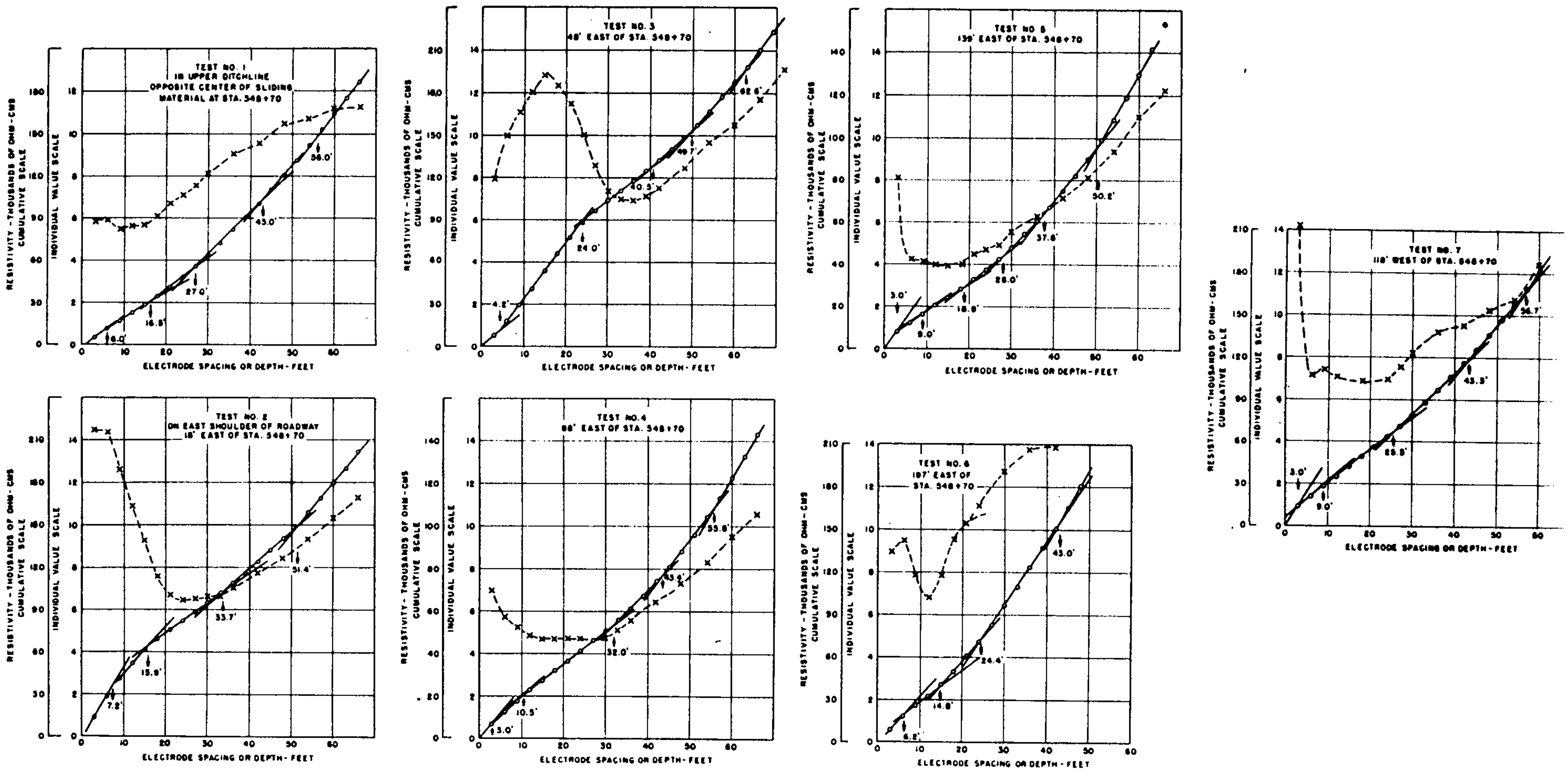

Figure 13. Resistivity-depth curves showing trends in resistivity produced by subsurface layers beneath landslide area on Shell Canyon Road in north-central Wyoming.

prior to attempting such borings may be quite helpful in placing them at strategic locations and indicating depths at which extra care should be exercised in sampling, as well as limiting their number to a bare minimum. This rapidly made test is also valuable in obtaining information on subsurface conditions existing between bore holes.

COMPARISON OF GEOPHYSICAL APPARATUS AND PROCEDURES

It has not been feasible for the Bureau of Public Roads to purchase each new seismograph or resistivity apparatus as they became available and make suitable tests to properly evaluate their relative efficiency and usefulness. As of January 1, 1961, there were at least 5 hammer-impact type, single-channel seismographs being marketed. These range in cost from 750 dollars to 2700 dollars. Add to these an equal number of 6- and 12-channel conventional seismographs along with 8 or 10 commercially built resistivity instruments and the problem of properly evaluating their relative merits can be fully assessed. However, some generalized observations can be made which may clarify to some degree questions arising from impressions or assumptions existing regarding claims made for a particular geophysical test procedure or type of apparatus.

There are definite limitations to the use of the seismic test on some subsurface problems. As previously stated, this test is satisfactory to locate hard rock surfaces beneath layers of less dense material. It has a more limited use in differentiating between the much less dense soil layers. In explorations for slope design the seismic test cannot be used to locate and evaluate less dense layers beneath more dense surface materials such as, for example, weathered lava buried beneath a more recent flow of hard unweathered lava. Formations involving interbedded sandstone and shale or limestone and shale cannot be proved to the full depth of the proposed grade line due to the more dense sandstone or limestone producing a high velocity seismic wave which precludes the registration of the slower wavefront arriving from the underlying layers having lower density. Thin but continuous layers of chert or other hard material could mask the presense of a decomposed rock layer below which would likely control the design of slopes in the cut.

The seismic test is not particularly useful in a study of landslide conditions, being more or less limited to those conditions where a hard, impervious layer constitutes the slip surface. The heterogeneous character and limited lateral extent of the material undergoing sliding could interfere with the proper evaluation of seismic data. This would be particularly true when using equipment, such as the hammer impact devices, which does not provide a permanent time record. Use of this test to locate construction materials is limited primarily to quarry sites, since a number of subsurface materials can produce wave velocities similar to those present in sand and gravel. The thickness of a layer of granular material could not be established unless the underlying formation possessed greater density. The water table in a granular deposit could produce a wave velocity that could be confused with that of an entirely different formation.

Seismic tests over water-covered areas, although possible, will require special equipment and field techniques more complicated and time-consuming than those used in land tests. For general seismic work a multi-channel seismograph, making use of three to twelve detectors and producing a permanent time record, is far more likely to produce acceptable results than are the small single-channel, hammer devices receiving so much publicity in recent years. Heterogeneity in the near-surface soils produces over-all time intervals, such as those recorded by the single-channel seismographs, which can produce confusion in the plotted time-distance graphs used to determine the wave velocity in a particular layer and the depth to underlying subsurface formations. Local "stray noise" levels and energy-absorbing surface soils can combine to limit the depth attainable by hammer impact blows to far less than those required on a given project. Seismic tests made with the single-channel units will require more time for completion than is generally referred to in many of the published reports, if they are carried out properly. This will be particularly true in areas having high natural noise levels causing much repetition of hammer blows to obtain points considered valid for use in the time-distance graph. Such noises may emanate from passing airplanes, wind in tree-tops, or in heavy grass, etc.

There are many instances when the presence of relatively thin intermediate layers will not be shown in time-distance graphs plotted from first arrival times. Secondary wave arrivals are used to advantage in such cases but a permanent time record is required which is not furnished by the hammer devices. Figure 2 illustrates this fact. The second layer would have been "missed" by a single-channel seismograph, yet it accounts for 27 percent of the total depth to the higher velocity layer. The data of Figure 5 would be obtained equally well by both single-channel and multi-channel units.

