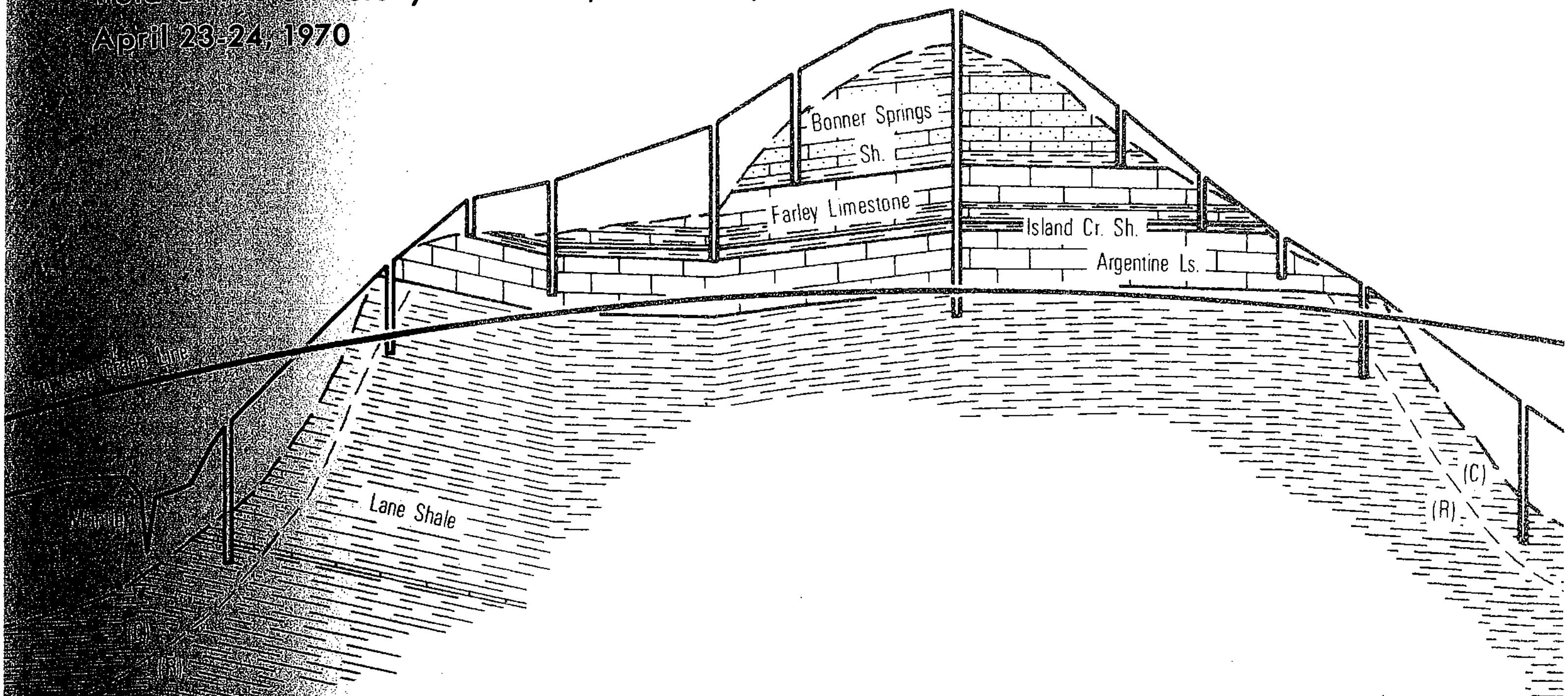
Proceedings of the Symposium
held at The University of Kansas, Lawrence, Kansas
April 23-24, 1970



TWENTY-FIRST ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

PROCEEDINGS OF THE 21ST ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

HASHINGER HALL

The University of Kansas
Lawrence, Kansas

April 23-24, 1970

PRESENTED BY

State Highway Commission of Kansas and
State Geological Survey

SPONSORED BY

University Extension

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PREFACE

The 21st Annual Highway Geology Symposium was held at the University of Kansas in Lawrence, Kansas, on April 23 and 24, 1970. A meeting of the National Steering Committee was held at the Holiday Inn the evening of April 22. Thursday's program consisted of a field trip to the Kansas City area of northeastern Kansas.

The field trip encompassed a variety of highway subjects, including observations of both old and new backslope design and construction, underdrain design, sidehill construction, flexible and rigid pavements, landslides, stream erosion problems, subgrading, special embankment construction, undermining, a proposed large interchange in an old extensive trash dump, and bridge design and construction over the Turkey Creek tunnel in Kansas City, Kansas.

Approximately one and a half hours in the morning were spent in touring an underground limestone mine that contains refrigerated storage space, manufacturing plants, and an active limestone mine.

The weather was ideal throughout the field trip. The lunch stop was held in an abandoned gravel pit where the participants ate their box lunches while basking in the warm Spring sunshine.

The annual banquet was held on Thursday evening, preceded by a social hour sponsored by Armco Steel Corporation, Topeka, Kansas. Virgil Burgat was Master-of-Ceremonies at the banquet and Dr. Wakefield Dort, Jr., was speaker for the evening. Professor Dort showed color slides and described enthusiastically his "Recent Experiences In Antarctica." A memorable event of the evening was presentation of the first three Highway Geology Symposium Medallions. The medallions signify meritorious service to the Highway Geology Symposium or to the geological profession in general. Carter Dodson, Chairman of the National Steering Committee, presented the awards to W. T. Parrott, North Carolina Highway Department; Dr. Paul H. Price, West Virginia Geological and Economic Survey, and Hugh D. Chase, Georgia State Highway Department.

Because of the recent burning of The University of Kansas Student Union, Friday's technical session was held in the dining room of Hashinger Hall. Welcoming speeches were given by Francis Heller, Dean of Faculties at the University of Kansas, and by John D. McNeal, State Highway Engineer of Kansas. Mr. McNeal is a former member of the National Steering Committee and a long-time supporter of the Highway Geology Symposium.

The morning and afternoon sessions were presided over by Frank C. Foley and Walter F. Fredericksen, respectively. Eleven technical papers were presented during the day. Small equipment, such as drilling tools and geophysical equipment, was displayed in the same room with the technical session. Large equipment was displayed at the Holiday Inn.

We wish to thank all of the 175 people who attended and participated in the two-day session, with special thanks extended to the speakers. Virgil Burgat spent many long hours working to make the Symposium a success, and Carter Dodson deserves recognition for having done an excellent job of guiding the Highway Geology Symposium for the last several years as chairman of the National Steering Committee. Richard Treece, University Extension, was an efficient performer in spite of many difficulties brought about by the burning of the Student Union. We acknowledge his great assistance to us.

Coordinators of the <u>Proceedings</u>

W. T. Parrott

Was instrumental in founding HGS while serving as Chief Geologist of Virginia Highway Department. Mr. Parrott is presently employed as Materials Engineer with the North Carolina Highway Commission.

Dr. Paul Price

Served as Director of West Virginia Geological Survey and Professor of Geology at University of West Virginia for many years. Dr. Price was on the Highway Geology Symposium Steering Committee for several consecutive terms and was very influential in the success of HGS. Dr. Price is now retired.

Hugh Chase

Employed as Geologist in Georgia Highway Department for many years and served a long term as Secretary of HGS. Mr. Chase is now retired.

We gratefully acknowledge the assistance of the following persons in preparing these Proceedings for publication, and we extend to them our thanks: Miss Cheryl Morgison and Mrs. Donna Saile, who typed the manuscript, State Geological Survey of Kansas; Mrs. Elizabeth Kolars, who designed the cover, Mr. Robert Kibby and Mr. Dale Mahan, who prepared the plates for printing, Location and Design Concepts Department, State Highway Commission of Kansas.

The Coordinators

CONTENTS

HIGHWAY PROBLEMS AND THE GEOLOGY OF KANSAS

Frank W. Wilson*

The purpose of this paper is to review briefly the geology of Kansas and to outline some of the problems of highway construction and maintenance which are related to it.

The particular examples were chosen as being generally illustrative of major, typical, or interesting problems in the outcrop area of each geologic system. It is beyond the scope of this paper to attempt to discuss all of the geology-related highway problems in all parts of the state.

ACKNOWLEDGEMENTS

I gratefully acknowledge the use of reports and other material from a number of my former colleagues in the Geology Section of the Kansas Highway Commission. Virgil A. Burgat, Chief Geologist, and Walter F. Fredericksen, Eastern Regional Geologist, were especially helpful in providing reports and information concerning projects with which I had not been personally involved. Gary F. Stewart, of the Kansas Geological Survey, reviewed the manuscript and made many helpful suggestions. Allison L. Hornbaker, Kansas Geological Survey, provided statistics concerning mined areas in Kansas.

INTRODUCTION

Bedrock units exposed at the surface or present in the shallow subsurface in Kansas range in age from Mississippian through Tertiary. Quaternary sediments are widespread throughout the state, occurring mainly as unconsolidated aeolian, glacial, or alluvial mantle over Paleozoic or Mesozoic bedrock.

In the following sections, the major rock units and the problems associated with them are described in ascending order, from

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the geologically oldest to youngest. Units which are not widespread or which cause no major engineering problems have been omitted or described only briefly. A selected bibliography of the geology of Kansas is appended.

MISSISSIPPIAN SYSTEM

Mississippian rocks, predominantly cherty limestones and bedded cherts, crop out in a very small area at the extreme southeastern corner of the state.

Mississippian bedrock is mantled by a residual cherty rubble which approaches 25 feet in thickness in the uplands. Exposures are sparse except along major streams. The tendency of the rock to weather more rapidly along or in the vicinity of joints results in an extremely irregular or pinnacled mantle-bedrock contact.

Power-auger or core-drill soundings into both the cherty regolith and bedrock are very difficult, and wear on drill-bits and drilling equipment is extremely rapid. Shallow seismic and electrical resistivity methods have been used with some success, mainly to confirm that bedrock was considerably below the depth of proposed excavation. Neither of the geophysical methods is effective in some areas because of the extreme porosity of the cherty mantle. Electrical resistivities beyond the range of the instrument are often encountered during dry weather and sledgehammer-type seismic methods do not produce sufficient energy to penetrate the rubbly mantle.

Classification of Excavation

The classification of excavation is complicated by the fact that the standard specifications for rock excavation in Kansas are based largely upon the degree of weathering in bedrock. This criterion works reasonably well in most other Kansas strata but fails when applied to the Mississippian. The cherty strata several feet above true bedrock are locked in place although the enclosing limestone has been completely dissolved and the chert beds themselves are fractured and altered. If standard specifications were strictly applied, this material would be classed as common excavation, even though ripping or blasting is required to loosen it sufficiently to allow its removal. In actual practice, any cherty material that retains part of the original bedding is classified as rock excavation.

Similar difficulties are encountered in the material classed as common excavation. The thick mantle is composed of as much as 80 percent angular chert rubble of cobble or boulder size. Although it is properly classified as common excavation according to the standard specifications, the material is probably more difficult to excavate and handle than materials classed as rock excavation in western Kansas. Failing to recognize these special conditions, despite the fact that they were pointed out in project reports, has caused considerable difficulty for several contractors who had no previous experience in the Mississippian outcrop area.

Undermining

The Tri-State mining district of southeastern Kansas, southwestern Missouri, and northeastern Oklahoma was once the nation's leading producer of zinc and the fourth ranking producer of lead. Extensive mining of the low-grade ores has left thousands of acres undermined. In the Kansas part of the Tri-State mining district alone, more than 8,000 acres are known to be underlain by abandoned mines. This figure does not include the very early mines in the Galena-Empire district for which records are unreliable or unavailable.

Mining began in the Galena area in the late 1890's and reached a peak in the early 1900's. The easily accessible shallow ores were depleted by the end of World War I, except for sporadic "gouging" or illegal mining during periods of higher ore prices. Most of the mining was done by small companies or by individuals and was entirely unregulated. Some of the mines within the city limits of Galena were of city-lot size. Few mine surveys were made and although mining under adjacent private property was likely to be fatal to the miners if discovered by the land owner, present evidence suggests that encroachment under public property or roadways was quite common.

Mining methods were aimed at removing all of the available ore, and in many instances the resulting cavity was the size and shape of the former ore body. Pillars were not used unless absolutely necessary and were often shaved or removed by later illegal mining. Unsupported rooms more than 100 feet in height and several hundred feet in width and length were common.

Because the ore was of relatively low grade it was crushed and concentrated on the site and only the "concentrate" was hauled to the smelter. The resulting piles of chert tailings or "chat" all but obscure the ground surface in the mined areas. Literally thousands of chat piles, shafts, and cave-ins can be seen on older aerial photographs of the Galena area.

An extensive engineering geology investigation of abandoned workings in the Galena area was made during 1959 and 1960 in connection with reconstruction and widening of U. S. Highway 66. Most of the mines are at depths of less than 100 feet and have a floor-to-ceiling height averaging about 50 feet. During the course of this investigation a partially flooded mine was mapped underground by the author and Bill Jones, a colleague in the Kansas Highway Commission. The roadway over the mine had undergone minor subsidence for many years. The maintenance foreman told us that numerous truckloads of hot-mix had been applied in an attempt to bring the settling area up to grade. While mapping the mine underground we discovered that part of the roof had collapsed and the patch material was resting on a pile of roof rubble which was gradually spreading under the load. The roof adjacent to this area consisted of only about 6 feet of weathered and leached rock. The entire room was subsequently collapsed by blasting and was backfilled with waste rock.

A deeper and completely flooded part of this mine extended under the roadway in a nearby area. The cavity was known to have a floor-to-ceiling height of about 80 feet but its lateral extent was not accurately known. The mine connected with a large system of flooded workings which were believed to extend under a part of the city of Galena. Because surface runoff from an area of several square miles drained into the mines, it was not feasible to lower the water level sufficiently to collapse the roof and backfill the hole by conventional methods. Blasting the roof down into the flooded opening was rejected because of the possible hydraulic-ram effect in the unknown reaches of the tunnel system. Because surface and subsurface drainage from the upstream area was channelled through the opening beneath the roadway, a strong current was present at the bottoms of shafts which extended into the cavity. Grouting was not considered feasible and without restraining bulkheads to keep the material from being washed away, the sluicing of fine-grained material into the cavity through drill holes or shafts was considered to be doubtfully effective.

After considerable deliberation, a decision was made to span the mine with a heavily reinforced concrete slab which would act as a bridge and decrease the loading on the mine roof. The slab was designed to support itself and the traffic load in the event that the mine roof collapsed without warning.

In addition to the work described above, a number of vertical shafts were excavated to bedrock and capped with reinforced concrete. An at-grade crossing was substituted for a proposed railroad overpass when core-drill soundings encountered collapsed workings and mine timbers at depths approaching 90 feet at one abutment site.

More recently, a section of U. S. 166 was located over deeper mines in an area west of Baxter Springs, Kansas. The mines are about 275 feet deep and are overlain by approximately 200 feet of competent Mississippian roof rock and 75 feet of incompetent Pennsylvanian shale and cherty mantle. The part of the mines under the roadway is considered to be safe from collapse. A number of old shafts and drill-holes adjacent to the proposed roadway were excavated to bedrock and plugged with concrete to prevent their enlargement by caving of the loose mantle into the underground workings. Cone-shaped holes ranging from 150 to 200 feet in diameter and 75 to 100 feet in depth have developed in this area due to the collapse of wooden cribbing which has allowed the loose mantle to flow through the vertical shafts into the mines.

Further westward in the Baxter Springs-Treece district, existing U. S. Highway 69 is undermined for a distance of nearly a mile by extensive workings at a depth of about 500 feet. The floor-to-ceiling height in this mine is 180 feet. Although the mine was in operation only a few years ago and is considered to be safe from collapse, cave-ins over similar workings have occurred recently in Picher, Oklahoma.

Considering the size and extent of some of the mines in this district, many geologists and engineers wonder what damage might occur if the area is ever subjected to an earthquake of even moderate force.

PENNSYLVANIAN SYSTEM

Outcropping Pennsylvanian rocks in Kansas can be divided into two general categories on the basis of their engineering characteristics. The lowermost part, the Cherokee Group, is made up predominantly of non-durable sandstones, shales, coals and underclays. The upper part of the Pennsylvanian is characterized by alternating units of durable and non-durable sediments, mainly limestones and shales.

CHEROKEE GROUP

The Cherokee outcrop is marked by subdued topography and low-gradient surface drainage. Heavy clay soils and relatively high ground-water levels are typical of much of the area.

Undermining

The Cherokee Group contains at least 14 coals, 12 of which have been mined and nine of which are of commercial importance. Underground mining no longer is practiced in Kansas but it was once the principal means of coal extraction. An estimated 60,000 acres, mainly in Cherokee and Crawford counties, are underlain by abandoned underground workings. Most of these mines were operated in the 1920's and 1930's. Few maps were made and the mining operations were essentially unregulated. Although mining under an existing roadway was illegal (except by permit to gain access to adjoining property) the practice was widespread in certain areas.

A proposed improvement project on Highway K-7 in Cherokee County was abandoned when the engineering geology investigation found that a large part of the roadway was undermined. The geologist making the investigation was told by landowners that during the 1930's one small mine had operated for 4 years in an area where the only remaining unmined coal was under the highway right-of-way. Subsequent investigation confirmed that the roadway was indeed undermined for a distance of nearly a mile. Abandoned underground workings are so extensive in that particular coalfield that no surface drainage system exists and all roadside ditches and minor streams drain into the mines.

Surface Mining

Commercial coal mining at the present time is carried out entirely by surface stripping. An estimated 40,000 acres have been strip-mined in Crawford and Cherokee counties. In some large areas of southeastern Kansas, section-line roads or major highways are the only undisturbed ground. As these routes become obsolete, new alignments will necessarily be located on strip-mined ground. Some of the shovels now in use can dig as deep as 70 feet and move as much as 90 cubic yards of rock per bite. Spoil piles created by these "monsters" are almost 100 feet high. Because of the extreme porosity of the castings and the relative impermeability of the substrata, a free water table ordinarily exists only a few feet below the elevation of the former land surface. The problems and expense of attempting to build major highways in this unstable material are considerable.

Construction of major highways in southeastern Kansas during the past decade has been limited mainly to reconstruction or widening of existing routes. Problems with mines have been relatively minor. There is considerable political agitation at present for a high-speed, multiple-lane turnpike or freeway southward from Kansas City along the eastern border of Kansas. Long segments of this proposed route would be located in strip-mined ground or above areas that are extensively undermined by coal mines or lead and zinc mines. Techniques for investigating and dealing with these problems should be developed now, in anticipation of their eventual need.

POST-CHEROKEE PENNSYLVANIAN ROCKS

Middle Pennsylvanian rocks are characterized by cyclicly deposited limestone, shale, sandstone, and minor amounts of coal.

Slopes

Differential weathering of interbedded durable and non-durable strata, much of it in thin beds, makes design and construction of backslopes extremely difficult. Many of the shale units are too thin to be cut separately to a suitably flat slope. In the past, economics of engineering and the limitations of construction equipment required that such thinly interbedded, alternating soft and hard strata be cut on average slopes which were suitable for the durable layers. Unfortunately, the interbedded soft shales weathered away and caused undercutting of the durable rocks, which failed almost as rapidly as the weaker strata.

Need for Research on Design of Slopes

Although the development of more powerful tractors makes ripping of these beds possible, and drilling equipment now exists which is easily capable of drilling angled blast-holes, the construction of flatter and more stable slopes in interbedded hard and soft rocks still is not widely practiced in Kansas. The reasons for this are in part based on operations philosophy. Design engineers, aware of increasingly high right-of-way costs, want to keep the backslopes as steep as possible without risking wholesale failure. Construction engineers and contractors favor steep slopes because they are simpler and cheaper to construct. These practices will continue until maintenance engineers, in self defense, produce data to show that long-term costs of cleaning ditches and repairing minor slides exceed the presumed savings in costs of right-of-way and construction. Research on this problem is long overdue.

Studies on design and construction of slopes in rock are also needed. Although line-drilling and pre-splitting are now used in special instances, their use is far from routine. Drilling of inclined blast holes has been practiced for 30 years in Europe and is being increasingly utilized in quarrying operations in the

United States. The technique is applicable to highway construction and seems to be capable of producing stable rock slopes at a considerable reduction in costs of blasting and excavation. Adoption of the technique is presently resisted by contractors who do not have suitable equipment and by powder companies who are not interested in developing a technique to reduce the use of explosives. An evaluation of the technique on a full-scale research project should be implemented soon.

The need for special studies on the long-term behavior of clay and shale slopes is indicated by a rash of delayed failures which have occurred in embankment slopes in eastern Kansas. Most of the slides have taken place rather suddenly in railway overpass embankments that were well constructed and that were stable for about 10 years after construction. At least seven failures of this type are known and a comprehensive survey would probably reveal more. The almost-exclusive involvement of railroad structures suggests that vibration is a causal factor, but the time-related deterioration of the embankment material probably is caused by more complex processes. Several of these failures have been extremely troublesome and difficult to correct. Considerable research is needed to determine their causes so that they can be prevented and also to find effective means of correcting them after they occur.

Ground Water

Vadose water migrates down dip along the upper surfaces of less permeable zones in the alternating limestone-shale sequence of typical post-Cherokee Pennsylvanian rocks. Although the quantity of water moving in any particular zone is not large, it is sufficient to cause failure of the road surface if it is allowed to saturate the subgrade. Where the engineering geology investigation indicates that the dip of the beds is toward the proposed grade line, a system of pipe or blanket underdrains is designed to intercept the water before it reaches the subgrade.

Construction Materials

Most of the Pennsylvanian limestones in the eastern part of the outcrop belt are suitable sources of highway construction material and are sufficiently plentiful to allow quarry sites to be developed at or close to most construction projects.

Dolomitic limestones from certain reef-like units in southeastern Kansas have recently been rejected for use in concrete aggregates, but they usually are suitable for other purposes.

Many of the thick limestone beds are developed as sources of portland cement and millions of barrels of cement are produced annually from plants operating in eastern Kansas.

Most limestone quarries use the open-cut method, but limestone is mined underground in northeastern Kansas where overburden thicknesses are excessive because of the rolling topography and thick glacial cover. Many of these mines have been converted to or are being developed for underground storage or other industrial purposes.

Roof failure and destructive collapse of parts of several limestone mines point out the need for research into the proper design and construction of such facilities, and the need for regulative legislation to assure the long-term safety and stability of the mined-out areas. Geologists of the Kansas Highway Commission recently investigated and designed stabilization measures for a mine under a newly constructed section of Highway I-635 in Kansas City, Kansas.

PERMIAN SYSTEM

The lower part of the Permian is similar to the post-Cherokee Pennslyvanian described above and the engineering problems and characteristics are the same.

Younger Permian rocks show evidence of an increasingly arid climate during the time of their deposition. The shales are characterized by alternating hues of red, green, and purple. Thin gypsum or anhydrite beds are present in some of the formations.

Some Permian limestones contain numerous beds of chert which weather to produce a thin flint-bearing soil that is unsuitable for cultivation. The upland outcrop of this group of rocks includes the Flint Hills, one of the few areas of true prairie that remain in the United States.