The earth-resistivity test has produced good results in tests involving most, if not all, of the subsurface problems discussed in this paper. There will be times, however, when the relative resistivities of the materials involved at a particular test site will be so similar as to produce questionable test data. Such conditions have been found rather infrequently in the work done by the Bureau.

As already stated, the resistivity test is satisfactory for locating solid rock foundations, sand and gravel deposits and quarry material. It is also useful in landslide studies, tests over water-covered areas, tunnel investigations and slope design studies. In the latter two applications the presence of less stable materials beneath the hard rock layers, a condition detrimental to use of the seismic test, can be recognized rather easily when based on careful calibration of the test over both types of material in the immediate area.

Buried pipe lines and other metal structures can be troublesome in the electrical test when they exist beneath the test area. When testing on salt water surfaces, special rheostats will be required in the current circuit to control the currents established in the highly conductive water. This is necessary in order to keep the currents to be measured within the range of the instrument's capacity to measure them. An extremely sensitive potential circuit is essential to permit successful tests to be made on salt water and over other materials of extremely low resistivitiy. In obtaining the data for Figure 8, the high degree of sensitivity in the potential circuit in the apparatus developed and used by the Bureau was responsible for the success of the test. Of 37 separate readings of potential obtained in this test at depths greater than 24 feet, the average potential drop measured was only 1.79 millivolts. This was less than 0.2 of one percent of the total range of the apparatus. Less sensitive resistivity instruments such as those making use of alternating current circuits and other special features designed to speed-up or simplify the field test would not produce satisfactory results under similar conditions. This high degree of sensitivity was required in the tests producing the curves of Figure 10, also, to permit recognition and use of the very minor trends appearing in several of the curves when analyzing the test results.

Any procedure for recording, computing and plotting the resistivity data in a suitable field curve that does not provide for its completion while the test is in progress will likely be unsatisfactory. This is true because of the need to recognize all trends appearing in the curve and prove their validity by check readings made before terminating the test.

In conclusion, since the fundamental principles involved are quite different for the seismic and resistivity tests, their use to corroborate each other is warranted. When both tests give the same answer, the use of borings or drill holes may be avoided for some subsurface problems. Use of the various direct exploration procedures will be dictated by the type of information desired and the detail required. All available methods, however, should be considered and used, if necessary, to assure as complete a knowledge of subsurface conditions as the particular problem warrants.

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THE ECONOMICS OF NATURAL RESOURCE VALUATION

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The term "Natural Resources covers such a broad field that for the purpose of this meeting I am confining my discussion of valuation to such resources more frequently encountered by appraisers and highway engineers, and those in which I believe geologists and highway engineers have the most common interest. It is my thought that sand, gravel, and rock are those deposits which meet this requirement.

In the valuation of land containing sand, gravel or rock and similar deposits, two basic methods of valuation are appropriate in arriving at an opinion of the Fair Market Value of the property.

These methods are:

(1) The market or sales approach, through which the estimate of value is based upon a comparison of the subject property with sales of properties containing similar mineral deposits.

(2) The income approach, through which the rent, or royalty, payable to the owner by the producing company for the privileges of extracting and removing the deposits is capitalized into value, to which is added the discounted value of the depleted land. This method, when applied to a property which is operated by the owner, is known as the relief from royalties method, since the owner is relieved from the necessity of paying royalties on leased deposits.

In the valuation of a special purpose property, it must be assumed that the owners are willing to sell for an equitable price, and that there exists a buyer having a need and desire for such a property. The willing buyer would have the option of (1) acquiring the property, (2) some other comparable property, or (3) locating in some other locality in the area with similar deposits and assembling a comparable property, if necessary, by purchasing existing smaller holdings.

One school of thought that holds that the deposits must be valued with the land and subscribe only to the market approach to value. This school has its arguments well documented by recognized informed authors and by court decisions on questions of Eminent Domain.

Section 611 of the 1954 Revenue Code states in part, "The district director will give due weight and consideration to any and all factors and evidence having a bearing on the market value, such as cost, actual sales, and transfers of similar properties and improvements, bona fide offers, market value of stocks or shares, royalties and rentals, valuation for local or state taxation, partnership accountings', record of litigation in which the value of the property and improvements was in question, the amount at which the property and improvements may have peen inventoried or appraised in probate or similar proceedings, and disinterested appraisals by improved methods."

I quote the above paragraph to indicate the market or sale approach is not the only approach and that valuation through royalties is one of a number of sonsiderations.

Under the "market approach" rarely does one find a property recently sold that is directly comparable to the property under review, and it is necessary, therefore, to make percentage adjustments to the sales price for the differentials among the properties. In making this comparison and adjustment, it is obvious one must go through a number of the mechanics of other approaches to value.

Elements of Value:

The adjusting and comparative factors to be considered are:

Area Involved
Quantity of Deposit
Quality of Deposit
Access to Deposit
Distance from the Market

Area involved and quantity of deposit can possibly be bracketed; the size and shape of a deposit has a varied effect upon the value of the land. A five-acre parcel is less valuable per acre than a fifty-acre parcel, because it has less recoverable tonnage per acre due to pit slopes. Depth of the deposit is important; also the amount of overburden. A parcel that can be excavated to a depth of fifty feet and with an overburden of one foot is obviously more valuable than one with a depth of thirty feet and of overburden of three feet. Both the potential depth and the amount of overburden have an effect on value.

Quality of deposit as well as quantity can only be determined by competent geologists. Appraisers are obviously dependent on reports from these qualified technicians in their attempt to value property.

Access to a deposit has a marked bearing on value and must be considered in making a sale comparison.

The distance from the market is vital in analyzing a sale; a large rock deposit in the center of a huge highway project could well have a value five times in excess of one located 25 to 50 miles distant as the element of freight or trucking readily can spell the difference between profit and loss in a competitive market.

Let us assume, under the market approach, there is under review a rock deposit containing 1,000 acres which assays 25,000 tons per acre and that in analysis we locate a sale of a deposit which is five miles closer to the market, containing 1,200 acres which assays 20,000 tons per acre and that the quality of the rock is the same. For the purpose of illustration, we will further assume the sale is located on the main line of the railroad and the property under review will require the expenditure of $50,000 for a spur line connection to the railroad.

These assumptions would indicate "area involved" and "quantity of deposit" practically offset one another although some slight advantage might be gained by the property under review, inasmuch as the "sale" property requires the working of 1¼ acres of land to obtain the amount of rock worked from one acre of the property being appraised.

The "access to deposit" must be recognized in any comparison made, due to the cost of the railroad spur, which cost, together with a proper return on the investment, must either be recovered over the life of the deposit or absorbed in the initial development costs, and does indicate a somewhat lesser value per unit for the property under review than the unit value of the "sale" property.

"Distance from the market" is the most important factor in making a comparison between a property under review and a sale, assuming quantity and quality of deposits are comparatively equal. Investigation of hauling charges, existing royalties on other properties and other data available might indicate the "sale" under review has a 25 cent per ton advantage in this respect; if the prevailing market at the quarry is $2.00 a ton this would indicate the property under review is 12½ per cent less desirable than the sale. In consideration of this differential together with the other differentials disclosed, the appraiser possibly might conclude the property being appraised is worth 15 per cent less than the sale and, if the sale indicated a unit value of $200 per acre, his conclusion would be that the appraised property has a value of $170 per acre when considered from the "market" approach.

The "income approach" is predicated on existing royalties of comparable properties and separates the deposit value from the land value.

Royalties as the term is used in connection with natural resources are defined as:

(1) "A royalty is an enhanced rental, enhanced by the right to remove a part of the corpus. As it is expressed at a certain amount per unit of measure, the total royalty rental fluctuates with the total minerals gotten," or

(2) "In real estate usage, it is the money paid to an owner of realty for the right of depleting the property of its natural resources. Usually the royalty payment is a stated part of the amount extracted, or a given price of per unit of material extracted."

Royalties paid to owners for operation of stone, sand or gravel properties generally are based on individual conditions. Higher royalties are naturally paid for properties which are close to the market or are fully improved and operating, while much lower royalties are paid for unimproved or distant properties where the operator would be required to bear costs of development, removal of overburden, construction of roads or rail trackage or both.

The conventional method, which has been used for years, of determining value through the capitalization of estimated annual income through royalties, is predicated on the estimated royalty per unit, the number of units available and the number of units to be removed each year which in turn indicates the life of the deposit. The value of the property is then estimated on the basis of the present worth of the estimated income over the life of the deposits.

Consideration must be given to the rate of capitalization at which the estimated future net income should be discounted to indicate present value. Elements of consideration must include rates of return from enterprises involving a similar degree of hazard, and the recognition that a potentiality, not an actuality, is involved. It must be a rate which would attract capital to an investment of this kind.

To fix the total value of the land and deposit, we must add the reversionary value of the land remaining after the depletion of the deposit. By reversionary value we mean the estimated value of the land at the date of depletion discounted over the life of the deposit. If the royalty unit used is that of an undeveloped property, and the property under review is developed and operating, then we must add to the result obtained the estimated cost of development. This would include exploration, survey and removal of overburden.

Seasoned judgment in this approach to value is critical since the quantity available is an estimate, the annual production is estimated and the percentage allowed for risk is an estimate.

The conventional approach to value through capitalization of royalties as stated before has been in use for years and does not reflect the changing situation of more recent years of increased income taxes and the U. S. Internal Revenue Service's allowance of percentage depletion. (Code Section 613 U.S.I.R.S.)

Depletion:

The basic method of computing depletion is known as "Cost Depletion" (Code Sec. 612). Determination of cost depletion requires first an estimate of the number of units (tons, barrels, etc.) which make up the deposit. Then that part of the cost or other adjusted basis of the property which is allocable to the depletable reserves is divided by the number of units. The quotient is the cost depletion per unit. This amount multiplied by the number of units extracted and sold during the year, determines the cost depletion deductible for the year.

In order to encourage the development of resources and to compensate miners, prospectors and oil drillers for the financial risks inherent in these enterprises, a special method of computing depletion is provided by the Internal Revenue Service which ordinarily results in a possible reduction in taxes. This is known as "percentage depletion" and consists of a flat percentage of gross income from this property taken as the depletion deduction. The percentage depletion deduction may not exceed 50 per cent of the taxable income from the property without regard to the depletion allowance. Percentage depletion ordinarily permits recovery of a great deal more than costs. In the case of a lease, the deduction under this section shall be equitably apportioned between the lessor and the lessee.

There are then two methods of determining value through income in general use today and for the purpose of this discussion we will describe these methods as:

(1) Conventional approach
(2) Relief from royalty approach

The workings of these approaches can be seen in a simple example.

For the purpose of illustration, we will assume:

(1) The property is fully developed and operating
(2) 1,000 acres of deposit are involved
(3) Reserve remaining—as established by recognized geologists is 17,500,000 tons
(4) Production—500,000 tons per year
(5) Remaining life—35 years
(6) Original cost of land and deposit—$200,000
(7) Estimated value of land after depletion—$100,000
(8) Royalties—20 cents per ton
(9) Capitalization rate—15%
(10) Land reversion @ 8%

Under the conventional approach the computation would be:

Gross Income

500,000 tons @ $0.20 royalty equals	$100,000 per annum	
Less		
Real estate taxes	1,500	
Net Income	$ 98,500 per annum	
Capitalization rate 15 per cent for 35 years equals 6.617 (Inwood Factor)		
$98,500 at 6.617 equals indicated value of deposit		$651,775
$100,000 value of land at reversion		
Reversion in 35 years at 8% equals .0676 factor		
$100,000 at .0676 equals indicated reversion value of land		6,760
Total value of land and deposit		$658,535
	Say	$658,000

Under the relief from royalty approach, we assume the same basic facts previously outlined and separate the computation between ownership of land and deposit in fee and interest of a lessee of the deposit under a royalty lease.

This method does require a study and review of the property's current and historical accounting and operational records; I would suggest that a 5-year period, at least, be reviewed. Consideration of all expenses including sales expenses, equipment investment and depreciation, overheads, realty and personal property taxes and all other operating expenses must be included.

In the illustration presented, you must bear in mind these items have been considered and, although our working papers do show all of these factors in detail, in the final computation they are combined for the purpose of clarity of the computation illustrated.

In valuing an operating property where all of these costs, production records, and other data are available, the problem is comparatively simple. In valuing an undeveloped deposit the estimate will, of course, be quite complex as it will call on the experience of the appraiser to determine and estimate a great many factors such as:

Cost of development
Possible production
Probable investment in buildings and equipment
Return on the above investments and depreciation
Probable production costs of unfinished materials
Other overheads, indirect costs, etc.

In the illustration of the relief from royalty approach I am assuming a fully developed property is being considered.

You will note the illustration shows two columns—one under which the property is owned in fee, and the other where the mineral rights are leased.

Relief from Royalty Approach

	Ownership	*Leased*
Sales	$1,000,000	$1,000,000
Expenses (excluding depletion)	620,000	620,000
Royalties	—0—	100,000

Amortization or depletion charge based on original cost less estimated value of land after depletion (2,857) say	2,850	—0—
Total cost of sales and operative expenses before depletion charge allowance for tax purposes as based on sales	622,850	720,000
Pretax profit before allowable depletion charges as based on sales	377,150	280,000
Allowable depletion charge as based on sales	178,575*	10,000*
Taxable Profit	198,575	270,000
Federal Income Tax	97,759	134,900
Net Profit	100,816	135,100
Add Depletion Charge	181,425	10,000
Cash Flow	$ 282,241	$ 145,100

Ownership Cash Flow	$282,241
Lease Cash Flow	145,100

Saving Incidental to Ownership $137,141—Relief from Royalty

Computed value of deposits 15% capitalization 35 years is 6.617 (Inwood Factor). $137,141 x 6.617 is value of deposit, $907,462 or say $907,000.

*Based on 50% of pretax profit less 10% of lessor royalties received.

To the resultant value of the deposit must be added the	$907,000
reversion value of the land as previously determined	6,760
Indicated Value of Land and Deposits	$913,760
Say	$914,000

There is a school of thought that is of the opinion the allowable depletion under existing income tax laws is unrealistic and represents a government fiat which is subject to the whims of Congress and could be withdrawn at any time and, therefore, only cost depletion should be used in this computation.

If we consider this contention, using the same basic figures the computation would be as follows:

	Ownership	*Leased*
Taxable Profit	$377,150	$280,000
Federal Income Tax	190,618	140,100
Net Profit	$186,532	$139,900
Add Depletion Charge	2,850	—0—
Cash Flow	$189,382	$139,900

Ownership cash flow, $189,382, minus leased cash flow, $139,900, equals $49,482 saving incidental to ownership of relief from royalty.

Computed value of deposit—15% capitalization 35 years= 6.617 (Inwood Factor). $49,482 x 6.617 equals $327,422 value of deposit.