The cherty beds become less numerous and limestone units become thinner and increasingly chalky in the western part of the outcrop. Neither the cherty limestones nor the soft, chalky limestones are suitable for use as concrete aggregate.

Solution Subsidence

A 325-foot-thick salt bed is present in the subsurface in the Wellington Formation of central Kansas. The bed has been mined commercially near the cities of Hutchinson and Lyons. A part of the Hutchinson mine, which has been developed at depths of about 700 feet, has been converted for the storage of important national documents and records. The Lyons mine and nearby areas of unmined salt are being actively considered by the Atomic Energy Commission as a national disposal site for radioactive wastes.

Sinkholes, undrained depressions, and other subsidence features are relatively common along the solution front of the Hutchinson salt where it occurs in the shallow subsurface in east-central Kansas. Kansas Highway Commission geologists were surprised, however, to find subsidence affecting a newly constructed section of Highway I-70 in western Russell County, nearly 100 miles west of the solution front, where the top of the Hutchinson salt is at a depth of about 1,300 feet.

Two areas of subsidence about one-half mile apart were discovered. In addition to affecting the roadway, an overpass bridge near the eastern site also was settling. Both areas of subsidence were near abandoned oil wells. A single well at the west site had been drilled in 1936 and was abandoned in 1957. Two wells about 50 feet apart at the east site had been drilled in 1937 and were abandoned in 1941 and 1945. Comparison of the original well-head elevations with present elevations showed that the west well had sunk approximately 9 feet and the pair of wells had sunk almost 20 feet. Aerial photographs taken in 1936, 1951, 1956, and 1962 showed that subsidence had begun at the east area in about 1954 and at the west location in about 1957. Further study indicated that during recent years the west location had been sinking at a rate of about one foot per year while the east location had been sinking at about half that rate.

A study of well logs and other subsurface data led to the conclusion that the subsidence resulted from solutioning of the Hutchinson salt by fresh water flowing downward from the Dakota or Cheyenne sandstones either through or alongside the well casings. The dissolved salt was probably being discharged into the deep former producing horizon at approximately 3,000 feet. A series of holes drilled to a marker bed in the shallow subsurface around the east site indicated that a roughly circular area of about 1,000 feet in diameter was subsiding. There was concern that a large cavity might exist at depth and that a catastrophic collapse could occur that would endanger traffic on heavily travelled I-70. To check this, the Highway Commission Geology Section drew up specifications and contracted for the drilling of an exploratory hole at the east site. This test hole, which was drilled near the abandoned wells, lost circulation in the Cheyenne Sandstone Formation at a depth of about 530 feet. After repeated unsuccessful attempts to cement off the lost-circulation zone, drilling was continued with no return of cuttings and with complete loss of drilling fluid at an estimated rate of 400 gallons per minute. Drill water was hauled to the site continuously by 10 water trucks.

Because of the lack of bit cuttings, an accurate drilling time log was kept. An analysis of this log indicated that the strata overlying the salt were of normal thickness but were about 50 feet lower in elevation than they were when the oil wells had been drilled. Approximately 50 feet of upper salt was missing but no cavity existed. Apparently the overlying strata had sagged as the salt was dissolved.

Three courses of action are now being considered: (1) repair the roadway and bridge, if necessary, as sinking continues; (2) attempt to plug the abandoned wells either to keep fresh water from reaching the salt or the solute from escaping into the lower horizons; (3) relocate the highway. Cost of such relocation is estimated at 1.5 to 2 million dollars.

JURASSIC SYSTEM

Jurassic rocks crop out in only a very small area of southwestern Kansas and are not important in the engineering geology of highways.

CRETACEOUS SYSTEM

Rocks of Cretaceous age may be divided into four categories on the basis of predominant engineering properties: (1) relatively thick, landslide-prone shales; (2) unpredictably contorted water-bearing sandstones; (3) monotonous sequences of alternating thin chalky limestones and thin chalky shales; and (4) massive chalks.

Landslide-prone Shales

The Kiowa Formation, the Janssen and Terra Cotta Clay members of the Dakota Formation, the Graneros Shale, the Blue Hill Shale Member of the Carlile Formation, and the Pierre Shale may be included in this category.

Prior to the interstate highway program, it was common practice in western Kansas to utilize side-hill alignments to keep excavation at a minimum. Despite attempts to stabilize these locations, failure by sliding often occurred in the unstable horizons mentioned above. Now the standard practice is to cross the outcrops of these units as nearly at right angles as practicable, in order to minimize the probability of sliding. Elaborate benches and subdrainage systems are also used to stabilize the slopes. Expansive shales and clays are removed from the upper part of the subgrade and replaced with more stable material.

Water-bearing Sandstones

Water-bearing sandstone lenses in the Cheyenne and Dakota formations frequently cause problems because of piezometric water levels that develop in the steeply dipping or highly contorted beds. Because of irregular dips and discontinuous bedding, correlation of these beds for long distances is difficult. The Kansas Highway Commission has sustained several lawsuits because road construction drained aquifers that provided the only water supply for adjacent properties.

Interbedded Thin Chalky Limestones and Shales

The various members of the Greenhorn Formation, the Fairport Chalk Member of the Carlile Formation, and the Smoky Hill Chalk Member of the Niobrara are characterized by interbedded chalky limestones and calcareous shales. The Shaly zones contain numerous thin bentonite beds.

Classification of Excavation

The classification of rock excavation in sequences of interbedded thin chalks and shales is determined more-or-less arbitrarily, according to the thickness of individual chalk beds and the relative proportions of chalk and shale. Because this is a matter of interpretation, it has been the subject of litigation between several contractors and the Highway Commission.

Backslopes

Slopes in interbedded thin chalks and shales are subject to the same problems as those described previously for thicker interbedded limestones and shales of the Pennsylvanian and Permian. If the beds are cut on slopes that are too steep for the softer portions of the section, the chalk beds tend to be undercut and to fail at about the same rate as the shales.

Strata dipping toward the roadway may fail as a block by sliding along a thin bentonitic unit. The likelihood of this type of failure can usually be anticipated. Unexpected minor backslope failures sometimes occur because of the intersection of undetected small-displacement faults by backslopes. Faults of this type are rather common in both the interbedded chalks and shales and in the massive chalks of western Kansas.

Ground Water

Lateral water movement in the vadose zone usually is concentrated in the bentonite horizons. Although subdrains in the bentonites ordinarily carry water only for a short time after installation, they seem to be a necessary and effective means of drainage and stabilization.

Massive Chalk

The Fort Hays Limestone Member of the Niobrara Formation is a 50-foot-thick, massively bedded chalk. The Fort Hays Member drills so easily that it is often difficult to determine the contact between the loose chalky mantle and the chalk bedrock.

TERTIARY SYSTEM

Tertiary rocks, mainly arkosic silts, sands, and gravels, occur widely in the upland areas of western Kansas. All of these deposits are included in the Ogallala Formation. Chert gravels present at high topographic levels east of the Flint Hills in eastern and southeastern Kansas are also tentatively assigned to the Tertiary System.

Classification of Excavation

Loosely cemented caliche or "mortar beds" are common in the upper part of the Ogallala Formation of western Kansas. Some of this material is sufficiently firm as to be classed as rock excavation, but cementation is so variable and patchy that it is difficult to show accurately on engineering cross-sections. Because of this fact, the general practice is simply to show the estimated percentage of rock excavation for each cross-section.

Although the calcite-cemented zones drill very softly, they considerably increase the resistance of granular material to penetration by driven pile. Excessive cost-overruns because of pile cut-off led to the use by the Highway Geology Section of an air-driven penetrometer to check the resistance to penetration in granular materials. Because it is relatively mobile and provides a rapid and economical method for supplementing core-drill data, the penetrometer is also utilized routinely in other sediments as well.

Ground Water

Unconsolidated granular material makes up most of the Ogallala Formation in western Kansas. This material lies directly above bedrock in most upland areas. Percolating vadose water tends to follow shallow depressions in the irregular bedrock surface, and where such channels or depressions are cut by backslopes or subgrade, subdrainage or surface diversion is usually necessary.

Construction Materials

Sands and gravels of the Ogallala Formation are important sources of construction aggregate in the eastern part of Kansas. Their probable stratigraphic counterparts, the clay-bound chert gravels of the eastern part of the state, are widely used as surfacing material for county secondary roads.

QUATERNARY SYSTEM

Unconsolidated Pleistocene deposits of various types discontinuously cover the uplands and the major stream valleys in Kansas. A description of all the various aeolian, glacial, fluvial, and lacustrine deposits is beyond the scope of this paper, but some of the major engineering characteristics and problems are described below.

Aeolian Silts

Loess deposits are recognized in the retreatal phases of each of the major periods of glaciation in Kansas. The most widespread units are the Loveland Formation of the Illinoisan Stage and the overlying Peoria Formation of the Wisconsinan. The sequence of grayish-tan Peoria silt underlain by reddish-tan Loveland silt is well known to highway geologists in all parts of Kansas. The contact between the Loveland and Peoria is marked by a prominent fossil soil that formed during the Sangamonian interglacial stage. Leaching of clay and other colloids from the "A" horizon and their concentration in the "B" horizon of the ancient soil is conducive to lateral movement of soil moisture along the contact between the two zones. Although the amount of water is small, it is sufficient to cause softening of the subgrade and failure of light surfacing. Subgrading of the Sangamonian soil is usually recommended where it is considered to be likely to cause maintenance problems.

Backslopes in Loess

The tendency of loess deposits to stand naturally on nearly vertical slopes has been utilized in the design of highway slopes. Not all of these designs have been successful. Even natural slopes in loess are not stable for long periods of time and they tend to fail in large slices. For this reason, the use of vertical slopes to preserve closely adjacent structures or property at the tops of cuts is generally unwise. In several instances of failure in Kansas, the cause was an unrecognized bed of fine sand near the bases of loess deposits. Investigation of the proposed cuts was made with a continuous-flight power-auger, and the subtle change in grain size near the bottom of the auger holes was masked by mixing of sand with the overlying silty material.

Glacial Drift

Glacial tills and outwash deposits are present mainly in the extreme northeastern corner of Kansas where they blanket the bedrock to a considerable depth. Large boulders and discontinuous and unpredictably contorted lenses of clay, sand, and gravel considerably complicate the engineering-geology investigation of these materials. Drainage of perched water tables or reduction of piezometric water levels which affected nearby local water supplies have been the cause of several lawsuits brought against the Kansas Highway Commission.

Terrace Deposits and Alluvial Valley-fill

Terrace deposits at various topographic levels mark the elevations of Pleistocene streams during various pre-glacial and post-glacial stages. These terraces contain much granular material and are important sources of construction aggregates.

The terraces are dissected in most areas by younger fluvial deposits which now occupy the floors of major valleys. The younger sediments, commonly of Wisconsinan or Recent age, are among the most important sources of ground water in the state. They are also dredged for sand and gravel.

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ROCKFALLS AND LANDSLIDES ALONG STATE HIGHWAY 79 AT CLARKSVILLE, MISSOURI

Alan G. Goodfield*

INTRODUCTION

Subject

This paper examines a series of slope stability problems which have developed along State Route 79 in the vicinity of Clarksville, Missouri. In particular, massive rockfalls involving a dolostone-limestone unit and a shale unit are described. Other forms of mass-wasting present in the area along the highway include slumping, debris flows and slides, and landslides. All of these examples of slope instability can be directly traced to weaknesses developed by weathering of the Maquoketa Shale.

Location

Missouri Route 79 is a two-lane state highway that parallels the western edge of the Mississippi River Valley from St. Peters in the south to Hannibal in the north. Clarksville, located in Lincoln County in eastern Missouri, is slightly north of the midpoint of this route. At Clarksville, Route 79 (fig. 1) leaves the Mississippi floodplain and is located in a one-sided cut, approximately 3,200 feet in length, along the foot of a prominent hill called "The Pinnacle." Major slope stability problems are confined to this cut although other areas of slope failure occurs (along the highway) north and south of Clarksville.

Acknowledgements

The writer acknowledges information regarding the rockfalls and landslides at Clarksville which was received from Mr. Ray Wagner, Geologist, Missouri State Highway Department District 3, Hannibal, Missouri, and from Mr. George F. Rubison,

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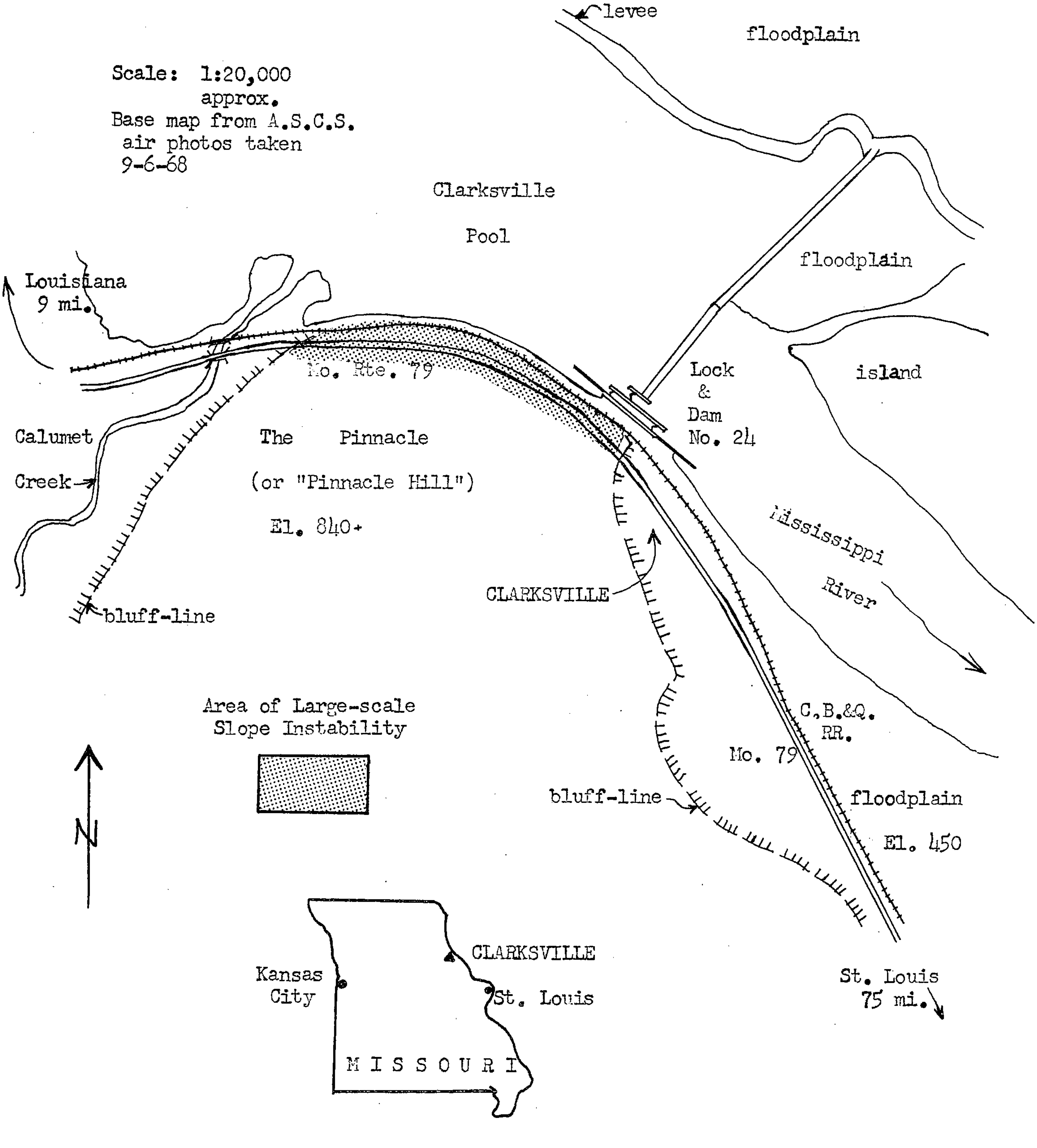

Figure 1. Location Map of the Clarksville Area

a retired engineer formerly with the Missouri State Highway Department.

GEOLOGY

Regional Geology

Clarksville is located on the northeast flank of the northwest-southeast-trending Lincoln Fold in northeast Missouri. The dips of Ordovician through Mississippian rocks on the northeast flank of the fold are extremely gentle, and for practical purposes can be considered to be horizontal.

Pleistocene deposits consisting of Kansan till and Wisconsinan loess are present in the Clarksville area but do not enter directly into the slope failures described in this paper. The rock units present in this area are described below.

The topography of the area studied is of moderate relief; the elevation of the Mississippi floodplain is approximately 450 feet while the top of "The Pinnacle" is slightly greater than 840 feet above sea level.

Geologic Section at "The Pinnacle"

Figure 2 shows the generalized geologic section in the bluff on the north side of Clarksville. A part of this section is also shown in Figure 3. A brief description of this section follows, beginning with rocks at the foot of the bluff.

<u>Maquoketa Formation</u>.- The Maquoketa comprises very clayey shale. The thickness of this unit is reported to be approximately 170 feet. Because of its tendency to weather rapidly, natural exposures of fresh Maquoketa are rare. The Maquoketa is Upper Ordovician in age.

<u>Edgewood Formation</u>.- The Edgewood, of Silurian age, is divided into two units. The lower Noix Member is a white, massive, oolitic limestone about 3 feet thick; the upper Bowling Green Member, is massive, sandy, dolostone approximately 21 feet thick.

Three rock units which have tentatively been assigned a Devonian-Mississippian age by the Missouri Geological Survey overlie the Edgewood Formation. These include, in ascending

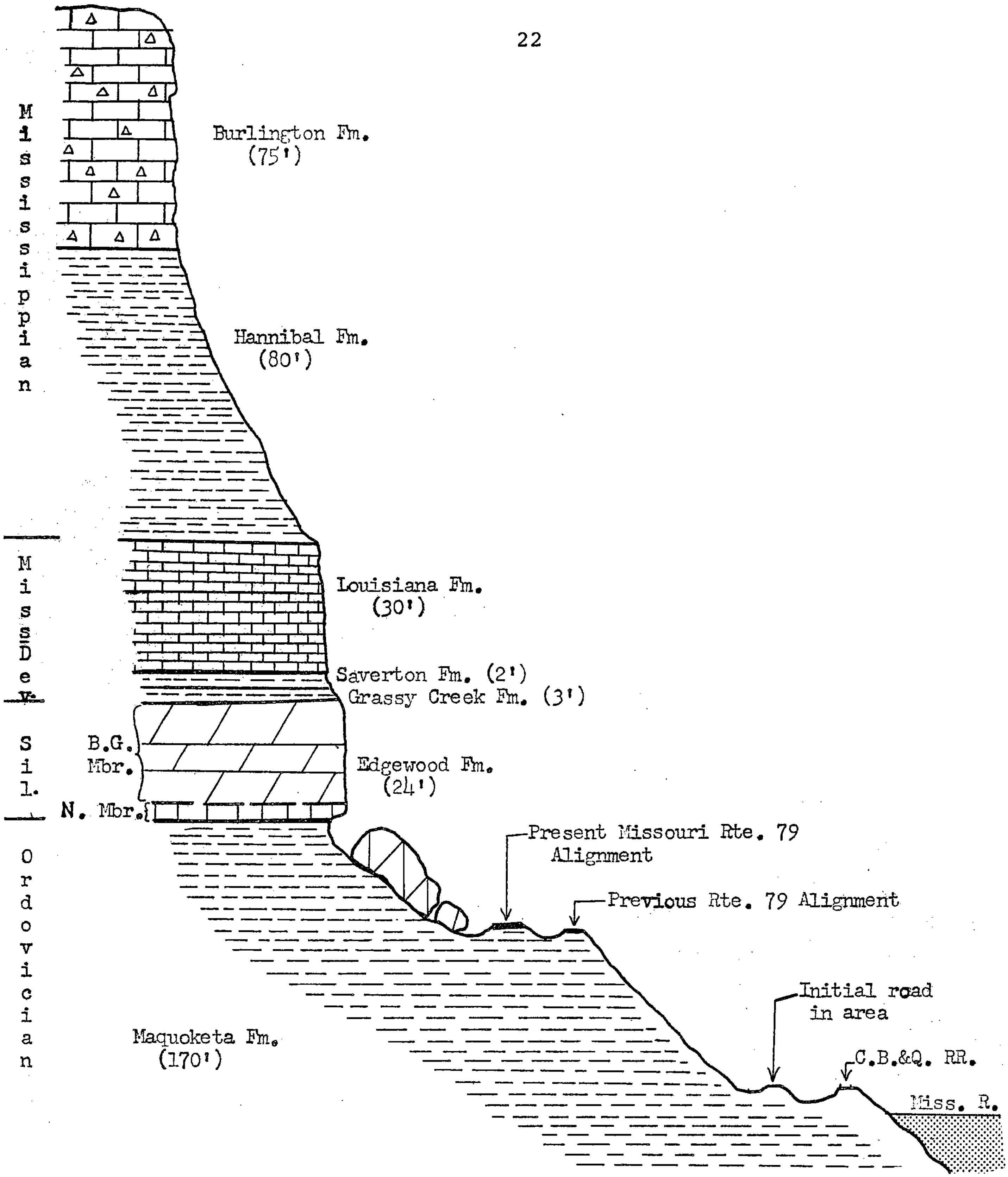

Figure 2. Generalized Geologic Section at "Pinnacle Hill", Clarksville, Missouri

order, the Grassy Creek Formation, a black fissile shale about 3 to 4 feet thick; the Saverton Formation, a green fissile shale about 2 feet thick; and the Louisiana Formation, an even-layered, medium-bedded, very fine-grained to lithographic limestone as thick as 30 feet.

Hannibal Formation.- At Clarksville, this shaly unit of Mississippian age is as thick as about 80 feet. On the bluff at "The Pinnacle", the Hannibal Shale crops out on a steep, tree-covered slope which contrasts with the vertical to near-vertical slopes developed on both the overlying and underlying geologic units.

Burlington Formation.- In the immediate Clarksville area the Burlington Formation is a medium-bedded, cherty, crystalline limestone. Approximately 75 feet of this unit caps "The Pinnacle."

SLOPE STABILITY PROBLEMS

Sequence of Highway Alignments at "The Pinnacle" Bluff

The present alignment of Missouri Route 79 along the bluff north of Clarksville is the youngest of three roads in this area. The first road was located at the foot of the bluff along the Burlington railroad. Following the end of World War II the alignment was moved up onto the face of the bluff. A large slide soon developed at that point. In the early 1960's, in an attempt to remove the highway from the dangers of sliding, the alignment was cut into the face of the bluff at a maximum of 100 feet from the previous alignment (fig. 2). The rockfalls and rockslides described below began after the final alignment was constructed.