To the above computation must be added the reversion value of the land, hence:

Deposit		$327,422
Land		6,760
Total Value of Land and Deposit		$334,182
	Say	$334,000

It is obvious the allowable percentage depletion has a marked effect on the value determined but, it is the writer's belief we must, in preparing a current appraisal, recognize all existing laws. The current allowances for depletion have been in existence since 1954 and we have no method by which we can determine whether the Congress will, in the foreseeable future, change these allowances in any way. I am confident the existing allowances will not be wiped out in their entirety at any time in the near future and it is my opinion full recognition should be given to existing depletion allowances in determining the value of the deposit through the relief from royalty methods.

ALTERING PHYSICO-CHEMICAL CHARACTERISTICS OF CLAY-BEARING SOILS WITH LIME

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In highway and airfield construction clay-bearing soils—particularly the swelling type—are encountered a nemesis. Highway routes are often selected to avoid the bad clay belts; if this is not practical, excavation is resorted to and the unsatisfactory material is replaced with select borrow soil. If, however, the borrow is at a premium, it may be necessary to use the clay; but even here little or no credit is given to it strengthwise. If the contractor has to work the clay, he hopes for good weather so his equipment won't bog down.

Where lime stabilization is employed, however, clay soils actually can be upgraded into satisfactory road building materials, and at a relatively low cost. And strange as it may seem, construction can proceed at a fast rate, even in inclement weather. In this type of stabilization the function of hydrated lime is two-fold: First, through base exchange, the clay particles are agglomerated, forming the more friable silt and sand sizes; this in turn reduces the plasticity and volume change. Secondly, a cementing reaction occurs which increases the strength and stability many times, transforming the clay layer into a relatively impervious working table. It is to be noted that lime's application in stabilization involves the highly plastic soils (P.I. of 10-50+), whereas soil-asphalt and soil-cement are generally restricted to the non-plastic to low P.I. soils.

Lime stabilization is far from new—the Romans actually used lime in constructing the Appian Way about 2300 years ago; yet it was not until 1945 that the first modern successful lime stabilized road was built in Texas. The early projects involved the upgrading of in-place high P.I. clay-gravels for base construction, using 2-4% lime (by weight). Later, lime stabilization was extended to the treating of plastic, swelling clay soils for subgrades under higher-type pavements, using 3-6% lime. In either case, highway engineers have given credit strengthwise to the stabilized layer, thereby enabling the over-all pavement thickness to be reduced.

Since 1945 lime stabilization has grown tremendously, especially during the past few years, and is no longer in the test tube stage. The National Lime Association estimates that highway contract awards reached 24 million sq. yd. in 1959, reflecting about 300,000 tons of hydrated lime. Starting off in maintenance work, this use now covers the gamut of highway construction—Interstate to farm-to-market roads, shoulders, parking lots, etc., as well as several non-highway uses—airport runways, building foundations, and it has even been tried on railroad subgrades. The use has also spread to more than 30 states, and to at least 25 foreign countries.

Texas, the pioneer in this method of road building, is by far the leading state; in 1960 the T.H.D. alone used more than 160,000 tons of lime for stabilization purposes. Many single projects have required over 5000 tons, and one involved 12,500 tons. To this can be added considerable work done by the military, county, city and private forces in that state.

Although Tennessee has not built a job as yet, several of the adjoining states have done so, with Mississippi, in particular being one of the leading states. In Mississippi most of the work has involved the highly plastic montmorillonite clays and silty clays characteristic of the alluvial valley of the Mississippi River (Delta country) and the Gulf Coastal Plain—not unlike some of the soils of western Tennessee. Virginia has also built several jobs, and one (in 1960) on U.S. 460 near Pembroke (in the Ridge and Valley Province in southwestern part of the state), involved soil conditions similar to those existing around Knoxville; on this particular project the clay was predominantly an illite, having developed residually from limestone bedrock.

What Lime Does—Numerous physical and chemical changes occur when lime is mixed with a clay-type soil. These changes, of course, vary according to the clay minerals present.

1. Plasticity Index (P.I) is reduced, due to a decrease in the Liquid Limit and an increase in the Plastic Limit. In some cases the reduction is three-fold or more (the reduction generally is not as appreciable in the lower P.I. soils).
2. Flocculation of clay particles, forming silt and even larger sizes. This is shown up by a substantial decrease in the soil binder content (—40 mesh). Simultaneously, the large clods are broken down by the action of lime and water, making the soil more friable and workable. (See Photo 1).
3. Density decreases and optimum moisture content increases. The change may be 5% or more. Normally a density decrease is undesirable, but not in the case of soil stabilization (see item 7 below).
4. A pronounced drying action occurs by lime releasing the bound water in the clay. This was aptly demonstrated during a ponding-stabilization foundation job[1] built in Texas over a year ago. On this job an expansive clay soil was pre-expanded by ponding for 30 days. After draining, 5% lime was added to the over-saturated soil; and within 4 hrs. after mixing started, the soil was firm enough to support a light pickup truck.
5. Volume change is drastically reduced. Lime has been used under concrete pavements, specifically to reduce subgrade expansion and prevent warped joints. On an Interstate job in Kansas,[2] 5% lime reduced the expansion of the subgrade clay from about 6% to less than 0.3%.
6. Resistance to water absorption and capillary rise, since lime forms a water-resistant barrier. This was one of the reasons for using lime on the foundation job mentioned above, i.e., the stabilized layer holds the moisture in and prevents later volume change.
7. Bearing value and stability are increased substantially, as measured by unconfined compression, CBR, triaxial, and other tests; this is due to a pronounced cementing action. Cores of lime-stabilized clay-gravel taken from a 14-year-old secondary road in Texas[3] showed an average compressive strength of 480 p.s.i. (the maximum was 600 p.s.i); at age 20 days the average strength was 276 p.s.i. (max. 368 p.s.i.).
8. Freeze-thaw resistance is increased. In Nebraska and other northern states experimental work has been carried out to correct frost boils through stabilizing with lime.[4]

Reactions Occurring

In recent years considerable research has been done on the fundamental reactions occurring between lime and clay minerals. Foremost among this research has been the investigations carried out in the Department of Geology of the University of Illinois under Dr. Ralph E. Grim, eminent clay mineralogist. The research fellow is James L. Eades, former Virginia highway research engineer, who is working under a 3-year research grant of the National Lime Association. Much of the information to follow is based on Mr. Eades' early work on pure clay minerals, which was reported at the 1960 H.R.B. annual meeting,[5] and on subsequent work relating to selected clay-bearing soils.

Three basic reactions are apparent in lime stabilization: base exchange, cementation (partly pozzolanic), and carbonation. The former is an immediate reaction, in which the (larger) calcium (Ca^{++}) ions from the lime replace the smaller weaker H^+, K^+, and Na^+ ions, of the clay. Accompanying base exchange is a lowering of the zeta potential, which causes a flocculation of the particles and a reduction in the thickness of the adsorbed halo-like moisture films surrounding the clay grains. This is reflected in a reduction in plasticity, i.e., if the films are thick, the clay particles are shielded so as not to come in intimate contact with their neighbors, thus causing low stability and a highly plastic state.[6] The above reaction is immediate and requires only a small proportion of the total lime used in stabilization.

Then, under conditions of high pH, the lime begins eating into the clay particles, destroying the structure, and releasing silica and alumina ions. In the presence of moisture, hydration occurs, forming calcium silicate hydrates (similar to tobermorite) and to a lesser extent calcium-silica aluminates (similar to hydrous garnets). Additional cementing compounds are formed if the soil contains natural pozzolans, in which case the lime reacts directly with the available silica, again forming calcium silica hydrates. As with soil cement, the soil aggregates become cemented into a strong, stable mass. Unlike base exchange, the cementing reaction is relatively slow, and still occurring after many years. Any lime not used in the above is available for the third reaction—carbonation. As in the case of hardening of lime mortar, carbonation occurring in the road base is also strength producing.

Eades, in his basic study, worked with four major clay mineral groups—kaolinite, illite, montmorillonite, and Grundite (a mixed-layered chlorite, illite, and montmorillonite), and he concluded that lime increases the bearing value of each mineral. However, the quantity of lime needed to effect stabilization varies with the mineral, as shown in Fig. 1 (In this test 4 x 4.5 in. cylinders were molded at optimum moisture content to the standard AASHO density, sealed to prevent moisture loss, and cured at 140°F for 72 hrs.) Note that kaolin reacted quite readily, with an increase in strength occurring with the first increment if lime; in contrast from 4-6% lime was required before any substantial strength improvement occurred in the other minerals. In the case of the montmorillonites having high exchange capacities, a certain amount of lime was required to drive the Ca^{++} ion into and onto the clay. Only after the clay had changed to a Ca^{++} variety did it develop strength. This explains why the Wyoming montmorillonite, which is a Na^+ clay, requires more lime to become wholly Ca^{++} saturated than the Mississippi samples which carry a little Ca^{++}, and thus exhibits a longer lag in strength development. Grundite also is slow in gaining strength, due to carrying K^+ ions, which must be replaced by Ca^{++}. In the case of the illite, the initial reduction in strength is explained by Eades by the presence of sulfate ions caused by the oxidation of pyrite. The sulfate ions react

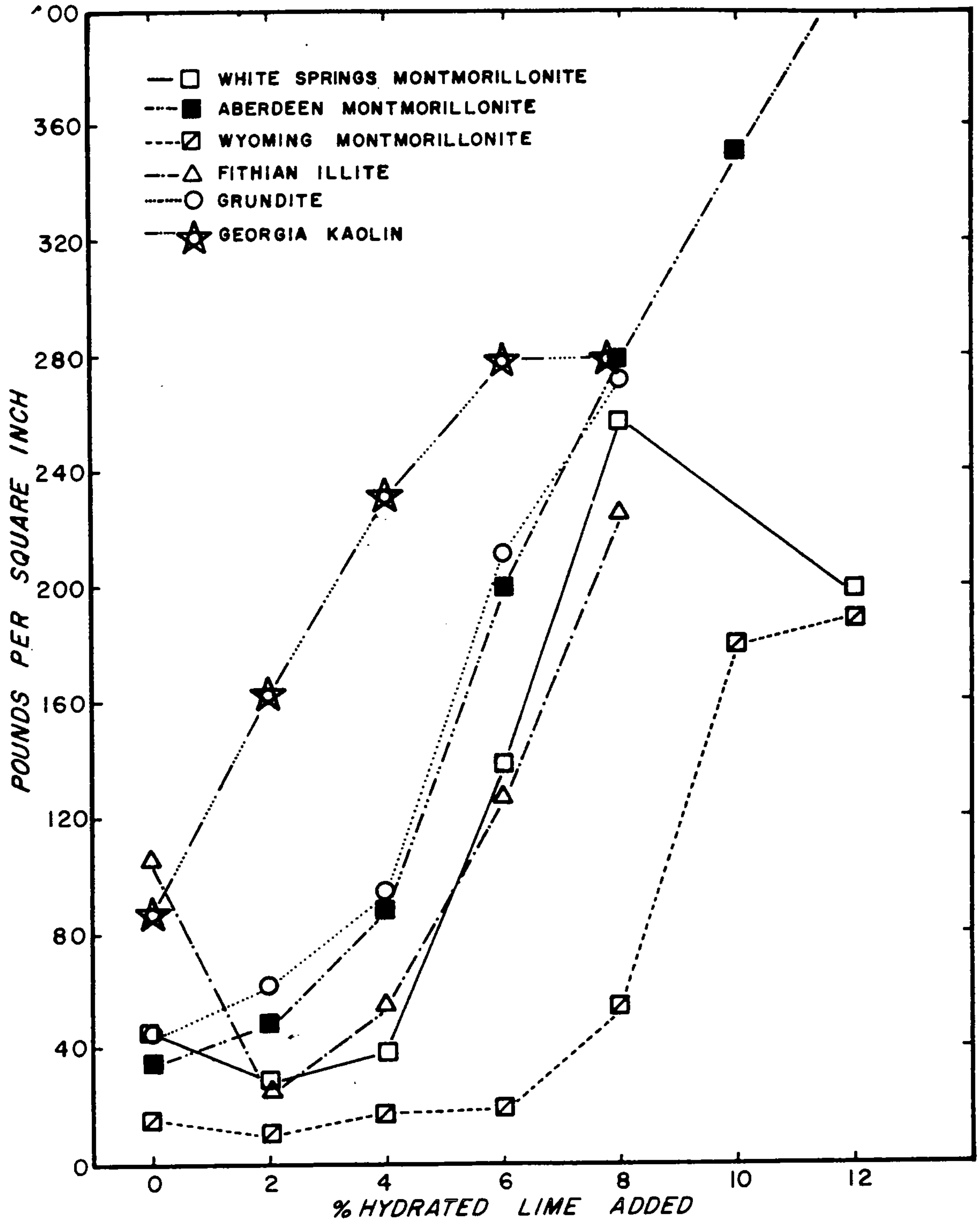


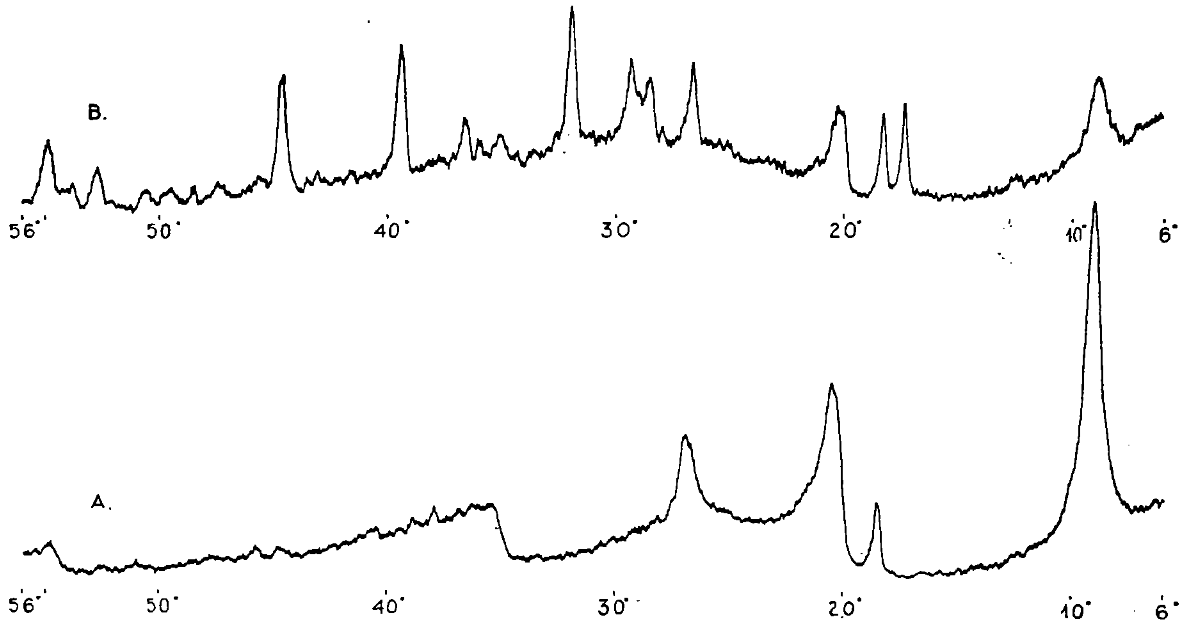
Figure 1. Compressive strengths of hydrated lime-treated samples

first with the lime and it is not until the $SO_4^{=}$ ions are satisfied that the clay becomes Ca^{++} saturated and the strength increases.