Rockfalls and Rockslides

The eastern half of the rock cut along the bluff at "Pinnacle Hill" has been the site of a large number of small to massive rockfalls and rockslides (fig. 3). The two rock units

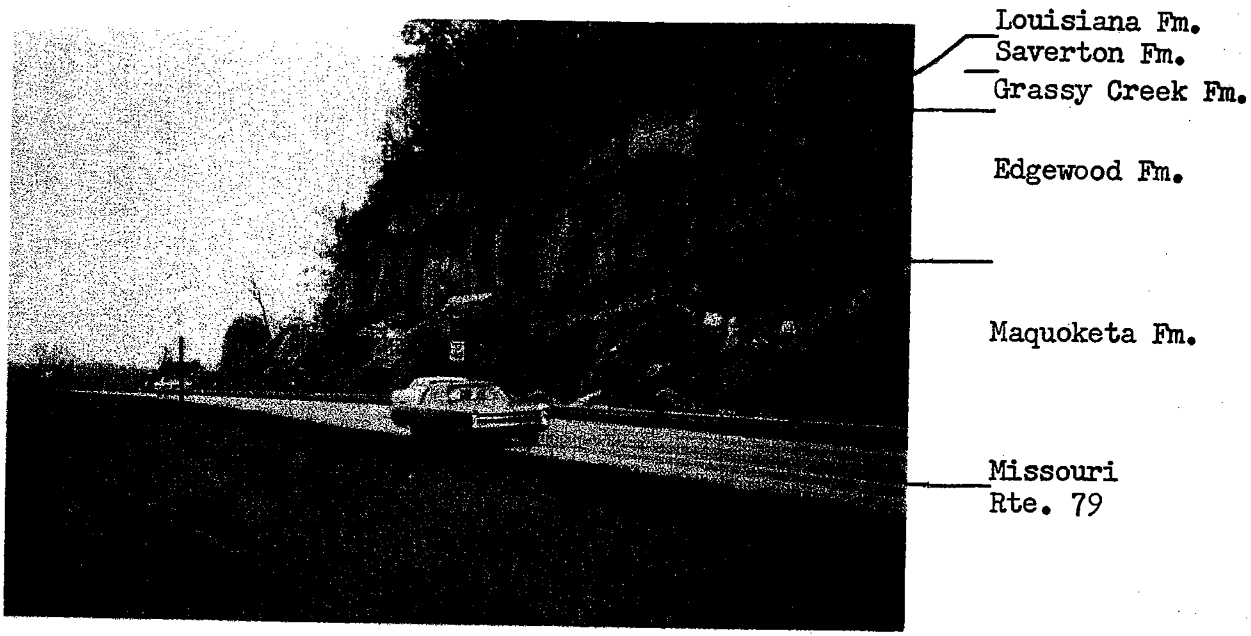

Figure 3. Rockfalls in the Missouri Rte. 79 rock cut at Clarksville. View to the southeast. Previous alignment of this highway is in the lower left-hand corner of the photo.

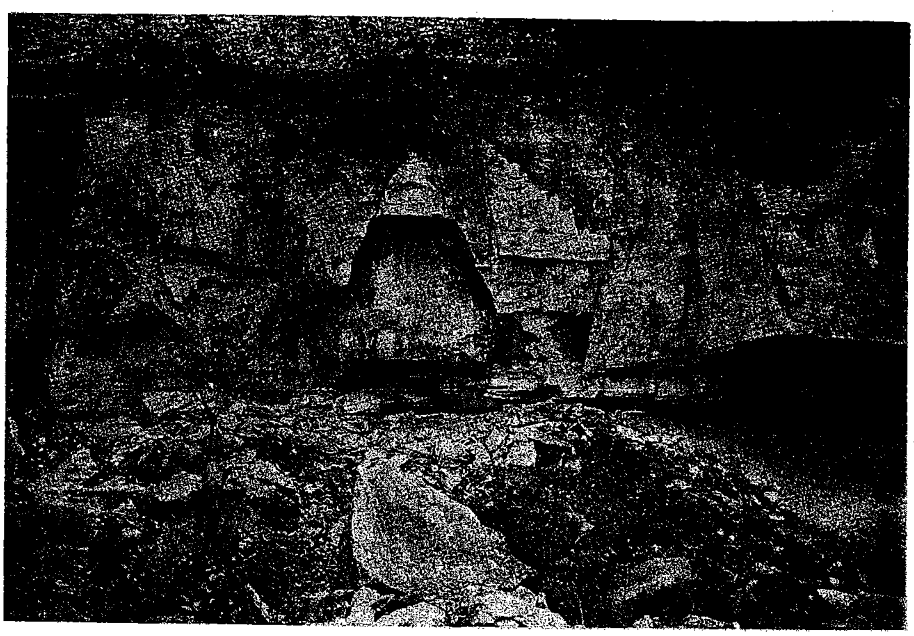

Figure 4. Joint pattern in the Edgewood Formation, weathering and erosion of the underlying Maquoketa Shale, and rockfalls of the Edgewood Formation.

directly involved in these slope failures are the Edgewood Formation and the underlying Maquoketa Shale. The present alignment of Route 79 is in a rock cut whose grade is between 20 and 40 feet below the top of the Maquoketa.

Failure of the massive, resistant, joint-bounded blocks of the Edgewood Formation is initiated by undercutting at the base of the unit. The undercutting is caused by differential weathering and erosion of the soft Maquoketa Shale (fig. 4 and 5). Where it is exposed at the surface, this shale weathers rapidly into a soft and somewhat plastic clay. Water has been observed flowing out from the contact of the shale and the overlying limestone. Analysis of the joint pattern in the Edgewood Formation indicates the presence of two major vertical joint sets striking N 70-80° W and N 20-50° E. These sets (fig. 4 and 5) are respectively about parallel and perpendicular to the face of the cut in the eastern half of the rock cut.

Figure 5 shows a large block of the Edgewood Formation whose failure seems to be imminent. At the southeast end of this block, the underlying shale has been eroded back approximately 8 feet from the face of the overlying block. During one period of observation, a large chunk of the weathered shale broke off from below this block, indicative of the mode of progressive failure affecting the face of the rock cut.

The size of the fallen pieces of the Edgewood Formation varies from small chunks to massive blocks. Between the latter part of 1966 and the Spring of 1968, the largest intact block of Edgewood fell. Measurements of this particular block (shown in the left side of Figure 3) indicate that its volume was about 600 cu. ft. and its weight about 450 tons.

One peculiar aspect of these fallen blocks of Edgewood Formation is that many are bounded by slickensided joint surfaces. Most of the slickensides appear to have been horizontal (in a vertical joint plane) but some seem to have been vertically oriented. The origin of these features is problematic. To the writer's knowledge there are no large faults or fault zones present; certainly the continuity of the rock units exposed

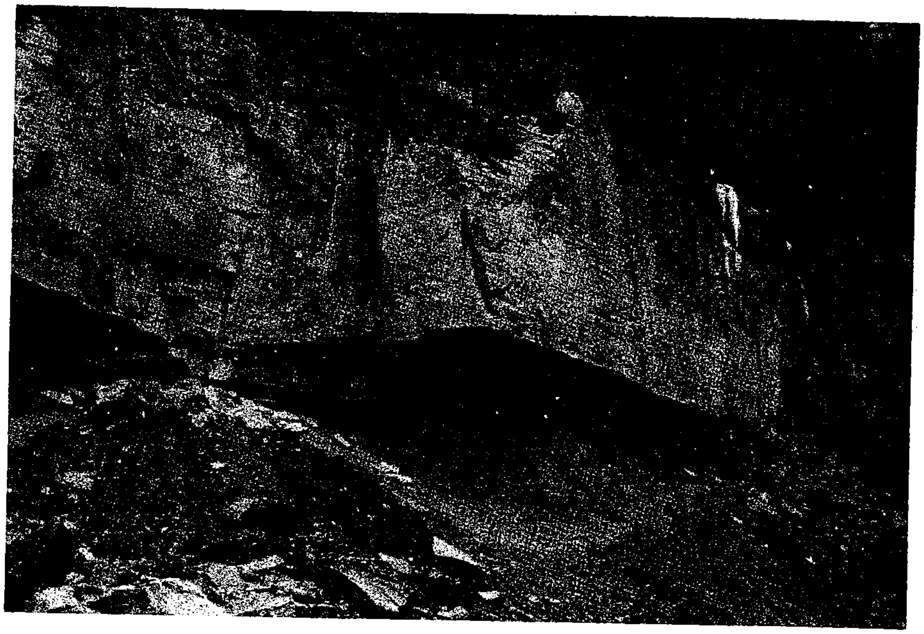

Figure 5. Edgewood Block in danger of falling. Note joints in the Edgewood Formation and erosion of the Maquoketa Shale.

Figure 6. Scar at the head of a recent slope failure and breakup of pavement of previous Highway 79 alignment. Failure block is moving downslope to the left toward the Mississippi River.

in the bluff shows no indication of offset. The slickensides may have accompanied the formation of joints during deformation that produced the Lincoln Fold.

According to Mr. Ray Wagner, it is hoped that the weathered Maquoketa Shale will eventually become naturally "paved" with fallen blocks of the Edgewood Formation and that a stable condition will result.

Landslides at the Clarksville Bluff Roadcut

In the Fall of 1969, the second alignment of Route 79 (fig. 2) started to break up and move downslope toward the Mississippi River. The head of this slide is shown in Figure 6. This latest movement appears to be largely a surficial feature as an apparent bulge or toe can be detected on the tree-covered slope about 20 feet below the old pavement. This apparently surficial slide is in contrast to the slide that developed after the second alignment was built, and that reportedly caused a small deflection of the C. B. and Q. railroad tracks toward the Mississippi River. Toward the western end of the "Pinnacle Hill" cut, lobate and hummocky forms on the weathered shale slope suggest that the weathered Maquoketa Shale has undergone some flowage or sliding.

The number and scale of rockfalls of blocks of the Edgewood Formation are considerably less in the western half of the Route 79 bluff-cut than in the eastern half. The reasons for this difference are not presently known.

Other Maquoketa Slope Failures

Along the Mississippi River approximately 2 miles northwest of Clarksville, Route 79 passes through a series of shallow, one-sided cuts in the Maquoketa Shale. The shale is highly weathered, generally appearing as a soft clay. Several relatively small slides have developed in these cuts, but generally, the only damage resulting from these slides is the blocking of the drainage ditches along the highway.

The moderately steep slope forming the bluff-line south of the city of Clarksville is extremely hummocky and shows considerable evidence of mass-movement. This slope is also developed on the Maquoketa Shale.

CONCLUSIONS

Virtually all of the examples of slope failures described in this report can be related to one major geologic unit - the Maquoketa Shale. The rapid weathering and erosion of this shale undercuts the more resistant, overlying Edgewood Formation. In addition, the weathered shale is highly susceptible to surficial sliding and flow failures. It may also be susceptible to deeper sliding failures.

This evidence leads to the conclusion that proper recognition of potential "geologic hazards" as early as possible in the planning stages would result in safer and more economical highways.

CORRELATION OF EXPANSIVE SOIL PROPERTIES AND SOIL MOISTURE WITH PAVEMENT DISTRESS IN ROADWAYS IN WESTERN SOUTH DAKOTA

D. W. Hammerquist and Earl Hoskins*

Pavement distress (or "bumps") have plagued highway engineers ever since they have built highways. This paper results from a research project that was directed at finding causes of these bumps. The project was sponsored by the South Dakota Highway Department and by the Bureau of Public Roads.

Primarily, the bumps seemed to be in cut sections. Fill sections--at least on the newly constructed roads--show little evidence of distress. However, as soon as some of the cuts are entered, the roller-coaster effect begins. The general attack of this project was to trench along the roadway to determine the type of material which underlay the roadway and to correlate these findings with pavement distress. The first trench was dug along an older section of a federal aid project. The subgrade material was a part of the Pierre Shale Formation. The beginning of the trench showed uniform shale--as uniform as any shale can be. The first discontinuity was a thrust fault, and upon looking along the fault trace, a large bump in the roadway was noted. After digging several trenches along almost 200 miles of highways, this same feature was noted--that is, sharp bumps associated with faults. Most of the trenches were made in cuts that showed bumps, but a few were dug in areas of undistressed pavement. All trenches were made into the Pierre Shale.

The Pierre Shale Formation is a large body of rock and probably has one of the largest outcrops of any formation in the world. Much of this shale contains a rather high percentage of montmorillonite clay. This clay mineral can be dispersed or concentrated in thin beds of bentonite. However, a lot of the shale is marly rather than bentonitic, and some of the marl beds are almost limestone. Fortunately, the marl outcrop area is large and covers much of the uplands.

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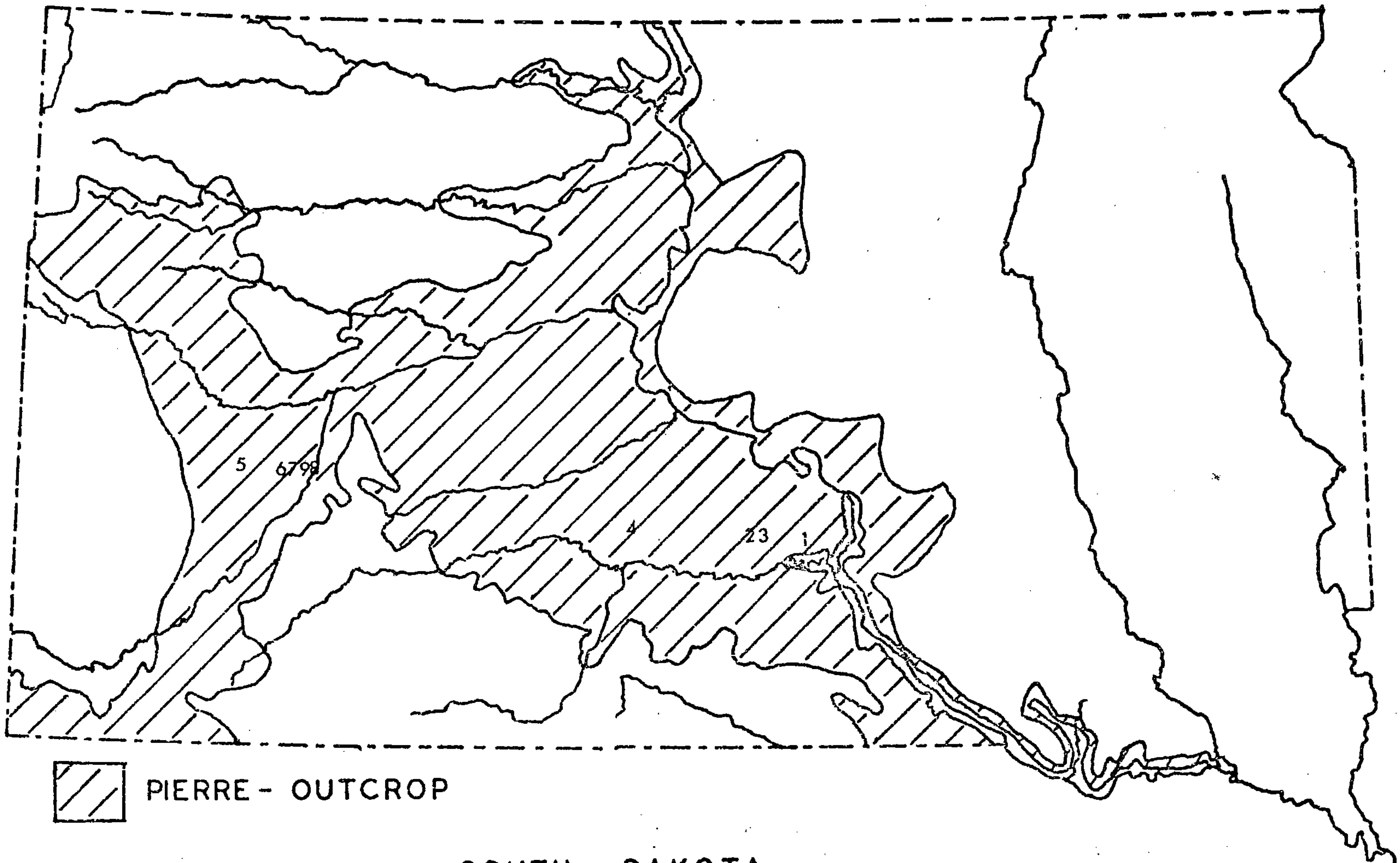
DISTRIBUTION OF THE PIERRE FORMATION
AND LOCATION OF NINE TEST SITES



SOUTH DAKOTA
Figure 1

If the shale beds were parallel, the problem would not be complex. However, in some areas, the Pierre Formation is cut by an intricate fault system--faults that vary in throw from a few inches to many hundreds of feet. Geological mapping of the Pierre Formation is difficult. The outcrops are many, but the continuity of beds is sometimes next to impossible to "walk out."

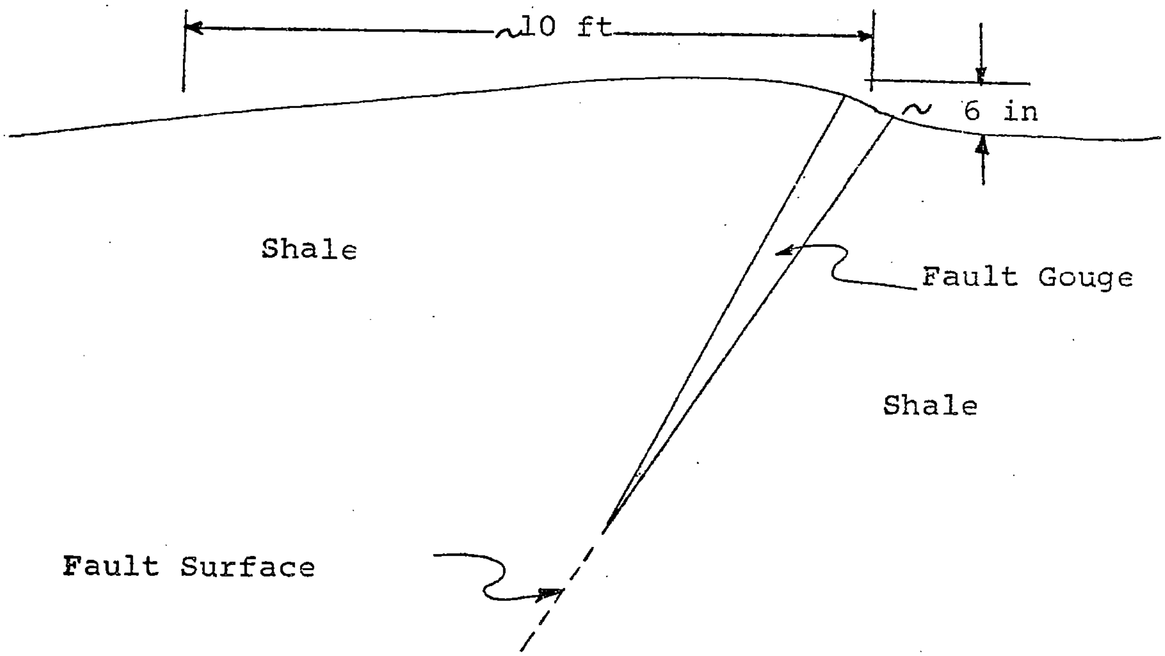
Bumps in the highway are caused by several factors. Differential consolidation of the embankment is one cause, but this is not a great factor in newer highways due to improved embankment control. Another cause is at the "daylight"--where the roadway goes from cut to fill. A few daylight bumps were found, but as their magnitude is low, most motorists will probably not notice them. The sharpest bumps were found within the cut section and were associated with faults. These bumps varied in height, but 6 inches seemed to be about average. The cause of these bumps is differential swelling of the fault gouge. The gouge is finer grained than the surrounding shale and water can change its volume faster. Also, unloading of the shale is a causal factor. Originally, the shale was under a load of bedrock, under which a state of equilibrium was reached. When a road cut is made through a hill, the shale is unloaded and there is a tendency for the shale to rebound or swell. If some material swells faster than other material, a bump is caused.

Some parts of the Pierre Formation show no evidence of bumps. The marly beds are among these units, apparently because they are non-swelling. Only a few units do have bumps, and these appear to be the ones with a certain combination of parameters. Time is one of these parameters, as some materials will swell faster than others. Also a source of water is necessary, and water was found in most of the trenches. Shale is commonly supposed to be impervious, but actually some layers are as porous as sand or gravel layers. Also, a swelling mineral is necessary for creation of a bump, and as montmorillonite swells more than other clay minerals, the volume change is of great magnitude, in some instances as much as 16 times the dry volume. A one-half-inch layer of fault gouge can thus become a layer of swollen material as thick as 5 inches or more. A characteristic of the bumps is their asymmetry. In other words, the bump is higher on one side because of the dip of the fault plane (fig. 2).

SUMMARY

It is believed that the sharp bumps in cut sections are caused by differential swelling of fault gouge. The gouge will swell faster than the surrounding shale and thereby create a "bump." Also, the material has to be high-swelling, and there must be a source of water.

Cross Section of a Typical Bump



FROM SITE 5

Fault strikes N 64° E and dips 52° SE

Section measured in North ditch

FUTURE RESEARCH

The most important fields of future research would be determination of the locations of faults and location of high-swelling members of the Pierre Shale. An additional field would be preparation of a good areal map showing units of the Pierre Shale. The authors of this paper are involved in a project that uses infrared photography to locate fault zones, water tables, and other geological features. Yet another field of future research would be the effects of inefficient ditch drainage.

GEOMETRIC ANALYSIS OF ROCK SLOPES

Charles L. Taylor*

INTRODUCTION

Geological separations or planes of weakness divide a rock slope into geometric figures. These separations are planar features such as fractures, joints, bedding planes, shear planes, and the intersections of these planar features. When these geological planes of weakness are undercut, the mass is potentially unstable and the slope has an unfavorable orientation or inclination. This paper is intended only to explore the geometric analysis and to indicate potential unstable conditions, and does not attempt to determine the stability or factor of safety for any slope.

During the initial phase of any slope stability analysis, preliminary field evaluation of the shear strength, geological separations, and cohesion along these separations should be made. If this evaluation indicates that no geological separations are present, or that the shear strength and the cohesion values for the material is such that potential movement or failure would be likely to occur within the slope material and not along geological separations, then conventional stability analysis should be performed. If after making the preliminary evaluation of a slope it is believed that potential movement would be possible along the geological separations, then a detailed geometric analysis of the slope is required.

The accuracy and completeness of the field data will determine the reliability of the final analysis. For this reason, extensive field mapping is required. The mapping will consist of measuring and recording the orientation of geological separations or planes of weakness in outcrops and in orientated sampling surfaces. The information recorded will include standard geological information such as attitude, width of the planes, coatings, weathering condition, thickness, strength of the rock material,

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and the cohesion along the planes of weakness. The recording of the data must be thorough, but contain no personal preference for any particular set of geological separations because of abundance or orientation.

Limited exposures of bedrock seldom yield complete data on the geometry of a rock mass. The data that are obtained are only samples and may not even be typical of the bedrock unit. The ease of collection, width of planes, and other physical conditions or personal selection control the choice of data. The orientation of the sampling surfaces may also present a built-in preference for certain orientations of the geological separations. Therefore, the orientation of sampling surfaces must be evaluated and corrections made to make the data more representative, or the sampling surfaces must be constructed with an orientation that will reveal representable geometric data.