An indication of the cementing reaction can be seen in Figs. 2, 3, 4, and 5, showing X-ray diffraction patterns of lime-treated Georgia Kaolin, Halloysite, Grundite, and Aberdeen (Ca^{++}) montmorillonite, respectively. The spectrometer tracings were obtained with a North American Philips Scintillation-Counter with a Pulse Height Analyzer. Powder specimens were prepared to minimize orientation, enabling both basal and prism reflections to be recorded.

Figure 2 gives the tracings for the hydrated lime used, Georgia kaolin untreated, Georgia kaolin containing 6% lime cured for 3 days at 140°F. and Georgia kaolin containing 20% lime cured for 30 days at 140°F. Note, first of all, that the peaks for lime are missing in the C and D tracings. Secondly, that the kaolin, which is well crystallized, gives a diffraction pattern of kaolinite, with sharp basal and prism reflections. As the kaolin is treated with small percentages of lime, the prism reflections lose their sharpness and intensity, whereas the basal reflections are not changed in sharpness. Eades infers that this mineral, which is usually considered quite stable, was attacked at the edges and within the two-layer silicate sheets, thereby causing a weakening of the prism reflections. That is, the initial reaction is not

Figure 2. X-ray diagrams of lime-treated Georgia Kaolin



A-UNTREATED HALLOYSITE
B-HALLOYSITE + 20% HYDRATED LIME($Ca(OH)_2$) CURED 60 DAYS AT 140° F.

Figure 3. X-ray diagrams of lime-treated Halloysite

merely one of spreading apart the silicate layers, but an attack on the basic structure. As the lime content and curing period were increased, there also appeared to be a reduction of the basal reflection, and most important, an increase in the percentage of new minerals as indicated by non-kaolinite diffraction lines. It is significant that new crystalline phases appear about immediately with the treatment of kaolinite with lime.

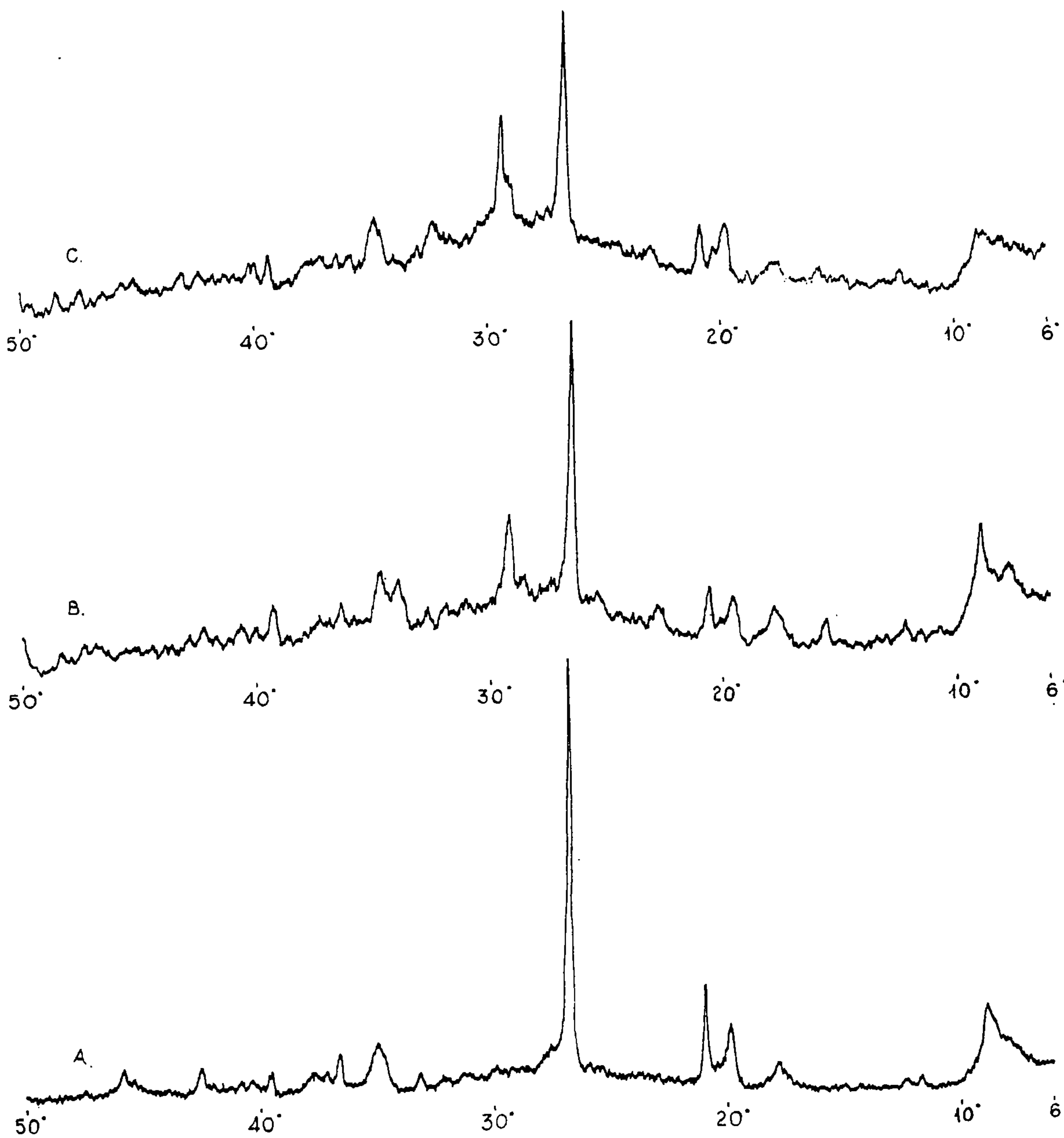
Halloysite, shown in Figure 3, which is the black sheep of the kaolinite family, has the basic kaolinite structure but contains layered water in the structure. This mineral gave the same reflections after it was treated with 20% lime as the Georgia kaolin.

The Fithian illite (not illustrated herein), which gave very low compressive strength until more than 4% lime had been added, did not react as quickly as the kaolinite. That is, diffraction patterns for the 20% lime series for 3 and 6 day curing periods still contained reflections for hydrated lime. The same new minerals, which occurred with the kaolinites, was present in diffraction diagram for the 20% lime and 3-day curing period, but they were in small percentages. The reflections for the illite were reduced in intensity, and the basal reflections were changed from a peak to a broad band, suggesting a general gradual destruction of the illite structure.

Figure 4, dealing with "Grundite," shows that after only 6 days, the calcium tends to separate the mixture into more distinct reflections for illite and montmorillonite. However, as curing was continued, as shown by the diffraction pattern for the 30-day curing period, the clay mineral structures were gradually lost. The treated sample did not give as many reflections for new minerals as the illite. Reflections for lime were present for the 3 and 6 day curing periods but were not present on the patterns for the 15 day period. It appears that lime reacts much more slowly with the illites and mixed-layered materials than it does with kaolinite.

Figure 5, illustrating the reaction of Aberdeen montmorillonite and lime, well represents the reaction of lime with all three montmorillonites Eades tested, since the diffraction data were the same for all of them. The diffraction patterns indicate a substantial breakdown of the structure after lime is in contact with the montmorillonite for prolonged periods. Diffraction data not shown in Figure 5 indicate that small percentages of lime do not affect the structure, and as a matter of fact, the basal reflections seemed to be intensified. This probably is to be correlated with adsorption of Ca^{++} between the silicate sheets replacing other cations. Only a few reflections were encountered that had not been present in the untreated state, showing a scant formation of new crystalline reaction products.