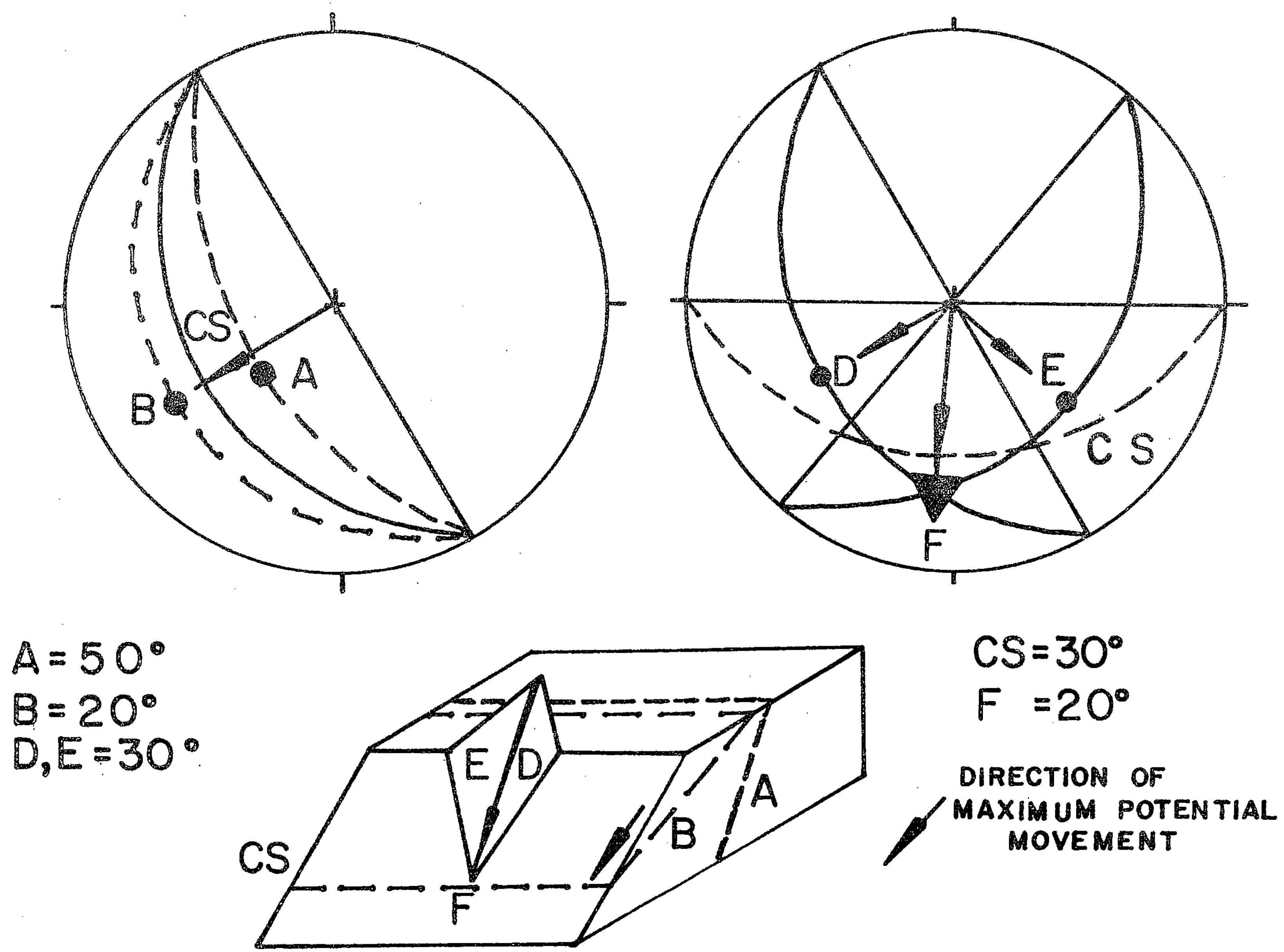
GRAPHIC METHODS FOR GEOMETRIC ANALYSIS OF GEOLOGICAL SEPARATIONS

Stereographic Projection

The stereographic system of graphic presentation permits the projection of three-dimensional, space-oriented planes onto a two-dimensional surface. Using the principles of this system, meridians and parallels are projected to construct a stereonet, on which the orientation of geological separations can be plotted conveniently. The methods and procedures employed in stereographic plotting will not be presented in this paper.

Geological Separations and Cut Slope Inclination.- Cut slopes and other planar geological features can be conveniently represented on stereomet diagrams by circles and arcs (fig. 1). The orientation and inclination of planar features can further be represented by points. In the stereographic system of projection, the strike of a plane is represented by a straight line through the center of the net, and the dipping surface is indicated by an arc. A point placed on the arc, at right angle to the strike, represents the intersection of the dip with the lower hemisphere and is referred to as the dip point. This is a unique point that represents the orientation, strike and dip point. This is a unique point that represents the orientation, strike, dip, of a "three-dimensional plane."

FIGURE 1 - STEREOGRAPHIC PLOTTING OF GEOLOGICAL SEPARATIONS, CUTSLOPE ORIENTATION AND INCLINATION. A LINE FROM THE CENTER OF THE NET TO THE INDIVIDUAL POINT INDICATES THE DIRECTION OF MAXIMUM POTENTIAL MOVEMENT.



In contrast with other plotting methods, these procedures use the lower hemisphere for plotting of data, but they do not use poles or normals to represent the orientation of planes. Points located at the center of the stereographic net represent a dip of 90 degrees, a vertical plane. Points located at the outer circle represent a zero-degree dip, or a horizontal plane. The dip is always plotted at a right angle to the strike.

The stereographic method for plotting intersecting features is similar to that described above. The strike and arc of both geological planes are plotted. The intersecting of the two planes forms a line of intersection. This line is indicated as being from the center of the net to a point "F" (fig. 1). The trend of this line represents a bearing. The plunge angle, or angle from the horizontal surface, is also represented by point "F." Point "F" represents the orientation of the intersection of the two geological separations and is referred to as the plunge downward angle at which the intersecting line plunges. The plunge point and the dip point indicate the direction and angle of maximum potential movement from the center of the net.

Any dip or plunge point which is outside an arc representing the slope angle (see fig. 1) as "B", and "F", represents a planar or linear feature having a given orientation to that particular cut slope because the plane is undercut by the slope's free face, as indicated in the block diagram in Figure 1. Any point that is inside a circle or arc (as "A", "D", and "E") is steeper than the cut slope angle, and therefore favorable to that particular cut slope orientation and angle.

Potential for Movement.- The potential for movement is greatest along steeper planes, if frictional resistance or cohesion and other factors remain the same. The plotted point may represent a single separation or a set of parallel separations with the same given orientation and inclination. The steeper planes or angles are represented by points plotted near the center of the net.

Interpretation of Data.- Once the results of a thorough joint survey have been plotted on stereonets, a wide variation in orientation and inclination of slopes can be studied. The most preferred cut-slope orientations can generally be obtained from the stereonet and density diagrams.

The diagram of the Sobrante slope (fig. 2) shows dip points and plunge points for geological separations observed in a proposed cut slope. The direction of maximum potential movement is from the center of the net to the individual points. Any point outside of any of the cut-slope or native-slope arcs is flatter than the proposed cut slope and therefore is potentially unfavorable for that slope orientation and angle.

Note the small number of points outside of the native slope arc (fig. 2). The number of points observed outside of the cut slope arcs increases rapidly as the proposed cut slopes become steeper. This diagram represents the geometric analysis of the geological separations of a proposed cut slope. It indicates the direction and angle of maximum potential movement for the individual planes and their intersections, and the relative stability of the different proposed slope angles.

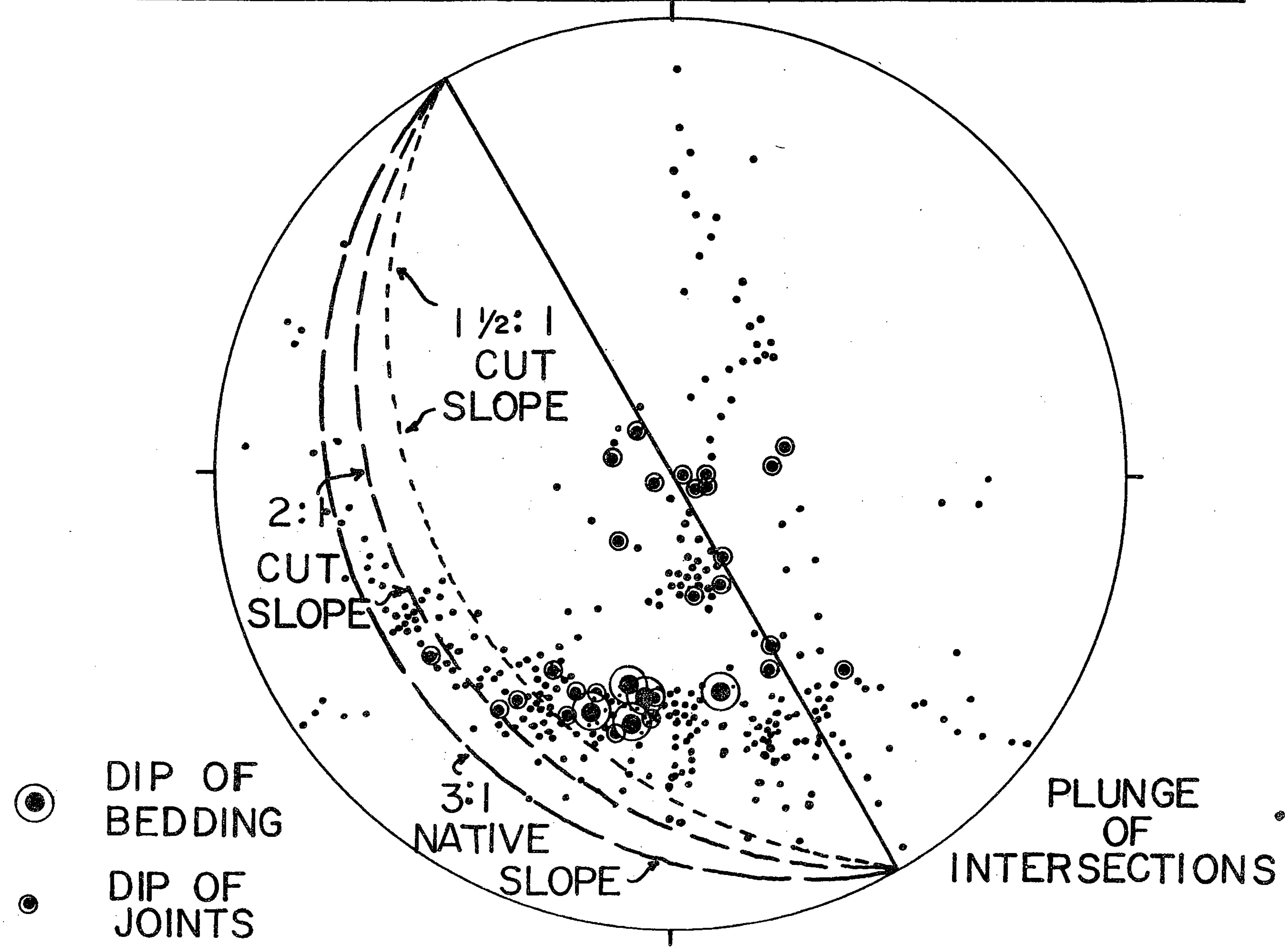
The geometric analysis is now complete, and at this point evaluation and additional study are necessary to obtain a stability analysis and to determine the factor of safety for each proposed cut slope angle.

CASE HISTORIES

In order to check the assumptions and to obtain a better understanding of the methods presented in this paper, five cut slopes were studied and analyzed geometrically. Slides of varying orientations, inclinations, and magnitude were developed on four of the slopes. The fifth slope was used as a control. The joint data were gathered as close as possible to the slide area, but not within the disturbed slide area.

The data were collected in the winter of 1966 from the Orinda Formation of Contra Costa County, California. The Orinda Formation consists of continental, fresh-water sediments of Miocene and Pliocene age. The sediments are gradational, discontinuous, and include lenticular beds of claystone-clay shale, sandstone, and conglomerate. Most of the rocks are highly jointed. The joints

FIGURE 2 - THE ORIENTATION OF THE GEOLOGICAL SEPARATIONS INDICATES THE DIRECTION OF MAXIMUM POTENTIAL MOVEMENT ALONG THE PLANES. ANY POINT WHICH OCCURS OUTSIDE OF AN ARC REPRESENTING THE CUT SLOPE, IS POTENTIALLY UNFAVORABLE FOR THAT PROPOSED ANGLE.



range from fractures one-fourth of an inch wide, spaced a foot or more apart, to hair-line cracks spaced only fractions of an inch apart. The claystone consists of 24 percent to 95 percent montmorillonite by weight, and its plastic index ranges from about 25 to 60.

Upper Happy Valley Slide No. 2

The diagram for Upper Happy Valley Slide No. 2 (fig. 3) is a representation of the orientation of bedding, joint planes, and their intersections. Note the high concentration of points located outside of the cut slope arc, and in the direction the slide moved. This slide has a long history of instability and has now been buttressed with a 2:1 fill slope. This diagram is typical of those based on other cut slopes which have a history of instability. The very high concentration of unfavorable plunge orientations and the strong preferred direction of plunge may be an indication of unstable or potentially unstable slopes.

Camino Pablo Cut Slope

The Camino Pablo cut slope (fig. 4) was selected as a control slope because of its stability. No major instability problems have existed during its 10 to 15 year history. It will be noted in the figure that no bedding planes, and only a few joint planes, are undercut by the 50-degree cut slope. Examination of the figure will reveal that there is a variation in the direction of the plunge, into the cut slope as well as out of the slope, and there is no concentration, or preferred direction of plunge points. The lack of a preferred direction of plunge, or potential movement, may be an indication of a geometrically stable slope.

Joint Intersection Density Diagrams

To aid in interpreting the distribution of the plotted points, a density percentage diagram can be constructed to indicate the concentration of orientations. The percentage values indicate the percentage of points within an area representing one percent of the total area, and the value indicates that within the designated circle, x percent of the plotted points have that orientation.

FIGURE 3 - THE GEOLOGICAL SEPARATIONS UNDERCUT BY THIS SLOPE ARE PREDOMINANTLY JOINT INTERSECTIONS. NOTE THE HIGH CONCENTRATION OF POINTS OUTSIDE OF THE CUT SLOPE ARC, AND IN THE DIRECTION THE SLIDE MOVED.

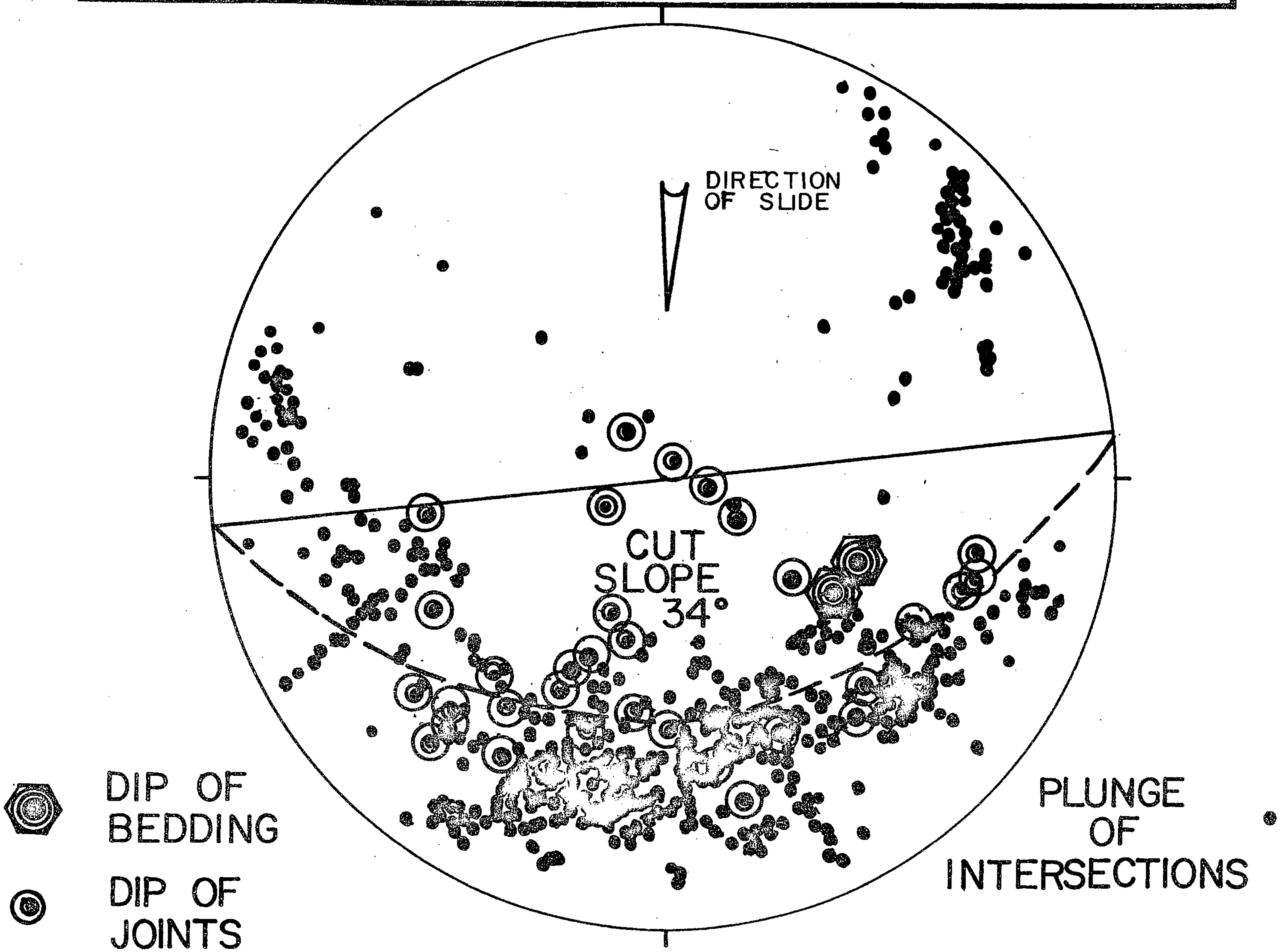
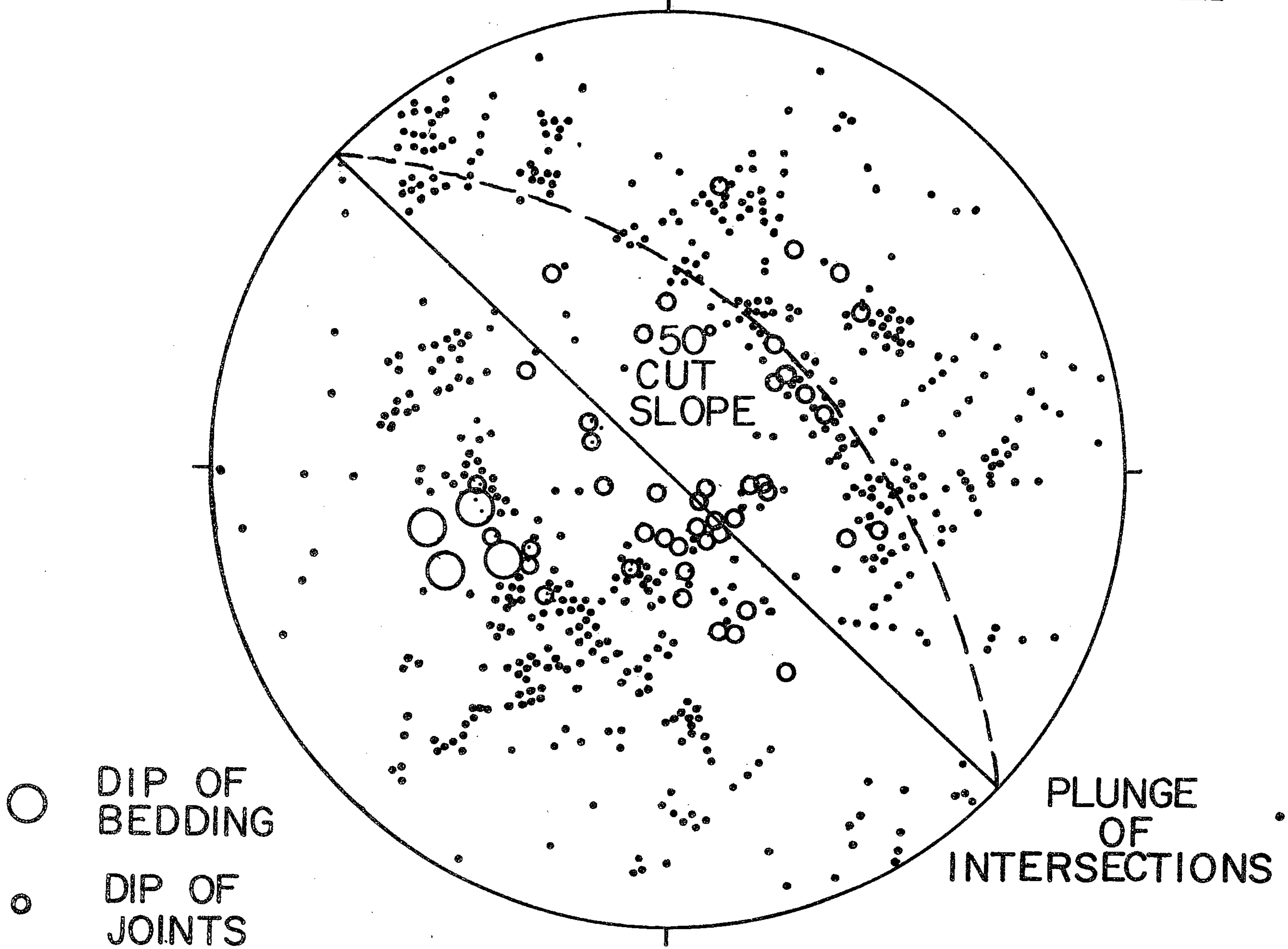


FIGURE 4 - THIS CUT SLOPE HAS HAD NO PROBLEMS OF INSTABILITY DURING ITS 10 - 15 YEAR HISTORY. NOTE THE WIDE RANGE IN ORIENTATIONS PRESENT WITHIN THE SLOPE.



A density percentage diagram of Upper Happy Valley Road indicates a high degree, 10 to 11 percent, of unfavorable plunge orientations, they show a preferred direction (the direction that the slide moved). The Camino Pablo density diagram indicates a less preferred orientation of intersecting features, and a low percentage of concentration, or density of points. The general good stability of this slope may be attributed to the "random" variation of the intersecting features, or possibly to other controlling factors.

RELIABILITY OF DATA AND GEOMETRIC ANALYSIS

Variations in Geometric Relationships with Changes in Orientation of Sampling Surface

A graphic model was constructed in order to study the influence of geometric relationships to the change in the orientation of sampling surfaces. This model includes 11 north-south trending joints, and 11 east-west trending joints (fig. 5). The orientation and construction values of the joints are shown in Figure 5.

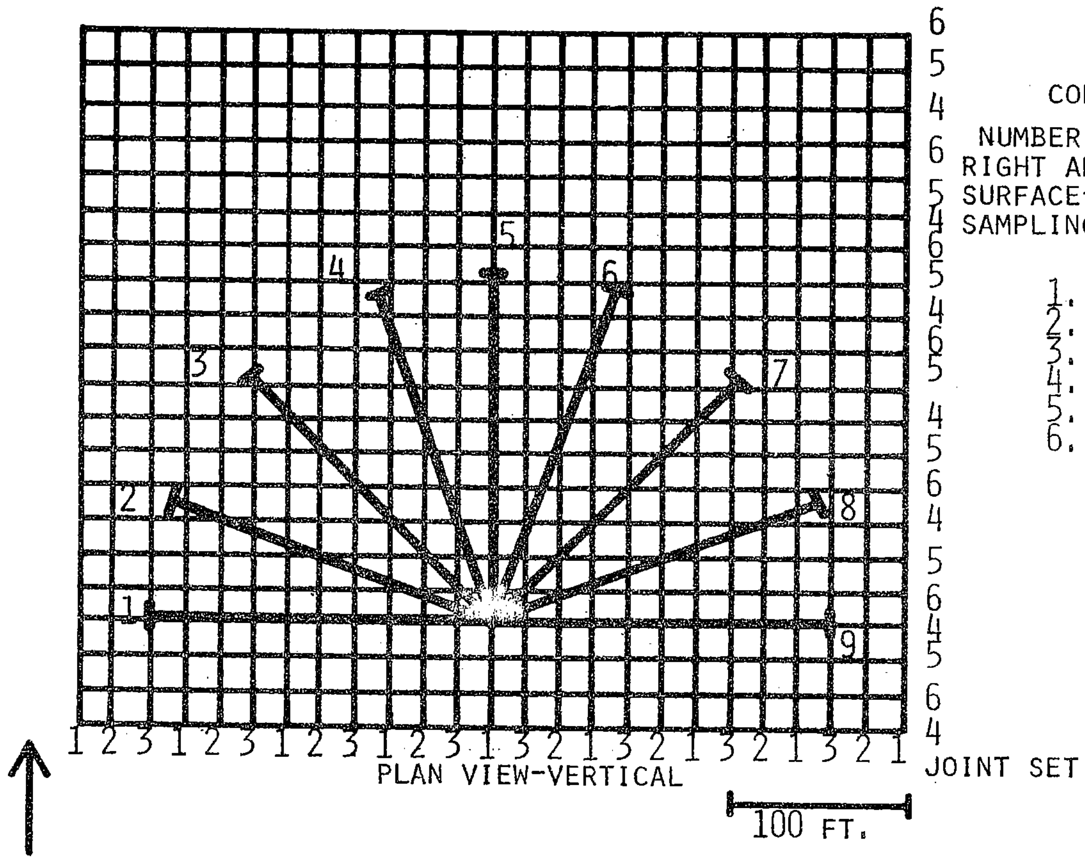
Nine 200-foot-long test trenches were graphed to scale through the model joint system. TT-1 intersects 11 north-south joints and only one east-west joint. TT-2 intersects 10 north-south joints and only four east-west joints. It can be seen in TT-3, TT-4, and TT-5 that the number of north-south joints decreases and the number of east-west joints increases, as we move through the model system.

The relationship between the number of observed joints and the angle between the test trench and the strike of the joint is shown in Figure 6. As the angle of intersection between the strike of the joint and the orientation of the test trench increases and becomes larger, the data recorded will more accurately represent the true conditions. When the strike of the joint is at a right angle to the test trench then, and only then, is the true geometric condition visible, and available for sampling. As the angle between the strike and the test trench approaches 0°, the less representative are the data exposed in the test trench. At angles less than 45 degrees the number of observed joints decreases rapidly.

FIGURE 5 ORIENTATION OF TEST TRENCHES WITHIN A MODELED JOINT SYSTEM. THE NUMBER OF JOINTS OBSERVED DECREASES AS THE ANGLE BETWEEN THE TEST TRENCH AND THE STRIKE OF THE JOINT BECOMES LESS.

CONSTRUCTION

1. N 0 W 60 S
2. N 0 W 10 S
3. N 0 W 20 S
4. N 90 W 60 S
5. N 90 W 10 S
6. N 90 W 20 S



CONSTRUCTION

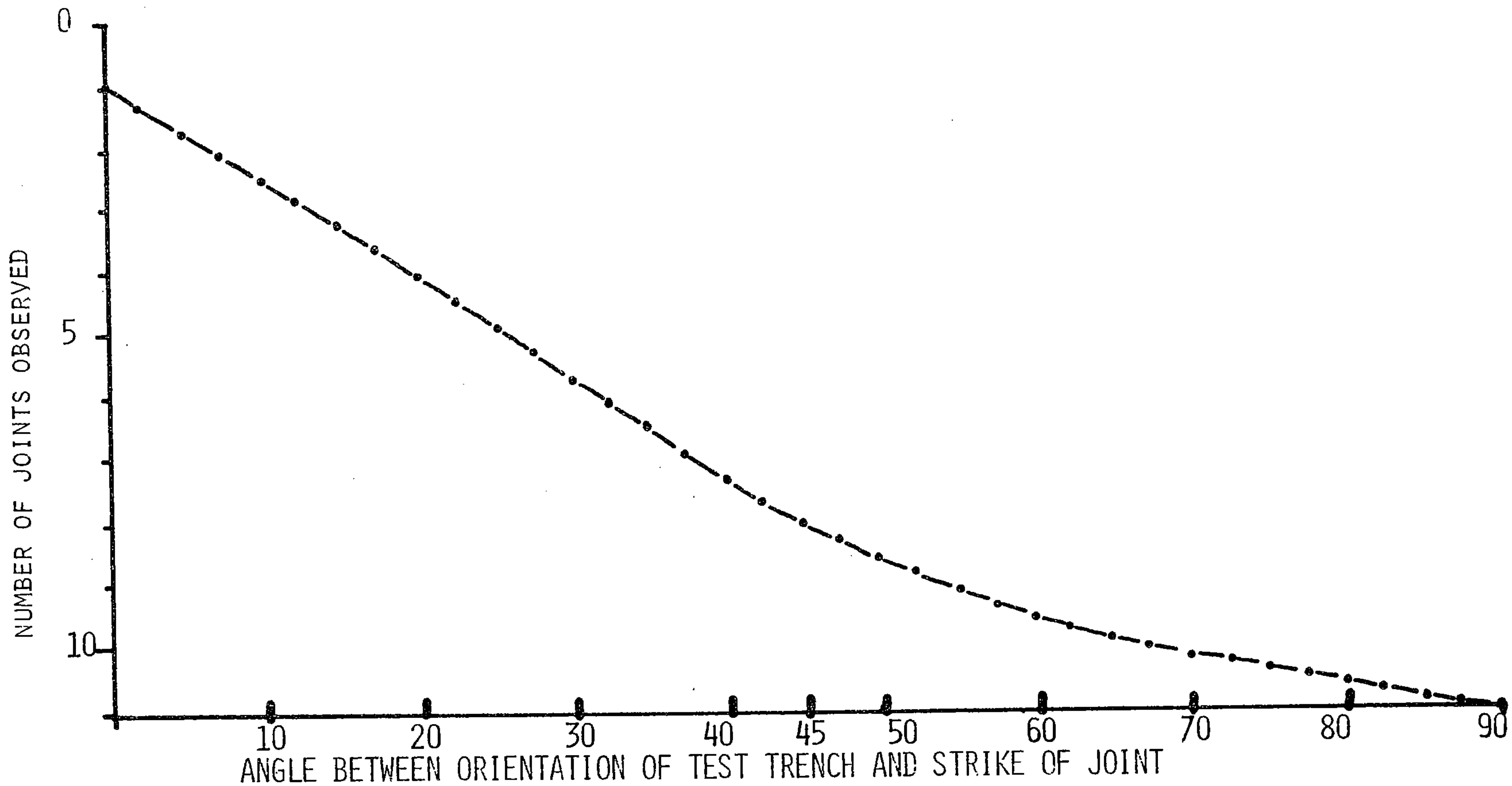
NUMBER OF JOINTS AT RIGHT ANGLES TO JOINT SURFACE-200 FOOT LONG SAMPLING SECTION.

1. 5
2. 19
3. 10
4. 5
5. 19
6. 10

NUMBER OF JOINTS OBSERVED IN TEST TRENCHES

	TT-1	2	3	4	5	6	7	8	9
NORTH-SOUTH SETS	11	10	8	4	1	4	8	10	11
EAST-WEST SETS	1	4	8	10	11	10	8	4	1

FIGURE 6 VARIATION IN JOINTS OBSERVED WITH CHANGES IN TEST TRENCH ORIENTATIONS. THE NUMBER OF JOINTS OBSERVED DECREASES RAPIDLY AT ANGLES LESS THAN 45 DEGREES.



The data as observed in the modeled test trenches indicate a wide variation between the true geometric condition and the sampled conditions. In this case the variation is due to orientation of the test trenches and joint planes, and not to poor sampling procedures within trenches.

The number of joints that can be observed in a test-trench wall will also vary as the dip angle changes. Figure 7 is the cross-sectional view of several hypothetical test trenches. Eleven joints have been constructed within a 200-foot-long test trench. The four sections on the left represent joint sets intersecting the test trench at right angles. Based on the above discussion concerning the angle of intersection, this orientation should present the true geometric arrangement of that particular joint set. However, the actual geometric condition is only available when the dip is 90 degrees, or at a right angle to a horizontal line. The 11 joints are visible in the first block, where they have a dip of 90 degrees. As the dip becomes less, the number of visible joints decreases. This decrease in observable joints is noted within the total test trench, and along any horizontal line.

The blocks on the right (fig. 7) represent the visible intersection between a test-trench wall and a 30-degree-dipping joint set that intersects the test trench at varying angles. With a variation in the intersecting angles, a 30-degree-dipping joint will intersect the test trench at an apparent dip angle. The true dip can only be measured or observed at a right angle to the strike. As an angle between the test trench and the strike of the joint becomes less, the apparent dip will decrease from 30 degrees, to 22 degrees, to 11 degrees, and finally to 3 degrees.

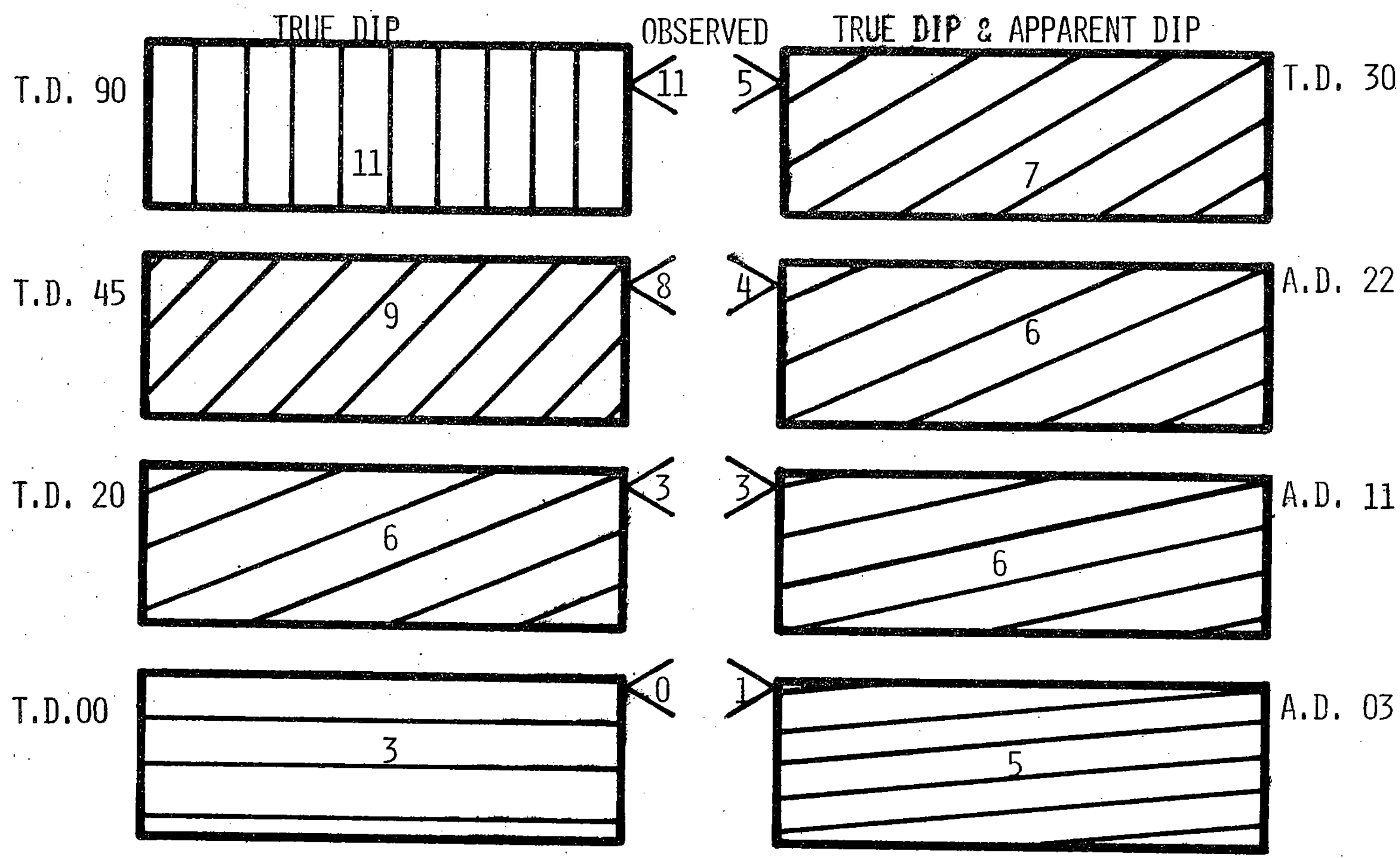
The number of joints observable in the test-trench walls will also decrease. When the modeled joint set (30-degree dip) intersects the test trench at 90 degrees, seven joints can be observed in the total trench, and five along a horizontal line. This value decreases as the apparent dip angle becomes less.

It is apparent that the number of joint planes that may be observed or sampled on any surface depends upon the angle between

FIGURE 7 VARIATION IN NUMBER OF JOINTS OBSERVED WITH CHANGES IN DIP ANGLE. THE TRUE DIP IS TAKEN AT RIGHT ANGLES TO THE STRIKE. THE APPARENT DIP IS TAKEN FROM SECTIONS AT VARYING ANGLES FROM THE STRIKE OF A 30 DEGREE DIPPING JOINT SET.

(CROSS SECTION VIEWS)

NOTE: SPACING EQUAL, 11 JOINTS CONSTRUCTED NORMAL TO JOINT SURFACE



the sampling surface, the strike of the joint (angle of intersection), and the dip angle.

These two angles produce what is commonly referred to as apparent dip. What we see on exposures of a sampling surface is an apparent geometric condition. The sampling surfaces do not represent true joint spacing, percentage of occurrence, or any other geometric values. The data obtained from these sampling surfaces cannot be used as reliable data until corrections have been made to obtain a more representative indication of the true geometric relationships.

Sampling Methods and Presentation of Data

Sampling surfaces that we refer to can be rock outcrops, test-trench walls, or core holes. Generally the area of rock outcrops is small, test trenches are small in a vertical direction compared to horizontal direction, and test holes have only one direction of observation. Rock outcrops and test trenches have only two directions of observation. Geological separations do not exist in one or two dimensions, but in three. Thus, any sampling must be done in three dimensions, or correctons must be made to obtain a corrected value.

Too often, sampling of joint data is done in a disorganized manner. The geologist moves over a hillside recording a great number of unrelated strikes and dips. More attention is paid to ease of collection and of readings than to whether the readings are representative of the joint population within the rock mass. In order to obtain representative data, each member of the population must be given an equal chance of being included within the sampled data.

The following procedures are suggested as a means of standardizing sampling methods from one series of test trenches or observation points to another.

1. Sampling should be done in a straight horizontal line along a single sampling surface, as indicated in Figure 4. This is necessary to allow for later correction of data. This method eliminates jumping around on an exposed face and requires an orderly recording of joints as they appear on the

sampling surface. A horizontal sampling line will intersect more high-angle dipping planes, and a vertical sampling line will intersect more low-dipping planes.

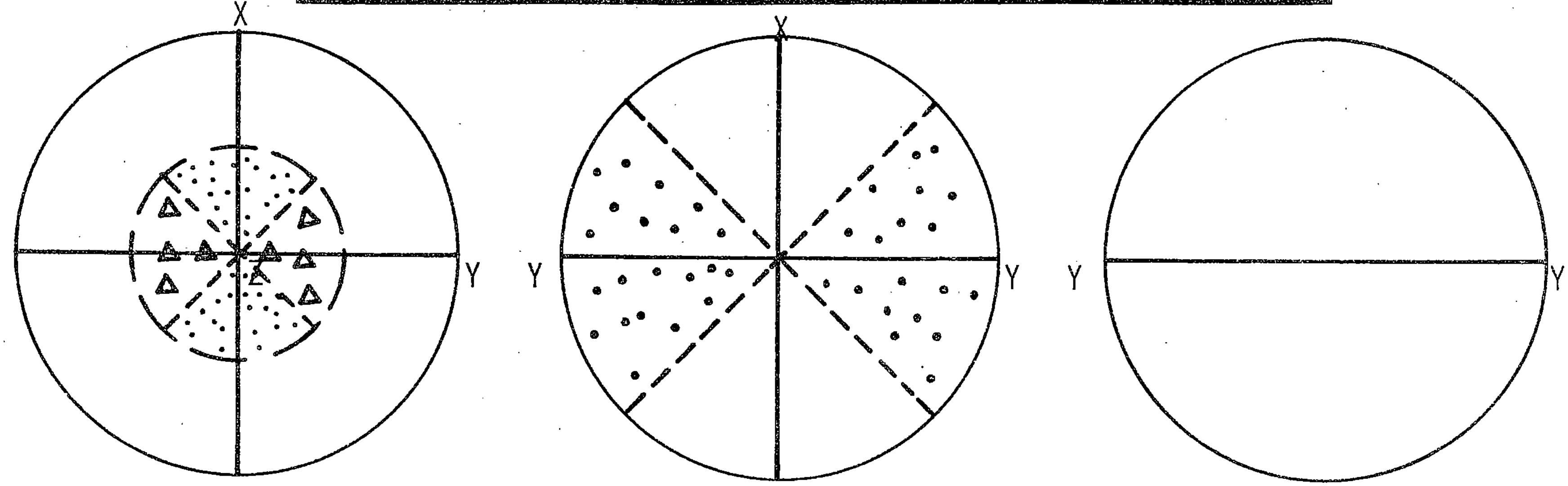
2. Sampling should be done along two sampling surfaces at right angles to each other, with correction made to the data according to the two-axis method. If very accurate and reliable data are needed for shallow dips (less than 30 degrees) a vertical observation direction should be provided and data corrected by the three-axis method. If this degree of accuracy is not required, then attention should be given to the possible errors in calculations for shallow dips, and the large correction factors required.

3. Data from two test trenches should be equal in number of points counted, and in length of test trenches from which the data were obtained. If this is impossible, then percentage-contour diagrams will be required for the different locations, to show percentage of occurrence for different joint sets.

4. Joint data should be plotted on stereographic diagrams, one for each direction of sampling. If two test trenches are sampled along the X and Y axes, respectively, then two separate diagrams are needed, (see fig. 8). If a vertical sampling direction (Z axis) is used, it will also require a diagram.

5. A composite diagram should be prepared for each two- or three-axis sampling location. These composite diagrams are shown in Figure 8. Only those points representing joints or planes intersecting a sampling surface at an angle greater than 45 degrees will be included in the diagram. For TT-X, with a north-south orientation, this includes only those points that fall within 45 degrees of the north-south line, and therefore represents planes that intersect the test trench at an angle greater than 45 degrees. For TT-Y with an east-west orientation, the points must fall within 45 degrees of the east-west line. If the points fall within these sections, the strike of the plane is at least 45 degrees from the axis of observation.

FIGURE 8 SAMPLING METHODS. THE TWO AXIS METHOD OF SAMPLING AND PRESENTATION OF DATA SEEMS TO BE ADEQUATE FOR MOST ENGINEERING PROJECTS. IF VERY DETAILED INFORMATION IS REQUIRED THE THREE ANIS METHOD IS NECESSARY.



THREE AXIS	TWO AXIS	ONE AXIS
THREE STEREOGRAPHIC NETS	TWO STEREOGRAPHIC NETS	ONE STEREOGRAPHIC NET
ONE FOR EACH SAMPLING DIRECTION. COMPOSITE OF THREE NETS REQUIRED. ONLY JOINTS INTERSECTING A SAMPLING DIRECTION AT AN ANGLE GREATER THAN 45 DEGREES WILL BE USED.	ONE FOR EACH OF TWO TEST TRENCHES AT RIGHT ANGLES TO EACH OTHER. COMPOSITE OF TWO NETS REGUIRED. ONLY JOINTS WITH A STRIKE WITHIN 45 DEGREES OF TEST TRENCH ORIENTATION WILL BE USED.	
CORRECTION P=SIN I SIN D P=COS D	CORRECTION P=SIN I SIN D	CORRECTION N(90)=#/SIN D
SMALL TO MODERATE CORRECTION REQUIRED. ACCURACY OF LOW ANGLE PLANES DEPENDS ON RECORDING METHODS.	MODERATE TO SEVERE CORRECTION REQUIRED FOR PLANES HAVING A DIP ANGLE LESS THAN 30 DEGREES.	SEVERE CORRECTION REQUIRED FOR PLANES INTERSECTING TEST TRENCH AT ANGLES LESS THAN 45 DEGREES, AND/OR HAVING A DIP ANGLE LESS THAN 45 DEGREES.

NUMBER OF SAMPLING POINTS REQUIRED TO PRODUCE RELIABLE INDICATIONS OF A JOINT POPULATION

The following discussion considers random sampling of populations containing three, six, or 11 families each. In random sampling, each member of the population is given an approximately equal chance of being counted. No distinction is made between individual members due to width of opening, spacing, sharpness, ease of measurement, color, etc. While the sampling of joints or planes of weakness may not be totally random, it should be the objective of the sampler to make it as nearly so as possible.

Figure 9 is a graph indicating the percentage of variation from the constructed distribution of a modeled population, plotted against the number of members counted. Note that as the number of points increases, the percent of variation decreases. An empirical classification is given of the adequacy of a given number of points to provide a reliable indication of the joint population. This type of random sampling is geared more to the gaming tables of Reno than to joint sampling. However, the system may be useful.

METHODS OF CORRECTION

General

The general method of analyzing geological separations in the not-too-distant past has been to accept the field data as representative of the geometric population. However, in 1965, Ruth Terzaghi published an article that has influenced changes in this concept. She described many sources of errors in joint surveys and suggested methods of correcting the data so it would be more representative of the true geometric conditions. The joints exposed in outcrops are a function of the angle of intersection between the joint plane and the direction of observation or measurement. These exposed planes generally do not represent the same geometric relationship as they would if they were observed at right angles. This geometric relationship is indicated in Figure 10.

FIGURE 9 SAMPLING OF POPULATION - APPLICATION TO JOINT SETS. THE REPRESENTATION OF THE SAMPLED DATA TO THE MODELED VALUES IS INDICATED.