According to Eades, it is impossible to be certain of the identity of the

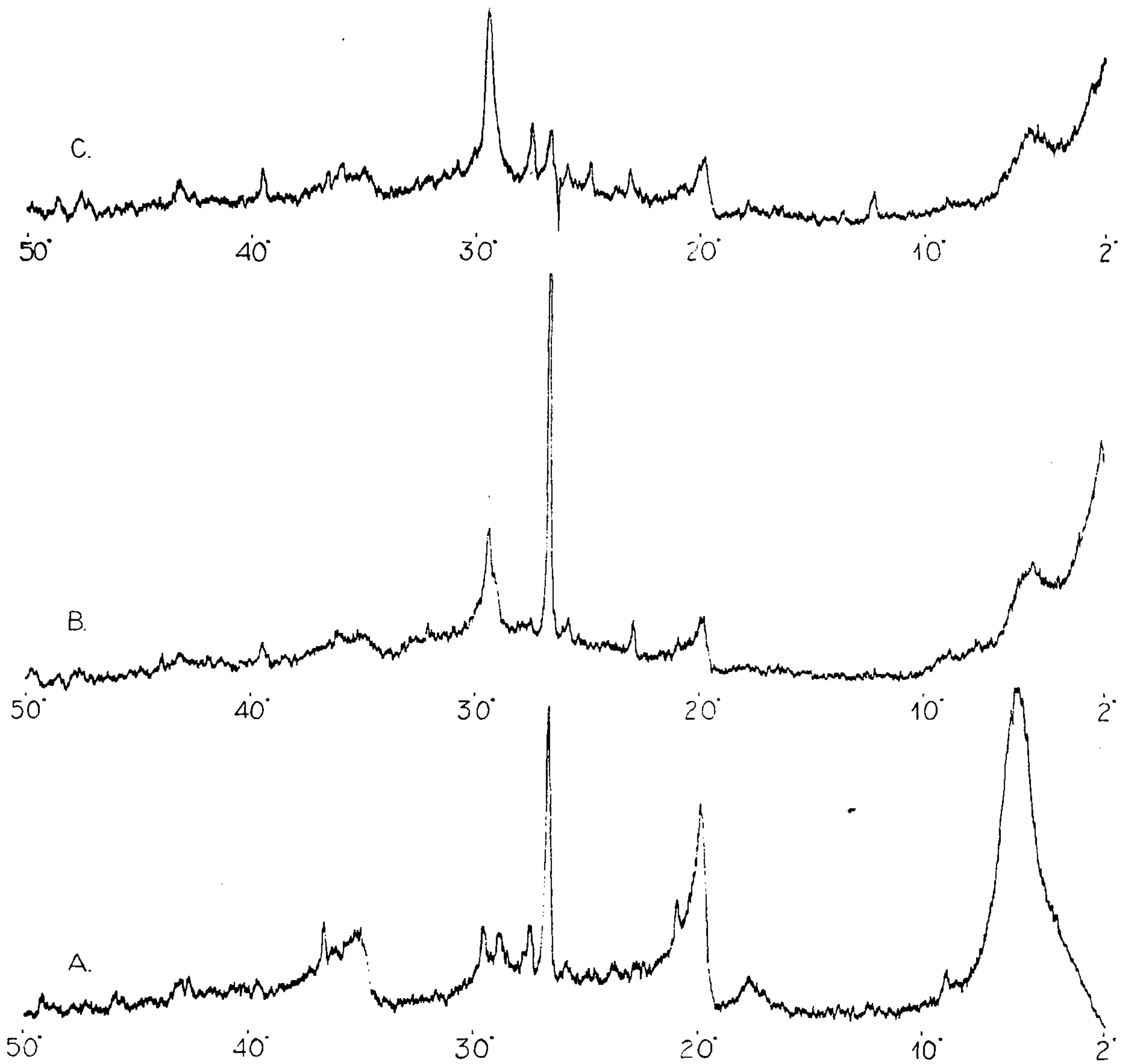


A-UNTREATED GRUNDITE

B-GRUNDITE + 20% HYDRATED LIME ($Ca(OH)_2$) CURED FOR 6 DAYS AT 140°F.

C-GRUNDITE + 20% HYDRATED LIME ($Ca(OH)_2$) CURED FOR 30 DAYS AT 140°F.

Figure 4. X-ray diagrams of lime-treated 'Grundite'



A- UNTREATED ABERDEEN MONTMORILLONITE

B- ABERDEEN + 20% HYDRATED LIME ($Ca(OH)_2$) CURED FOR 30 DAYS AT 140°F.

C- ABERDEEN + 20% HYDRATED LIME ($Ca(OH)_2$) CURED FOR 60 DAYS AT 140°F.

Figure 5. X-ray diagrams of lime-treated Aberdeen Montmorillonite

new crystalline reaction products, but the data give every indication that the new minerals are hydrous calcium silicates, similar to tobermorite. The new reaction products for kaolinite give essentially the same X-ray diagram characteristics as those given for the low temperature calcium silicate hydrate and dicalcium silicate hydrates which are formed during the hydration of cement. Since there are a number of intermediate products in the calcium silicate hydrate system, it is believed that several different compounds may exist at one time until equilibrium between the silica, which is removed from the structure of the clays, and the excess lime is reached.

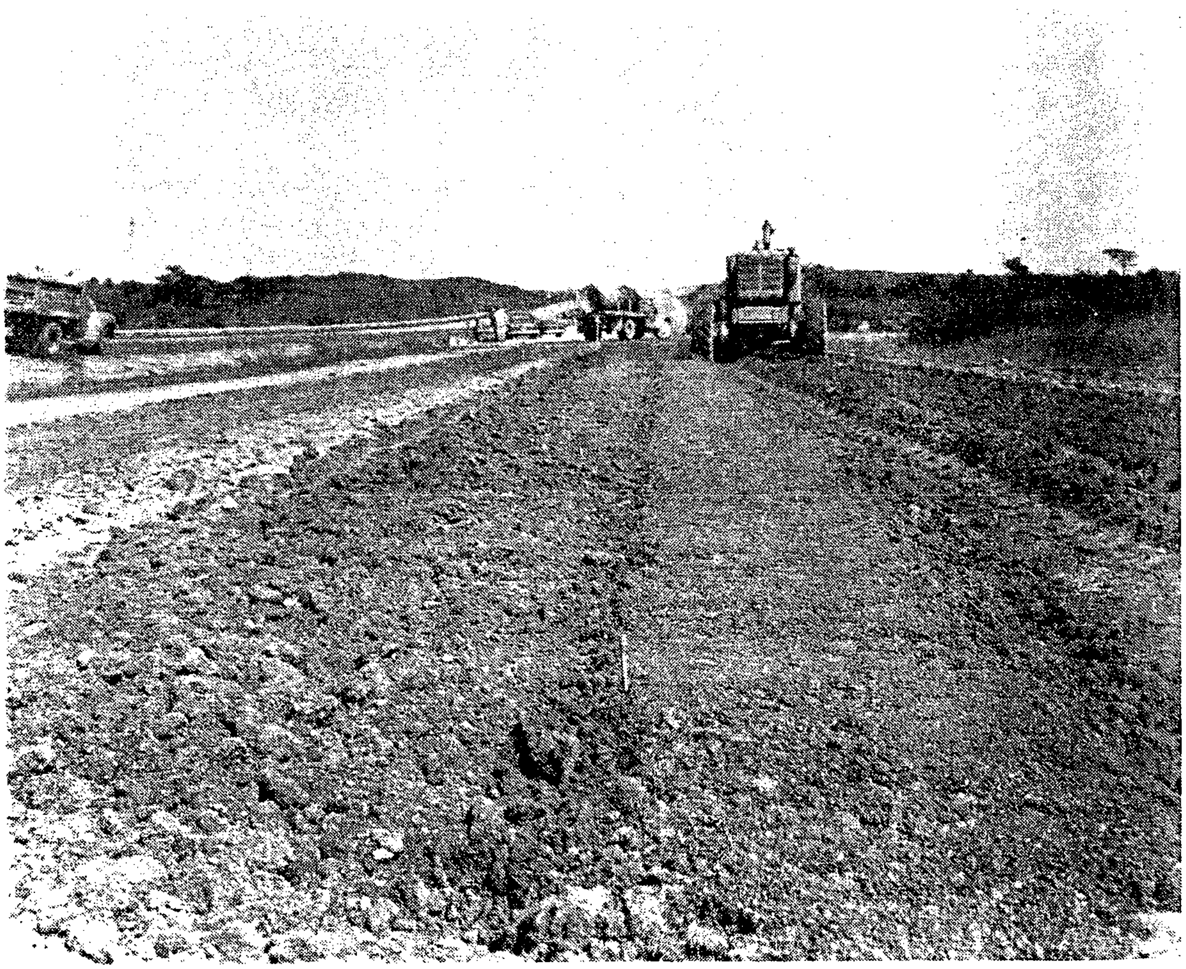
For the illites, the diffraction data indicates only scant formation of new crystalline phases. It suggests, however, that the phases are the same as for the kaolinite.