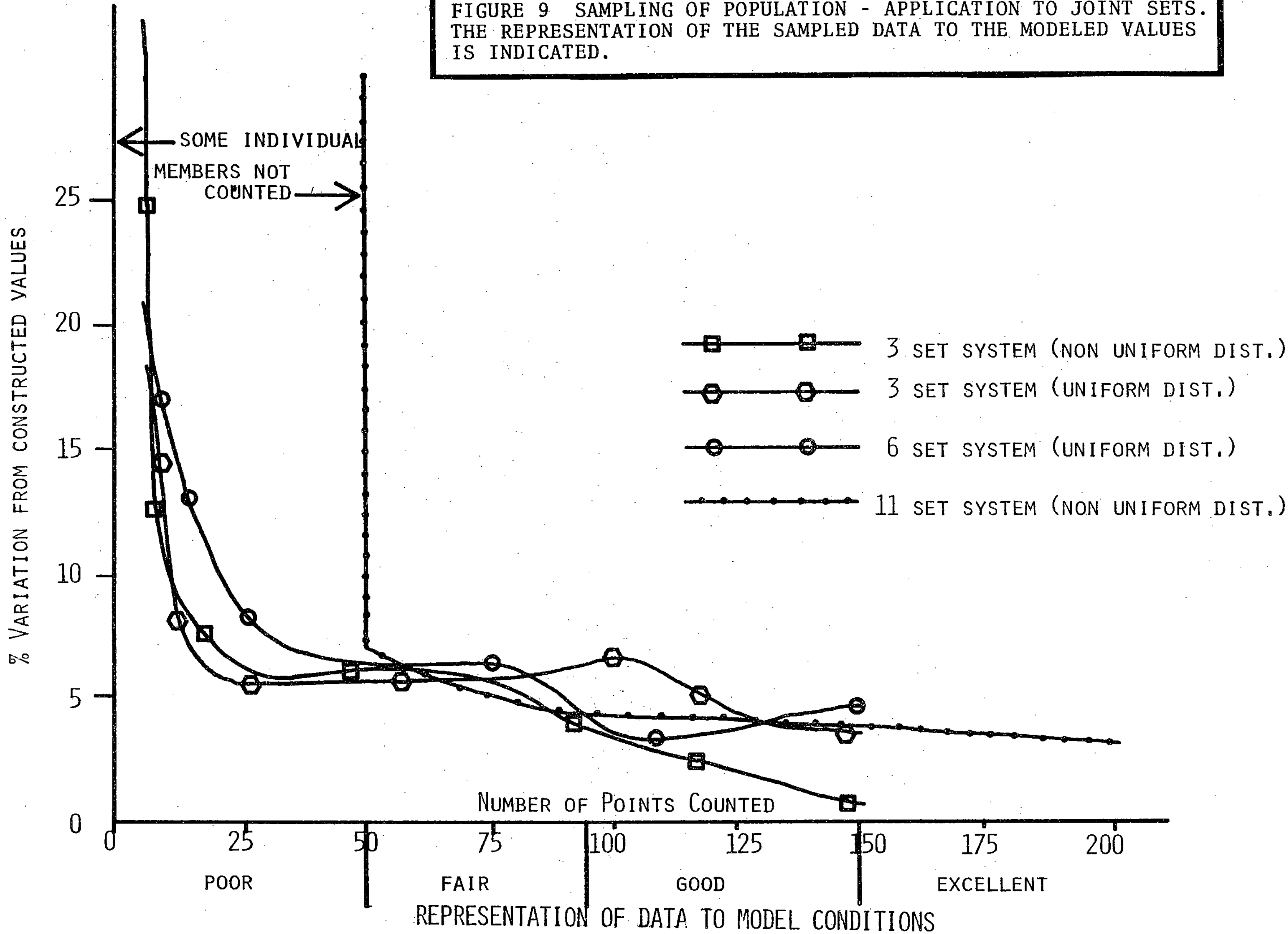
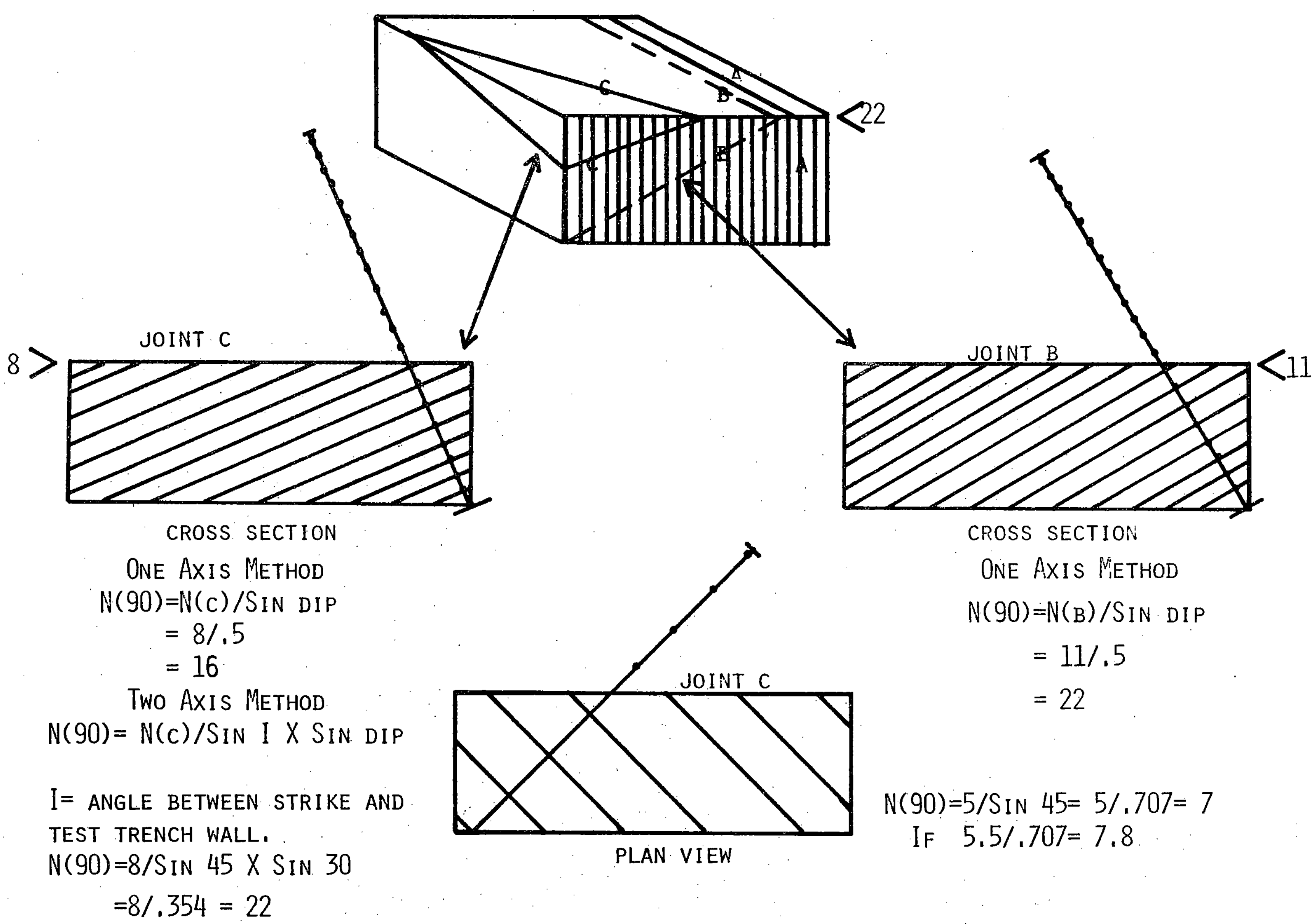


FIGURE 10 METHODS OF CORRECTION. THE TWO METHODS OF CORRECTION HAVE BEEN USED TO ANALYZE THE SAMPLED MODEL DATA. SEE TABLE I.



The joint set represented by the letter "A" intersects the cut face at a right angle, with a dip of 90 degrees. This set of joints is representative of the population for that joint, since the number of joints exposed would be the same for any test trench within the general area if: 1) the test trench is at right angles to the strike; 2) the dip remains 90 degrees; 3) the spacing between the joints remains the same; 4) the length of the test trenches remains the same; and 5) the direction of observation is normal to the plane.

The joint set "B" strikes the exposed face at a right angle with a dip of 30 degrees. This joint set, as will be exposed in this test trench, cannot be representative of the population because the direction of observation is not at a right angle to the planes' surface. However, it is possible to obtain a representative count of the number of joints that would be found if a section were observed normal to the surface of the plane, and with an equal test-trench length. This value will then be a representation of the joints' true geometric population within the rock mass.

One Axis

In Figure 10, joint set "B" is a very strong, well-defined system, striking at a right angle to the sampling surface. The following one-axis method of data correction can be used to obtain an approximate evaluation of the number of joints occuring at right angle to the dip, or normal to the plane, in a length equal to the sampling surface (Terzaghi, R., 1965).

$$N(90) = X/\text{Sin } A$$

Where:

N(90) = the number of joints at right angles to the joint surface and in a section of equal length.

X = the number of joints observed in the test trench, along a horizontal line.

Sin A = the angle between the line of observation and the

dip of the plane (or the strike of the plane).
See Figure 10 where N(90) = 11/.5 = 22.

Eleven joints were recorded with the orientation of joint "B". By using the above relationship it was calculated that the field population for a section of the given length would have 22 joints of that orientation. The model was constructed with 22 joints in a 60-foot-long section. These results would indicate that a section at a right angle to the strike can yield representative data, which can be corrected. A correction factor of 2 was used in the above correction. This method may be used when making corrections for either strike or dip.

The one-axis method may not be reliable in most engineering projects because of the large potential error. One point recorded within a test trench can represent the orientation of an individual plane or the orientation of a joint set representing an unknown number of individual planes. The correction factor applied to the recorded point would vary greatly depending on the angle of intersection and angle of dip. The one axis method generally has correction values on the order of 2, 6, 11, 18, and 36.

Two Axes

In Figure 5, it is noted that TT-1 through TT-3 intersect the north-south joints at an angle greater than 45 degrees, and TT-3 through TT-5 intersect the same joint sets at an angle less than 45 degrees. TT-7 through TT-9 intersect the same joint sets at an angle greater than 45 degrees. It will be apparent upon detailed examination that any two test trenches located at right angles to each other will provide the most desirable method of sampling. Consider, for example, TT-2 and TT-6 of Figure 5. Joints 1, 2, and 3 intersect TT-2 at an angle greater than 45 degrees and joints 4, 5, and 6 intersect at an angle less than 45 degrees. The sampled data indicate that joints 1, 2, and 3 are representative of the joint population, and joints 4, 5, and 6 are not representative. Joints 4, 5, and 6 intersect TT-6 at an angle greater than 45 degrees and joints 1, 2, and 3 intersect at an angle less than 45 degrees. TT-6 yields representative data for joints 4, 5, and 6, but non-representative data for joints 1, 2, and 3. It is apparent that intersection angles less than 45 degrees do not yield representative exposures.

The following data can be obtained by sampling test trenches which are constructed at right angles to each other. The joints that intersect sampling surfaces at angles less than 45 degrees have a very high correction factor and are eliminated from the following evaluation.

	Joint Sets					
	#1	#2	#3	#4	#5	#6
	(Number of joints observed in test trenches)					
TT-1 and 5	4	3	4	4	3	4
TT-2 and 6	4	3	3	4	3	3
TT-3 and 7	3	2	3	3	2	3
TT-4 and 8	4	3	3	4	3	3
TT-5 and 9	4	3	4	4	3	4
Construction	4	3	4	4	3	4

The above method has been demonstrated (see fig. 5 and table 1) to provide reliable figures which agree very closely with the constructed values. This method will also reduce the magnitude of the correction factor that must be applied to raw data.

In Figure 10, joint set "C" intersects the sampling surface at a 45-degree angle, and has a dip of 30 degrees. However, only an apparent dip of 22 degrees is observed on the cut surface. Using the one-axis method of correction, 16 joints are calculated to represent the population of this joint (in this trench). The model was constructed with 22 individual joints present in a 60-foot-long section.

In order to obtain a more representative correction, a two-axes method is suggested. This method was derived from Kells and Stotz, Analytic Geometry, 1949, pp. 250-251. It is suggested for use with two sampling surfaces at right angles where the angle between the sampling surface and the strike of the plane is less than 45 degrees.

TABLE I

VARIATION IN JOINT POPULATION WITH CHANGES IN TEST TRENCH ORIENTATION

Test Trench No.	Joint Set No.	Number of Joints Observed	Number of Joints Constructed	Methods of Correction - Number of Joints calculated normal to plane surface (rounded off to whole numbers)		Percent Variation	
				One Axis	Two Axis	One Axis	Two Axis
1	1	4	5	5	5	0	0
	2	3	19	17	17	9	9
	3	4	10	12	12	15	15
	4	1	--------------insufficient data----------------------------------				
	5	0	--------------insufficient data----------------------------------				
	6	0	--------------insufficient data----------------------------------				
2	1	4	5	5	5	0	0
	2	3	19	17	18	10	6
	3	3	10	9	9	9	9
	4	2	5	2	7	60	40
	5	1	19	6	17	70	11
	6	1	10	3	8	70	16
3	1	3	5	3	5	30	2
	2	2	19	12	16	37	15
	3	3	10	9	12	10	20
	4	3	5	3	5	30	2
	5	2	19	12	16	37	15
	6	3	10	9	12	10	20
4	1	2	5	2	7	40	40
	2	1	19	6	17	70	11
	3	1	10	3	8	70	16
	4	4	5	5	5	0	0
	5	3	19	17	18	10	6
	6	3	10	9	9	9	9
5	1	1	--------------insufficient data----------------------------------				
	2	0	--------------insufficient data----------------------------------				
	3	0	--------------insufficient data----------------------------------				
	4	4	5	5	5	0	0
	5	3	19	17	17	9	9
	6	4	10	12	12	15	15
6	--------------------------------Results same as TT-4----------------------------						
7	--------------------------------Results same as TT-3----------------------------						
8	--------------------------------Results same as TT-2----------------------------						
9	--------------------------------Results same as TT-1----------------------------						
Vertical							
	1	2	5	4		20	
	2	17	19	17		10	
	3	9	10	10		0	
	4	2	5	4		20	
	5	17	19	17		10	
	6	9	10	10		0	

The two-axes method is as follows:

$$P = \frac{X}{\text{Sin D Sin I}}$$

Where:

P = the number of joints normal to the plane's surface;

X = the number of joints recorded in the test trench;

D = the dip angle of the recorded joint or separation; and

I = the angle between the test-trench axis and the strike of the plane.

From Figure 10, joint C: P = 8/Sin 45 X Sin 30 = 8/.354 = 22. The construction value for this joint set is 22.

The correction factor for the two-axes data is P = 1/Sin I Sin D. Since Sin I should never be greater than 45 degrees, the maximum correction of I will be 1.4. However, Sin D can range from 0 to 90 degrees and the correction values can range from 1 to +15. The correction factor is generally on the order of 2, 3, 4, 6, 8, much lower than the one-axis values of correction.

Three Axes

The three-axes method of correcting sampled data is probably the most reliable. This method uses three axes of reference (X, Y, and Z) for sampling. The data-correcting factors are reduced to a minimum. Three stereographic nets are used, one for each axis, and a composite is prepared.

The joint data as recorded on the vertical section (Z axis) are corrected using the relationship P = Cos D (dip), where D equals the angle between the viewing direction and the plane. The other two axes, X and Y, use the correction P = Sin I Sin D. It is apparent that angles less than 45 degrees require a higher correction and therefore have the highest potential error. Since D has a maximum of 45 degrees, the maximum correction will be 1.4 for the Z axis, and 2.0 for the X and Y axis. D has a maximum of 45 degrees because any plane intersecting the sampling direction at an angle less than 45 degrees is not included in the composite diagram. The three-axes method is unique in that no plane is intersected at an angle less than 45 degrees by a reference or sampling direction.

In the two-axes method, two test trenches (X, Y) are used for sampling. No planes are included that do not intersect the test trench at an angle greater than 45 degrees. However, dips less than 45 degrees are included. With a vertical direction of observation (Z axis) dips less than 45 degrees will be intersected by a sampling direction at an angle greater than 45 degrees. Thus all planes will be intersected by a sampling direction greater than 45 degrees.

The results of the model study as presented in Table 1 indicate that the vertical direction may not be required unless extreme accuracy is desired. For the two right-angle test trenches, and the two-axes method of correction, sufficient joints must be recorded to provide a basis for correction calculations.

The three different methods of analysis were used to analyze data obtained from the test trenches constructed in the model (fig. 5). The results are indicated on Table 1. It is interesting to note that calculations by the one-axis method were reliable within approximately 20 degrees of an angle normal to the sampling surface.

The two-axes method produced some values which are not considered to be acceptable. In TT-3, joints numbered 2, 3, 4, and 5 indicate variation values of 15, 20, 15, and 20 percent respectively. With an increase in number of constructed joints from 19 to 37 for joints 2 and 4, and an increase from 10 to 38 for joints 3 and 5, the percent of variation declined. The revised variation is 5 percent for joints 2 and 4, and 11 percent for joints 3 and 5. The percent variation was also reduced for joints 2 and 3 (test trench 1), and 5 and 6 (test trench 5), by this same method. The revised percent of variation is 8 percent for joints 2 and 3 (test trench 1), and 8 percent for joints 5 and 6 (test trench 5).

The results of this study indicate that the two- and three-axes methods produce acceptable values if the following conditions are met:

1) Planes that intersect a sampling surface at an angle less than 45 degrees or having a dip less than 45 degrees should

not be considered as reliable as those planes having angles greater than 45 degrees. This is to reduce the magnitude of the correction value, 1.43 X 4 as compared to 11.8 X 4.

2) A sufficient number of joints must be counted to obtain a reliable sample.

CONCLUSIONS

Geologists and engineers using joint data should become familiar with sampling procedures, methods for correcting sampled data, method for presenting data in statistical terms, and the degree of error inherent in sampling procedures. The degrees of error or accuracy required should be determined for each individual project.

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GEOLOGIC FACTORS IN DESIGN OF EXCAVATED ROCK SLOPES

Robert B. Sennett*

Many times we have seen unstable slope conditions in rock cuts on highways. These are generally in the form of broken rock on the road itself or in the ditches alongside. If you happen to be a tourist, you would growl about the maintenance department and go on. However, if you are a highway engineer or an engineering geologist, you should ask why this has happened.

Rockfalls and slides have been a problem in the past and will continue to be problems in the future; however, proper design and construction techniques can reduce the number and magnitude of the falls and slides.

Philbrick (1963) lists two major factors that must be considered in the design of rock slopes. These are 1) the engineering requirements and, 2) the geologic conditions involved in the proposed cut.

The engineering requirements are:

1) Purpose - The design of the cut is based primarily on its ultimate use.

2) Physical size of the cut--depth, width, and extent. After the purpose has been established, the engineer must decide on the limits of these factors to make a complete design.

3) Geographic location - The climatic conditions vary from one geographic area to another. This variation is extremely important to the design of the excavation because it involves precipitation, freeze-thaw cycles, weathering, soil cover and ground-water conditions.

4) Topographic location - There is considerable difference in weather, ground-water movement, and soil cover in the same rock material when it is found at different elevations and in the same geographic area.

5) Orientation of the cut - The location of the cut with respect to the sun gives the conditions of exposure that the

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slope will undergo. Some of these conditions are wet-dry cycles, freeze-thaw cycles and vegetation growth; all are closely related to satisfactory slope design.

6) Esthetics - A long overdue requirement that must not be overlooked.

The geologic conditions are:

1) Materials - Proper identification of the rock and its physical and chemical characteristics are vital in the design of rock slopes.

2) Shear strength of the rocks - This information describes the competency of the rock material itself.

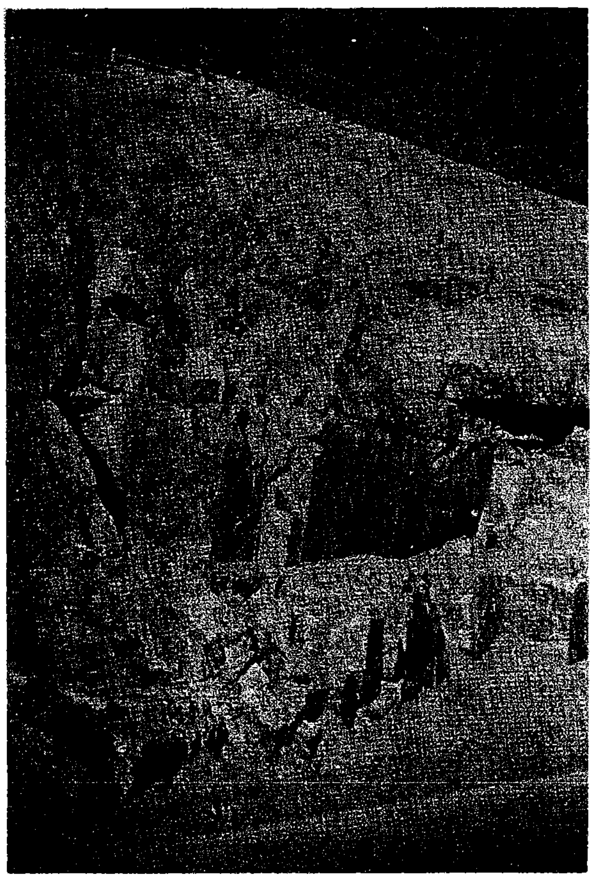


Plate 1. Granite rock exposure south of Deckers, Colorado. This shows the different joint systems created due to shrinkage of the rock material as it cooled. Also, note the upper portion of the back-slope where the same rock has been weathered.

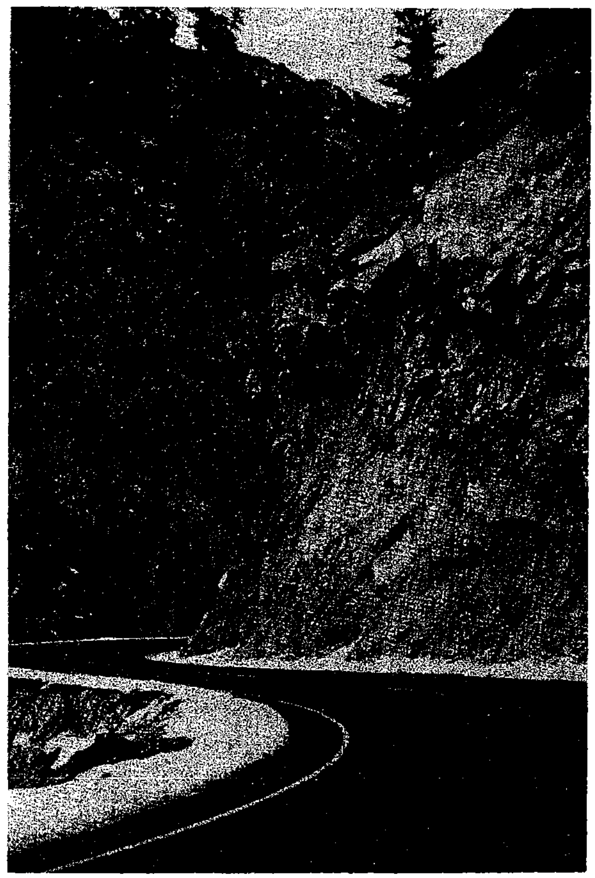


Plate 2. Igneous rock exposure south of Ouray, Colorado. Another detrimental cooling-crack joint system in the intrusive igneous rock.