The X-ray data for the montmorillonites demonstrates there is a destruction of the mineral structure with little formation of new minerals. However, the compressive strength values of the lime-treated clays seem to indifor new minerals, there is a possibility that calcium silicate hydrate gels are formed which are not crystalline, i.e., the ions are arranged so as to not give a repeating pattern and therefore cannot be picked up by X-ray. These gels could just as readily interlock the particles together and give the added strength without having a crystalline structure.

Carbonation was mentioned earlier as a third important reaction occurring in lime stabilization. This is best illustrated in a recent study Eades* made of several 4-yr. old road cores obtained from a lime-stabilized micaceous clayey-silt subgrade in southern Virginia. The native soil, which was predominantly kaolinitic, contained no calcium carbonate; it was stabilized with 5% lime to a depth of 6 in. The overlying pavement consisted of a 6-in. soil-aggregate base course and a 2-in. hot mix surface. Indication of the strength gain after stabilization can be seen as follows: raw soil, average CBR of 3; after stabilizing, CBR at 1 day, 15; at 30 days, 38; and at 1 year, 80. Chemical analysis of the 4 yr. cores indicated the presence of $2\frac{1}{2}\%$ $CaCO_3$ cement surrounding the soil particles, which could only have originated from the hydrated lime. X-ray patterns also showed an indeterminate amount of calcium silicate present, which was not found in the native soil.

Some engineers have questioned the permanency of lime stabilization, feeling that if it only involved flocculation and base exchange, the reaction could be reversed and the stabilization benefits nullified. However, based on the definite development of new cementing compounds accompanying the breakdown of the clay structure and pozzolanic action, it seems certain that the changes are relatively permanent.

* Personal communication.

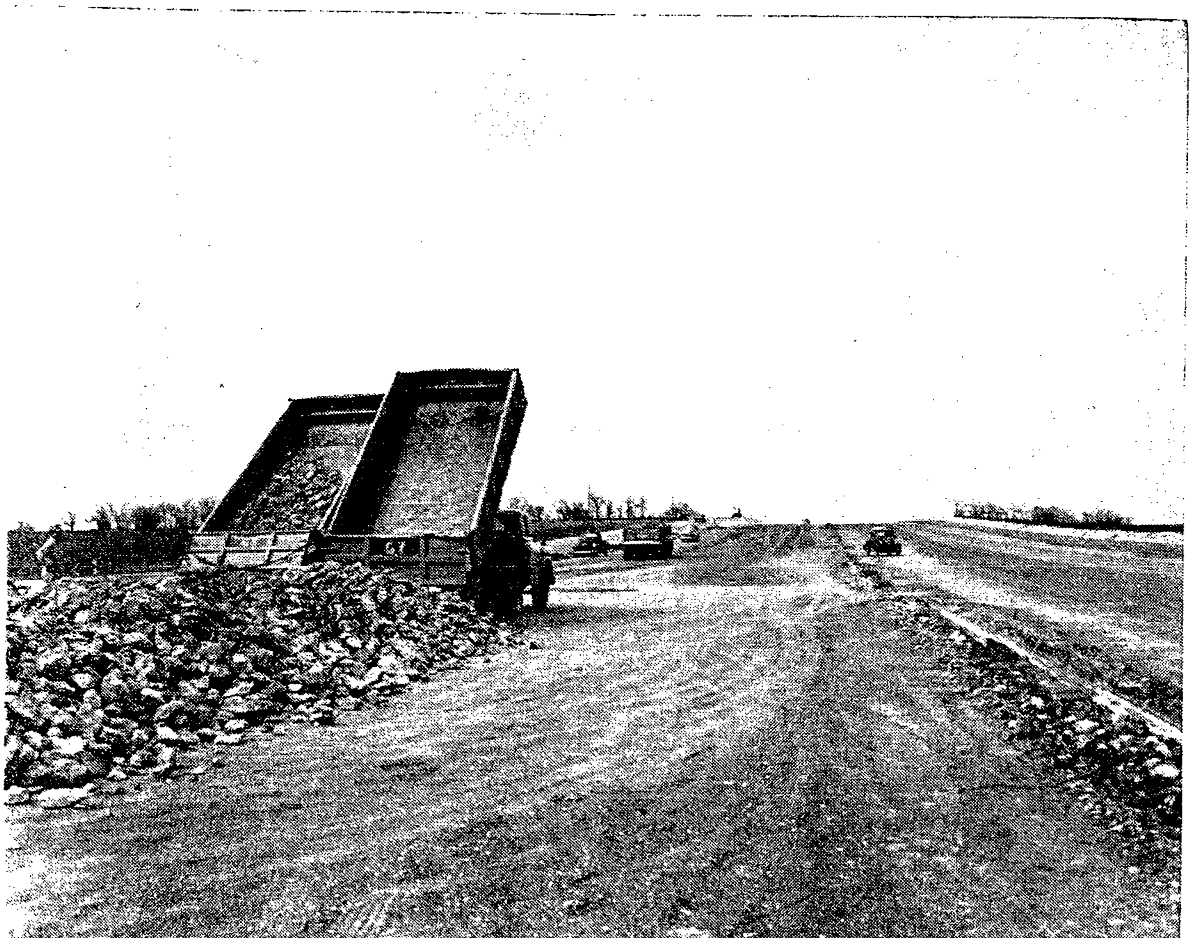


Pulverizing action of lime.

Bryan, Tex. warehouse foundation job. Placing 50 lb. hydrated lime sacks. Wet condition of clay subgrade following draining of pond indicated by tractor ruts. Photo taken about 10:00 A.M. At 1:00 P.M. same day, mixing with disc harrow started.

Bryan, Texas warehouse foundation job. Condition of subgrade after mixing. Photo taken at 5:00 P.M., 4 hrs. after mixing started. Note how clay has dried out.

Bergstrom AFB job, Austin, Tex. (1957). Men are standing on 30-ft. wide stabilized test strip. Tracks made by crawler tractor 3 days after 8 in. rain. Test strip is firm, surrounding clay still wet, as shown by ruts.



Texas Interstate job near Belton, 1957 (I.H. 35). Dumping foundation stone on lime-established subgrade 30 hrs. after 3 in. rain stopped. Subgrade has shed water readily, and was firm enough to hold up 25-ton trucks.

Texas Interstate job near Temple, 1958 (I.H. 35). Ruts made by Plymouth next day after rain. Lime established subgrade (to rear) was barely dented, whereas shoulder was badly rutted.

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