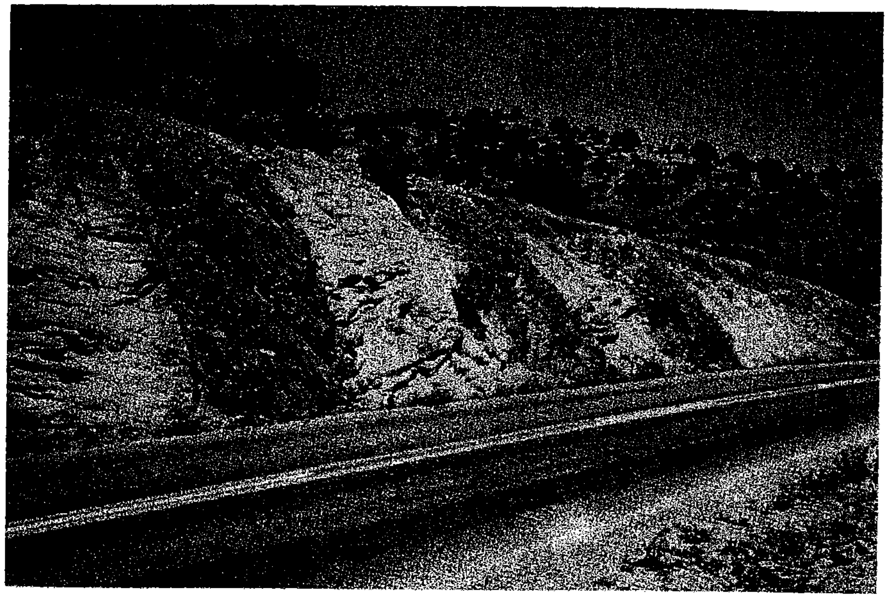


Plate 3. Four igneous dikes intruded into a nearly horizontal sandstone formation, exposed in the backslope of one cut east of Carrizozo, New Mexico. The dikes are very competent and the sandstone is rather soft and easily eroded.

3) Strength of the structural planes - The overall strength of a rock formation is decided by the strength across and along the joints and fractures in the rock.

4) Rock segments - Determination of the size, shape, orientation, and stability of the rock segments is required prior to the design of a rock slope.

5) Depth of weathering - This factor depends on the rock type, the size and number of fractures, climatic conditions, and topographic location.

6) Condition of existing cuts - An investigation is not complete until all of the existing cuts in the area have been studied and evaluated.

7) Water table and subsurface drainage - It is imperative that the engineer recognizes the fact that the proposed cut may be affected by the ground-water table and the subsurface drainage. If the cut is deep enough, it will certainly be affected.

This paper was not written to be a detailed study in rock mechanics or the geologic conditions involved in a particular rock cut. Its primary purpose is to alert the reader to the problems concerning geology that must be resolved in the design of rock cuts.

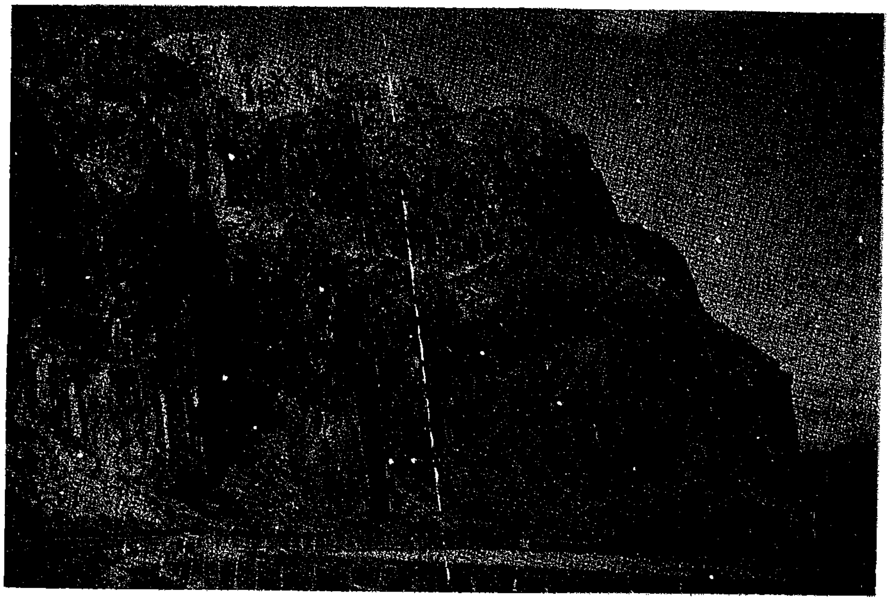


Plate 4. Extrusive igneous rock area 20 miles east of Salina, Utah. Detrimental cooling-crack joint system in extrusive igneous rock. The columnar jointing of the basalt resulted from cooling of the lava at the earth's surface. No matter what system of blasting was used, vertical or near-vertical slopes would result with breakage occurring along the joints. Any spalls that work loose fall directly toward the road.

The previous discussion of engineering requirements and geologic conditions point out, in general terms, the problems involved in the design and construction of excavations in rock. However, the geologic condition and characteristics of rock materials are fundamental to rock-slope design and will be presented in more detail.

Designers of rock cuts are faced with a large variety or rock types, thus making a knowledge of petrology very important to the successful design of rock slopes. The three main types are igneous, sedimentary, and metamorphic.

Igneous rocks are those formed from the cooling and solidification of molten rock material. There are two general types of igneous rocks - intrusive and extrusive.

Some intrusive rocks are cooled slowly within the earth, creating a coarsely crystalline rock such as granite. More rapid cooling within the earth creates finer grained rock such as is commonly found in dikes (Plates 1, 2, and 3). Extrusive igneous rocks are cooled rapidly on the earth's surface as lava flows or volcanic ejecta. The flow rocks, such as basalt, are finely crystalline. Ejecta can be either volcanic ash or breccia (Plates 4 and 5).

Plate 5. Extrusive igneous rock area east of Prescott, Arizona. Compound slope produced from a cut through an area where highly jointed basalt lavas were extruded over volcanic ash. Ash weathers more readily, creating a maintenance problem in the ditches by allowing chunks of basalt to fall. Construction problems are apparent.

Sedimentary rocks are those composed of material deposited from water, ice, or air at ordinary temperatures. They include shale, sandstone, limestone, and conglomerate (Plates 6 and 7).

Metamorphic rocks are formed by alteration of any pre-existing rock by heat or pressure, or by a combination of both. Examples of metamorphic rocks are schist, gneiss, quartzite, and marble (Plates 8 and 9).

In addition to the basic rock types, the material directly associated with them, such as residual soil, and colluvial or alluvial deposits.

To be considered along with the basic rock types and their associated materials are the characteristics of the rocks such as unit weight, porosity, the physical and chemical make-up of the rock, the overall structural deformation present, and stresses involved within the rock.

The unit weight depends on the specific gravity of the constituents, the porosity, and the amount of water contained in the pores of the rock. Rocks having a high unit weight, such as granite, are generally more competent than rocks having a low unit weight, such as volcanic tuff. Physical and chemical action in the form of

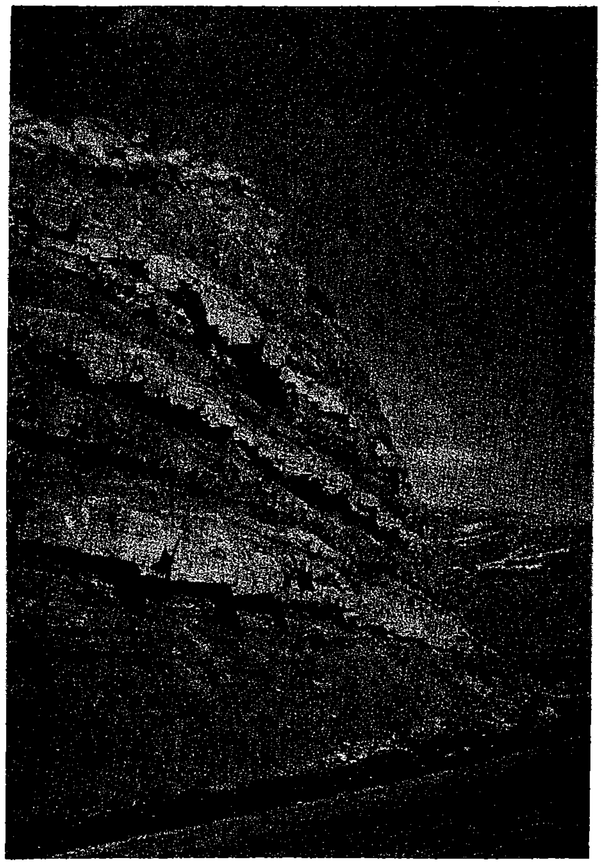


Plate 6. Interbedded sedimentary rocks northeast of Castle Gate, Utah. Softer shales are eroding rapidly allowing the jointed sandstone to fall. All material here is rippable and due to shale erosion characteristics, the slope was designed too steep.

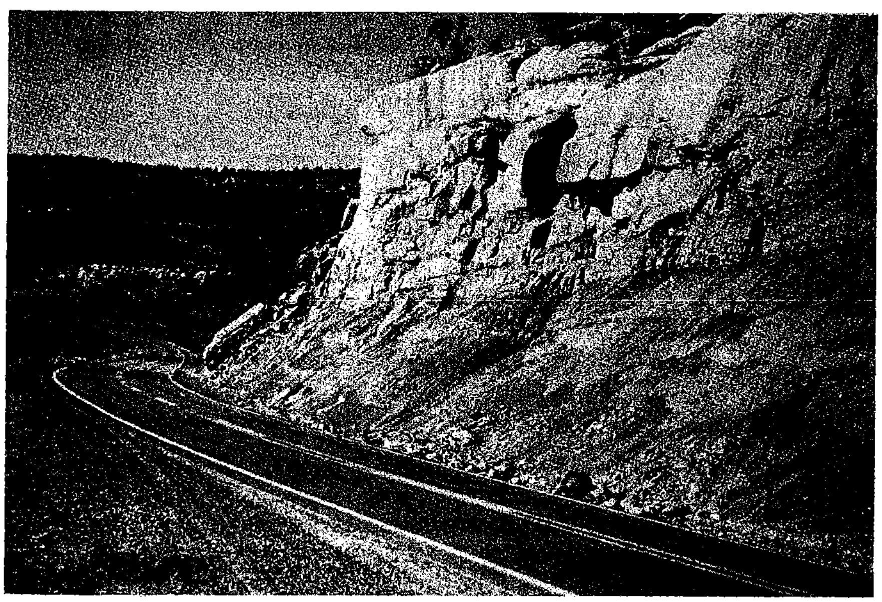


Plate 7. Sandstone overlying a soft shale south of Durango, Colorado. Erosion of the shale allows the jointed sandstone to fall.

Plate 8. Metamorphic-rock exposure northeast of Bailey, Colorado. Exposure clearly shows the joint patterns and foliations occurring in a gneiss, with backslope construction and maintenance problems apparent. Note rock hammer in center of picture.

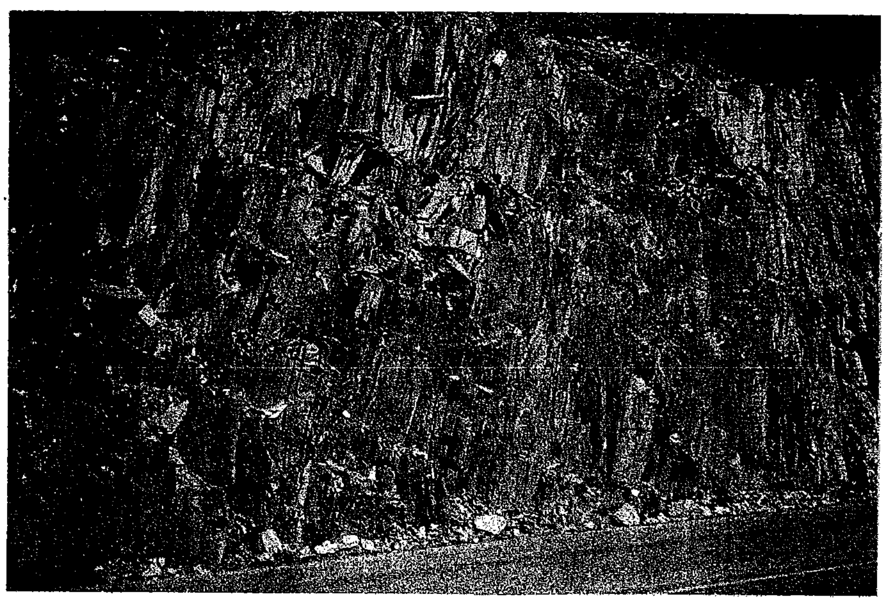
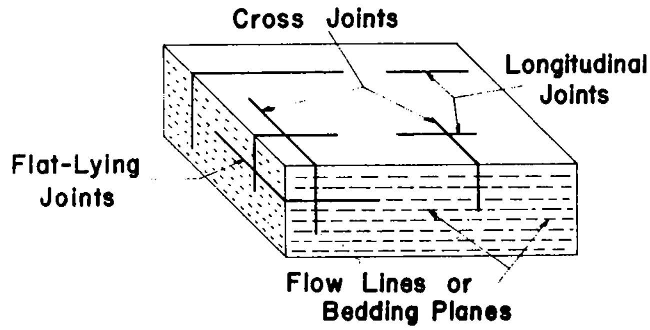


Plate 9. Metamorphic-rock exposure east of Prescott, Arizona. Exposure of schist where foliations are nearly vertical and a joint system is nearly horizontal.



BASIC JOINT PATTERNS
FOUND IN ROCKS

Plate 10. A joint is a fracture or parting plane along which there has been very little, if any, movement. They can occur individually but are more commonly found in sets and systems.

weathering can reduce the quality of the more competent rocks to a point where they become incompetent.

The porosity of a rock is the ratio of the volume of voids (pores) to the overall volume of the rock specimen. This characteristic is important to slope design and rock work in general, because a change in the moisture content of the rock affects its compressive strength. An increase in water content decreases the strength.

When considering rock structure in cut-slope design, the word structure is used in two different ways. One is the structure within the rock, such as bedding, foliation, and jointing (Plate 10). The other is the over-all structural geology of the area, such as folding, faulting, and tectonic movement (Plate 11).

In design and construction, both of these characteristics are important, and in some cases work together to cause unstable rock-slope conditions. An example is a rock cut in upturned sedimentary rocks where fractures along a bedding plane allow broken rock to slip along the bedding plane, and then to fall into the cut (Plates 12, 13, 14, 15 and 16).

Rock deformation include joints, faults, and folds. Joints or fractures in rocks are generally caused by tensile stresses. These stresses may be caused by a decrease in volume resulting from a drop in temperature or reduction of the moisture content, by

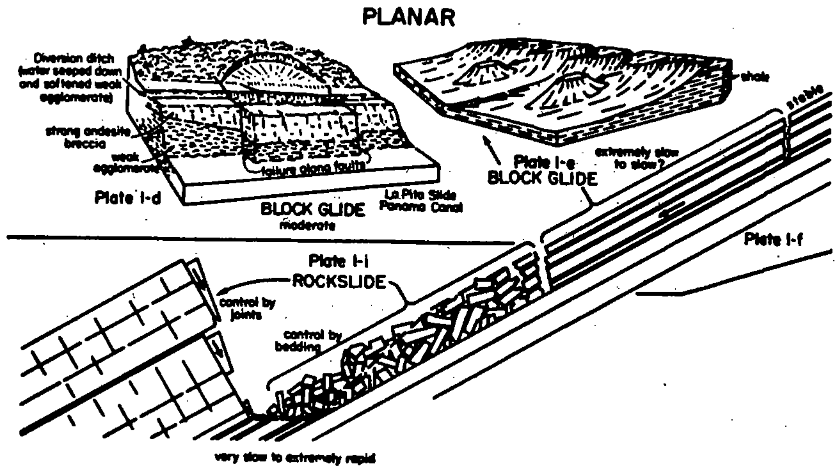


Plate 11. Drawing showing how a rock slide can occur along bedding planes on right side of road cut while rock falls occur along joints on left side of cut. From David J. Varnes, "Landslide Types and Processes," HRB Special Report Number 29.

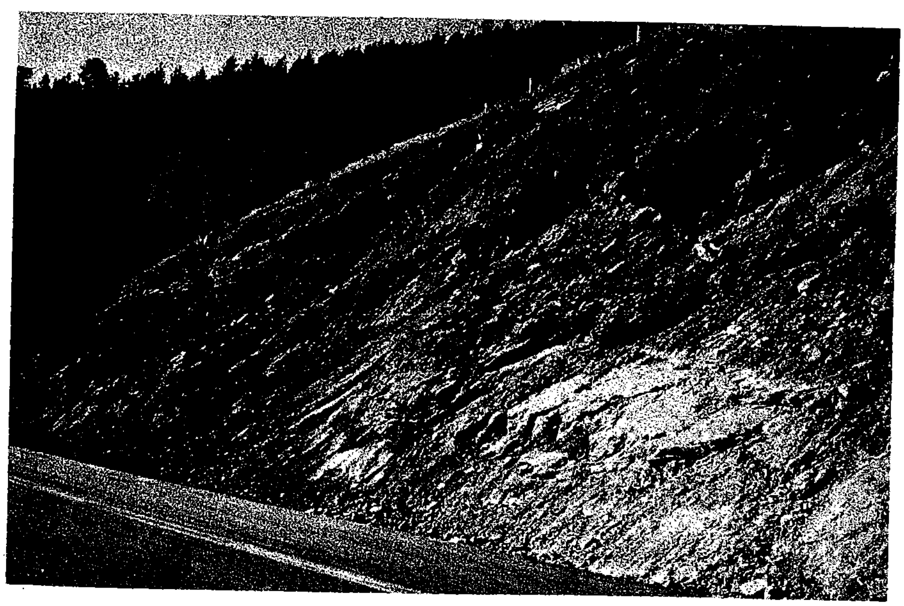


Plate 12. Upturned sedimentary rocks west of Dayton, Wyoming. Example of a bedding plane rock slide.

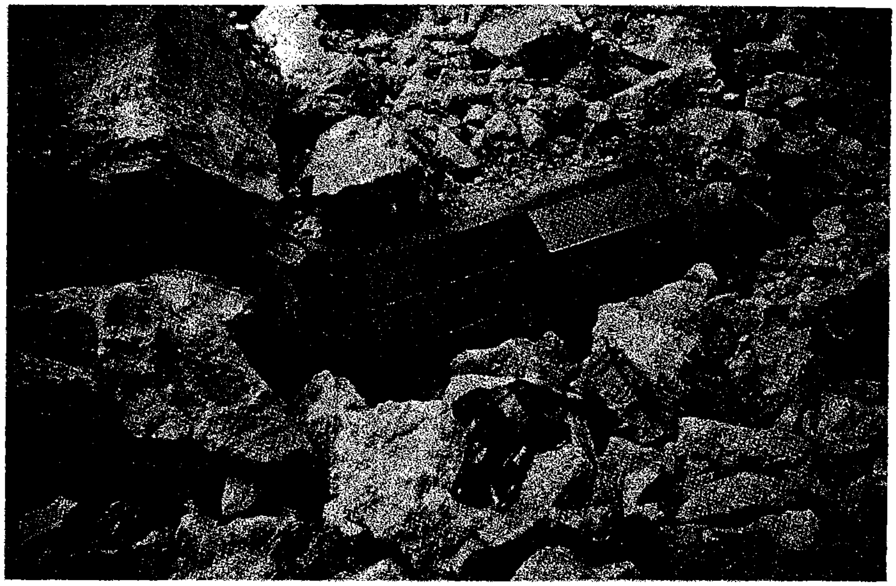


Plate 13. Compressor covered by rock slide. Slide was a bedding plane failure as described in Plate 11. Overall view of this failure shown in Plate 14.

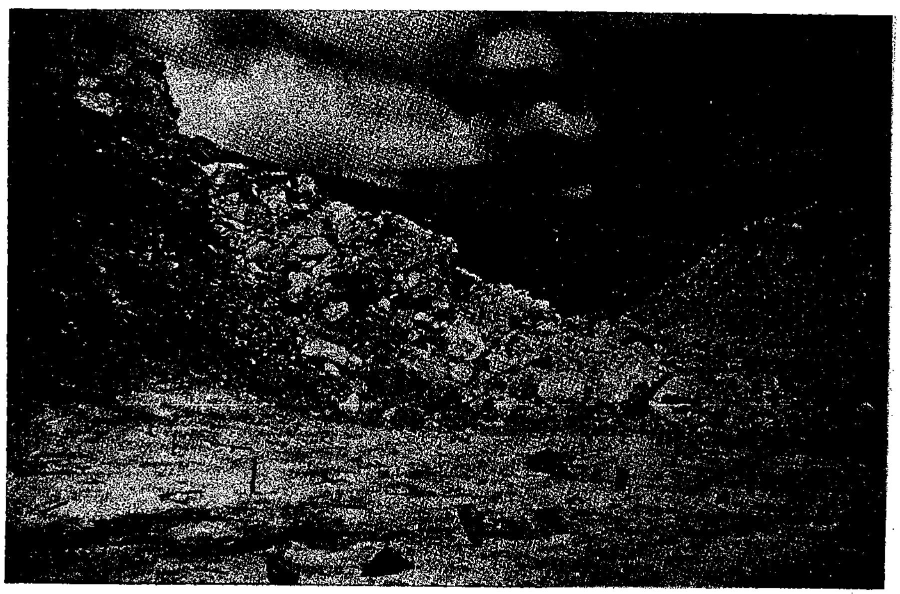


Plate 14. Bedding plane failure in limestone on road-construction project on east side of the Big Horn Mountains in Wyoming.

removal of lateral support or as a result of an uplift, or by a fold-type movement of the earth's crust. Jointing usually has a definite relationship to the bedding or flow lines of the rock (Plates 1, 4, 6 and 9).

Faults are fractures along which the opposite sides have been displaced relative to one another. Some of the factors to be considered when faults are encountered in cuts are: 1) slope stability problems caused by two or more completely different rock types on opposite sides of the fault; 2) the possibility of future movement along the fault; and 3) excess water in the fault zone itself (Plates 17, 18, 19 and 20).

Folding of rock material is caused by bending of the earth's crust due to the movement of molten magma beneath. Non-brittle rocks bend and fold under this movement, while brittle rocks usually break, producing joints and faults.

When reference is made to a joint, fault, bedding plane, foliation, or other phase of structural geology, the terms dip and strike are used. These terms are defined as follows: Dip is the angle which a stratum, vein, fissure, fault, or similar geological feature makes with a horizontal plane, as measured at a right angle to the strike (Plate 21). Strike is the direction of a line formed by the intersection of a bedding plane, vein, fault, slaty cleavage, schistosity, or similar geologic structure, with a horizontal plane. It is measured at a right angle to the dip (Plate 21).

Since stresses have been mentioned previously in the discussion of jointing and faulting of rock, these terms should be defined:

1) Compressive stress is a stress that tends to decrease the volume of the material.

2) Shear stress is a stress that tends to move one part of a specimen with respect to another or to make the rock material flow.

3) Tensile stress is a stress that tends to produce cracks by pulling the specimen apart.

4) Torsion is sometimes present in rock. It tends to twist the rock material.

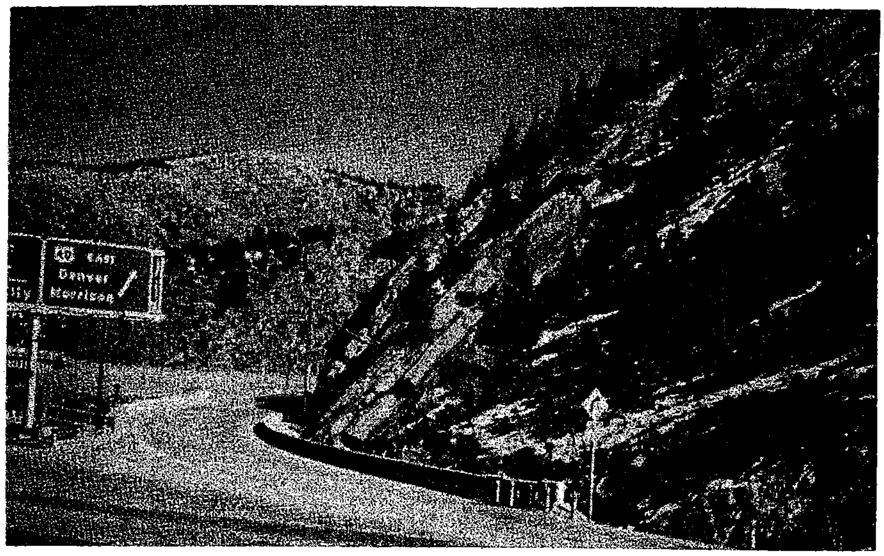


Plate 15. Rock cut east of Idaho Springs, Colorado. Showing rock-slide area as described in Plate 11. Road is on down-dip side of the foliation planes in the rock.

Plate 16. Rock cut east of Idaho Springs, Colorado, on opposite side of canyon from Plate 15, showing up-dip side of the same rock formation. This slope is nearly vertical whereas slope in Plate 15 is following the foliation plane.

Generally, when a rock mass is subjected to compression, the action is one that has three stresses acting at the same time, that is, compressive, shear and tensile stresses.

Another phenomenon found in some rock areas is residual stress. Residual stresses are caused by excessive loading or some other disturbance, either natural or man-made (such as blasting), which affect the rock for a period of time after the load has been removed. Residual stress may cause movement of rock into a cut or excavation because of removal of lateral support by excavation (Plate 22).

Several general comments may be made concerning strength of rock materials:

1) Strength decreases as lateral restraint decreases. Removal of rock from a cut reduces the strength of the rock in the slopes.

2) Compressive strength depends on the direction of the acting stress. With relation to bedding or foliation, the highest strength is obtained when the load is normal to the bedding or foliation. If it is applied parallel to bedding or foliation, the material has a tendency to shear.

3) Compressive strength is influenced by the texture and crystallinity of the rock. Finer-grained material generally has greater strength.

Crystallinity refers to the size and orientation of crystals within the rock. Strength is generally greatest when the crystals are small and closely spaced. Here again, strength depends on the loading of the material and the crystal orientation.

4) The bonding material contained in some rocks (generally sedimentary rocks) influences strength. For instance, sandstone bonded with silica is much stronger than sandstone bonded with calcium carbonate.

5) Interbedding of shale with sandstone or limestone creates planes of weakness in otherwise competent rock.

It must be remembered that the foremost principle of slope design is to fit the slope to the rock material, not the rock

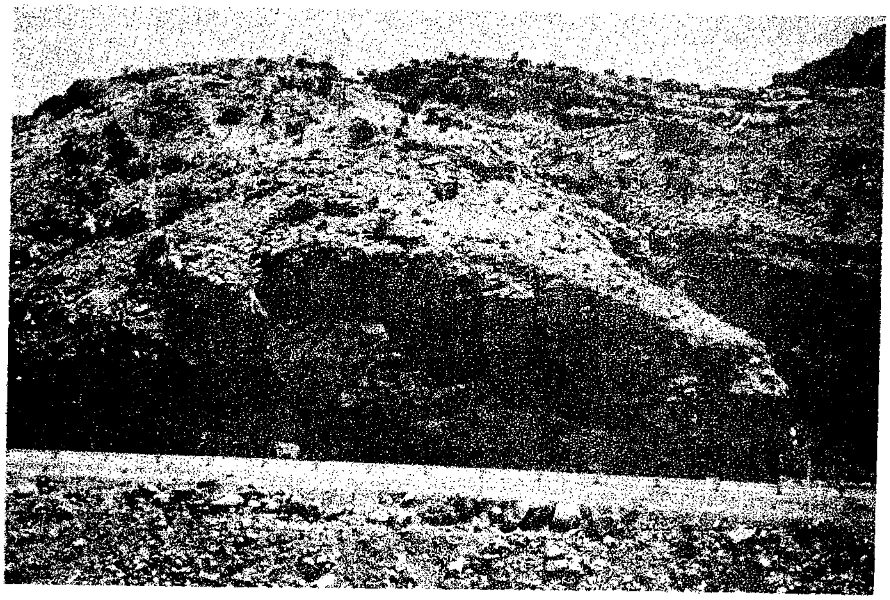


Plate 17. Faulted sedimentary rocks near entrance of Arches National Monument in Utah. Changing rock types due to faulting can create slope stability and construction problems.

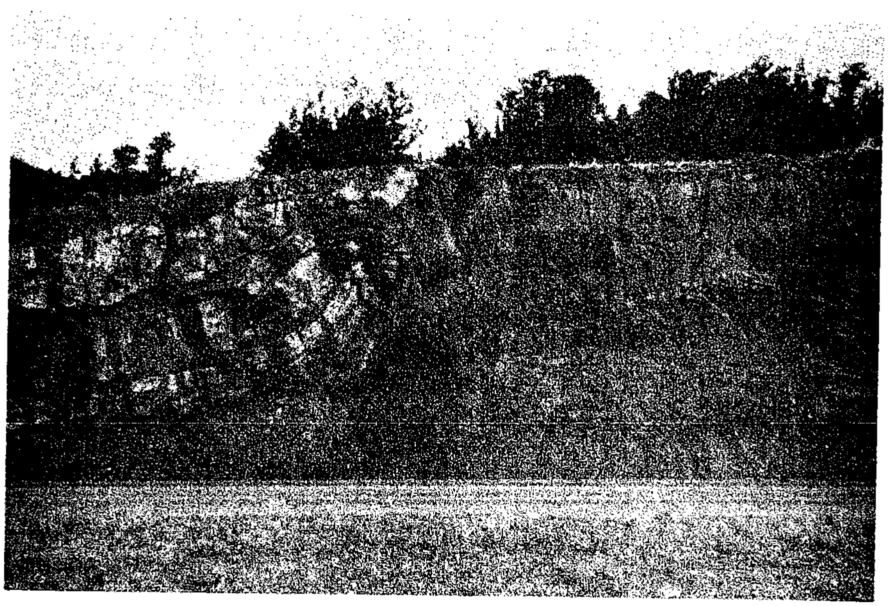


Plate 18. Faulted sedimentary rocks east of Albuquerque, New Mexico. Changing rock types due to faulting can create slope stability and construction problems.

material to a predetermined slope. When this principle is adhered to, competent results can be attained in the design and construction of rock slopes, (Plates 23, 24, 25, 26 and 27).

Plate 19. Folded sedimentary rocks southeast of Taos, New Mexico. Fracturing of rock due to folding can create slope-stability problems.

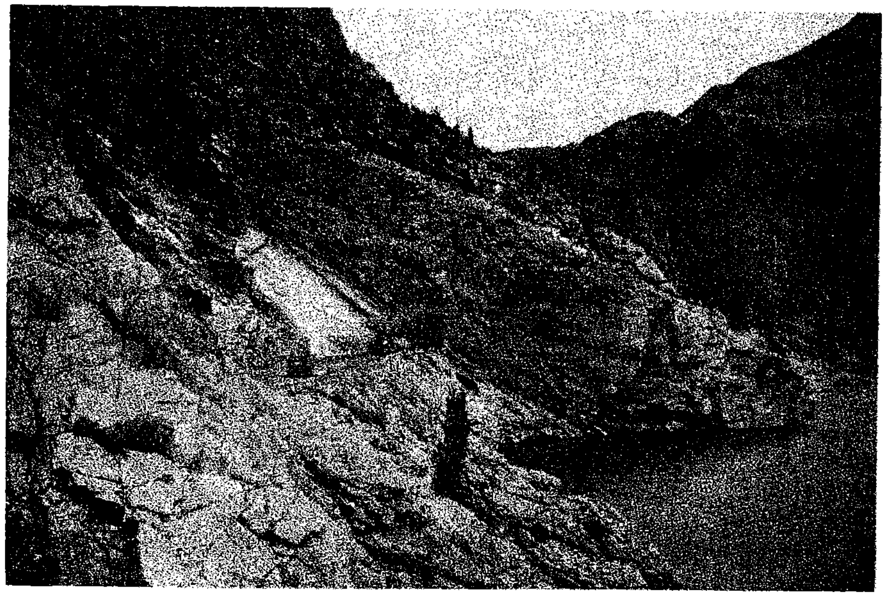
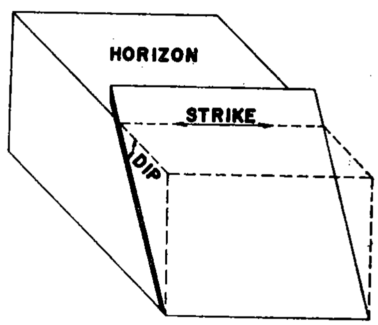


Plate 20. Upturned, horizontal, and faulted sedimentary rocks west of Cody, Wyoming. Bedding-plane failure occurred in dipping rocks to left of trucks. Note the thick layer of limestone creating top of tunnel and above that, the broken rock mass created by a fault. Note the horizontal limestone layers. Slope stability problems very apparent.



ATTITUDE OF BEDS

Plate 21. The dip is the angle which a stratum, sheet, vein, fissure, fault or similar geological feature makes with a horizontal plane, as measured in a plane normal to the strike. Strike is the direction of a line formed by the intersection of a bedding plane, vein, fault, slaty cleavage, schistosity, or similar geologic structure, with a horizontal plane. Strike is measured at right angles to the dip.

Plate 22. Example of a sandstone being over-shot during cut excavation. Over-shooting may create residual stress zones in the rock which could cause spalling as these stresses are relieved.

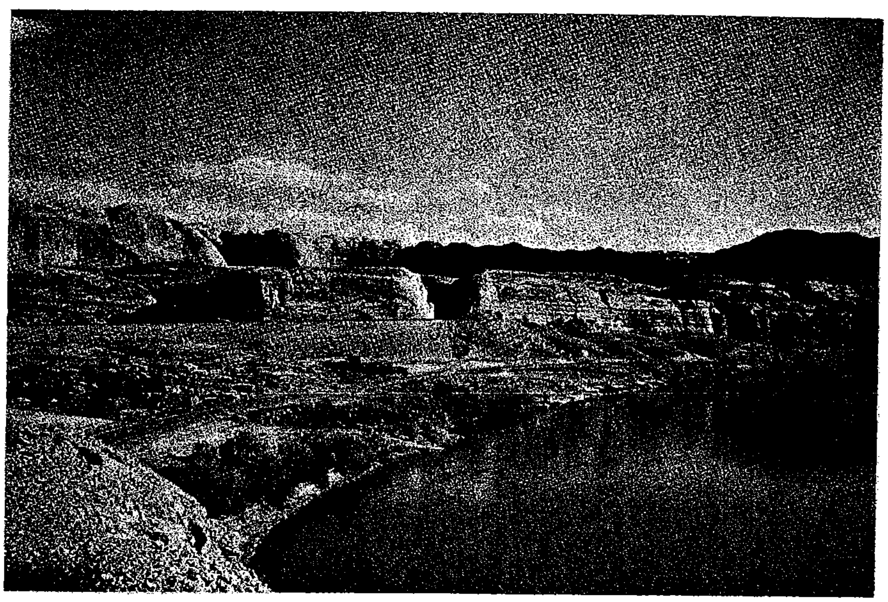


Plate 23. View of a pre-split cut for a railroad west of Moab, Utah. This cut is truly perfect and is due to good workmanship and a nearly homogeneous sandstone with a minimum amount of jointing and weak interbedded material. The entrance to the pre-split cut is near center of photo, and the Colorado River is in the foreground.

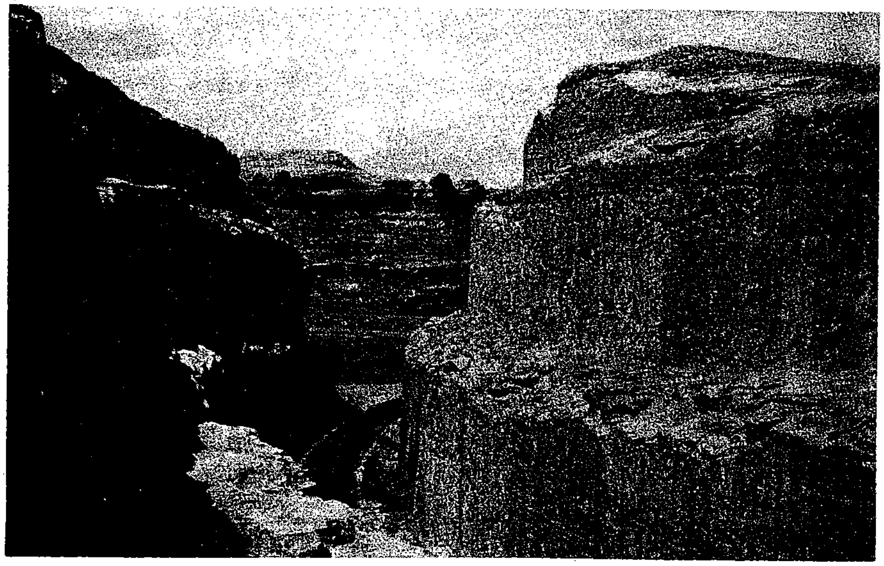


Plate 24. The same pre-split cut as shown in Plate 23. From the top of the cut looking west toward the Colorado River. This view shows the near-perfect, vertical-walled, pre-split cut.

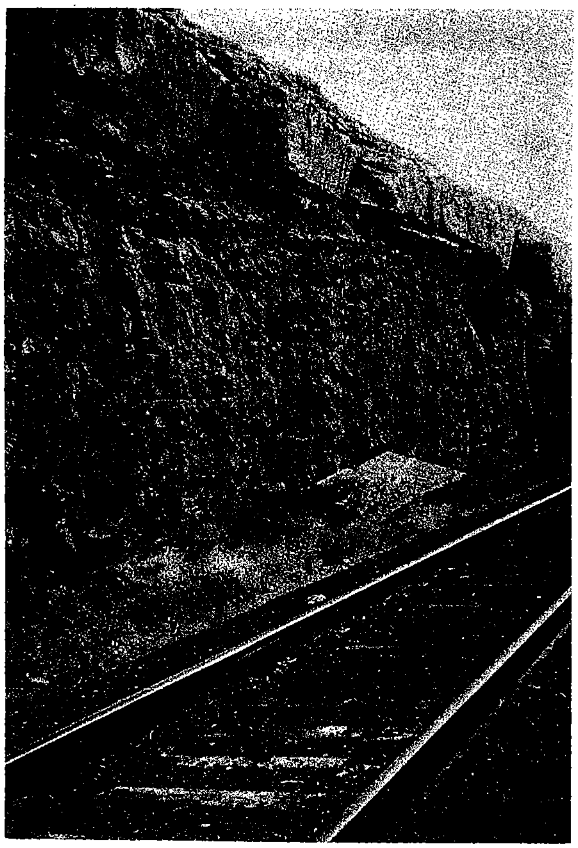


Plate 25. Rockfalls due to the combination of jointing in the sandstone and weak, easily eroded layer working out from under the sandstone. This is a view of a pre-split railroad cut north of Moab, Utah. This cut is an example of the maintenance problems arising from cuts of this type due to the type of rock involved and not due to poor workmanship, and the like.

Plate 26. View of a pre-split railroad cut south of Santa Fe, New Mexico. This cut was placed through a highly faulted zone with very satisfactory results. Even though considerable shattered rock areas were noted throughout the cut, very little rock has fallen since construction. Note the elevation changes of the reddish sandstone in the lower bench area. It is a series of grabens and horsts.

Plate 27. The same pre-split cut as shown in Plate 26. The whitish-gray material on the right is a gypsum deposit which was pre-split along with the faulted sediments on left.

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HYDRAULIC BORROW MATERIALS IN URBAN AREAS

Gomer Jenkins, Jr.*

ABSTRACT

This paper discusses the use of Missouri River sand as the most economical borrow available in large quantities for highway embankment construction in an urban area of high land values.

Design specifications and construction procedures are reviewed.

INTRODUCTION

One of the last projects on Interstate Route 435, a six-lane circumferential route by-passing Kansas City, Missouri, on the south and east, is a 2.8-mile section extending from the Missouri River south to U. S. Route 24. The project, when completed, will give a functional highway facility from Interstate Route 35 in Lenexa, Kansas, to Interstate Route 35 in Claycomo, Missouri.

This section of Interstate Route 435 traverses the Missouri River flood plain for 2.4 miles of the 2.8-mile project. Access to construct the embankment is complicated by the presence of six railroads, two city streets, the Big Blue River, and a railroad switching terminal for Armco Steel Corporation. The Missouri River at the north end of the project acts as a natural barrier to access from that direction.

Required borrow for construction materials to complete the project as designed is 1,900,000 cubic yards. Numerous borrow possibilities were considered during the preliminary investigation. The Missouri River flood plain from the Corps of Engineers' levee south to the loess bluffs was eliminated for borrow. It consisted of silty clay to clay (gumbo) soil mantle 2 to 3 feet deep underlain by 15 feet of a fine, silty, blow sand. The latter had a gradation range of 90 percent $\pm$ passing the number 80 sieve and 20 percent $\pm$ passing the number 200 sieve. Silt and clay contents ranged up to 16 percent. The deposition was very erratic with silt- and clay-filled meander channels occurring randomly throughout.

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*District Geologist, Missouri State Highway Commission, District 4 Kansas City, Missouri.

The land adjacent to the right-of-way was zoned for heavy industry with an appraised value of $1.00 to $1.50 per square foot. Side borrow of 1,500,000 cubic yards to a maximum depth of 3 feet would be prohibitive in cost.

The loess bluffs near the south end of the project and at another site north of the Missouri River contained excellent borrow materials having thicknesses ranging up to 100 feet. The loess borrow could be obtained for a royalty of 7¢ per cubic yard. Estimated cost of the loess in the compacted embankment, with proposed fill heights from 6 to 40 feet, was $1.50 per cubic yard, plus. Preliminary estimates from earth moving contractors determined that an average cost of $1.00 per cubic yard per haul-mile was a realistic figure for the loess embankment. Average haul distances from either of the proposed borrow sites were 1.4 miles.

The sand deposits on the south bank of the Missouri River were investigated and eliminated because of the presence of 15 feet of fine, silty, blow sand cut randomly by clay veins and meander channels filled with organic material. The Missouri River flood plain has a high water table which fluctuates with the water level of the river and prevents excavation by normal methods. These factors led to investigation of the possibility of using hydraulic borrow from the Missouri River.

Final core borings in the Missouri River channel established the presence of a well-graded clean sand approximately 15 feet below the stream bed and extending to depths of more than 50 feet. Preliminary checks of two small hydraulic sand projects that had been constructed in this part of Missouri revealed that they were well-compacted and performing in a structurally sound manner.

Preliminary cost inquiries of contractors in the area and correspondence with other governmental agencies revealed that the hydraulic sand borrow in place should range in cost from 70¢ to 85¢ per cubic yard for a maximum pumping distance of 2.0 miles. Most contractors stated that larger borrow quantities would result in more favorable per-yard costs; a minimum of 1,000,000 yards of hydraulic borrow was frequently mentioned.

Hydraulic borrow has the following advantages: elimination of necessity for mechanical compaction because of densification by drainage of the hydraulically placed material; no levy required for removal of sand from the river; lower construction costs compared to soil borrow transported for distances greater than 1.0 mile; elimination of the need for a large excavation in an urban area; elimination of costs of right-of-way for borrow easements; and elimination of traffic congestion and the nuisance of hauling large amounts of fill material through urban areas.

The use of hydraulic-fill material was considered as an alternate bid item for the project section from the Missouri River south to the Big Blue River, a distance of 2.0 ± miles. The short section from the Big Blue River to the bridge approaches adjacent to the loess bluffs was specified to be constructed of 269,000 cubic yards from the bluff borrow area. The topsoil capping for the hydraulic sand embankment was specified to be obtained by stripping the shallow topsoil beneath the roadway as an alternate.

The project was let in the spring of 1968 with the paving excepted. The Ted Wilkerson Construction Company of Kansas City was the successful bidder and prime contractor with earth work subcontracted to the Clarkson Construction Company. The 1,544,000-cubic-yard hydraulic sand portion of the project was subcontracted to the Clark Construction Company. The contract bid price for the hydraulic sand embankment was 80¢ per cubic yard.

The specifications for the hydraulic fill were compiled by the consulting firm of Howard, Needles, Tammen, Bergendoff, and the Missouri Highway Department. No state specifications for hydraulic sand fill were available as this method of highway construction had not been extensively used in Missouri prior to this project. The specifications required that not more than 50 percent of the material could pass the number 40 sieve and not more than 15 percent of the material could pass the number 200 sieve. The hydraulic-fill contractor stated that these gradation limits were too rigid and could not be met. However, material obtained from actual dredging operations met the gradation requirements when the sand was obtained from the graded sand 15 feet below the stream beds, as had been previously established by core sampling.

Pumping of hydraulic fill began on November 11, 1968 and was completed the first week of September, 1969. Six weeks of down-time were lost because of high water and ice flow. Using a 18-inch dredge and booster pump, the contractor placed 15,000 cubic yards of hydraulic sand embankment in 22 hours.

CONCLUSION

The use of hydraulic sand borrow for fill construction in high-land-value urban areas within the 2-mile maximum pumping distances from the Missouri River resulted in at least 70¢ per cubic yard cost differential with the loess from the adjacent river bluffs. Experience indicates that the critical specification for hydraulic sand gradation should probably be the quantity passing the number 200 sieve. The future control in Missouri has tentatively been set for not more than 10 percent passing the number 200 sieve. The 1,544,000 cubic yards placed on the I-435 project had an in-place gradation change from 1 percent to 2.5 percent passing the number 200 sieve.

Construction procedures were reviewed by a series of 35mm color slides.

CONTRACT SPECIFICATONS
PROJECT I-IG-435-1 (52) 13UA
ROUTE I-435
JACKSON COUNTY, MISSOURI

Special Provisions

I. EMBANKMENT IN PLACE

Description. This work will consist of either dredging and pumping acceptable hydraulic-excavation material from the Missouri River or excavating and hauling acceptable borrow-excavation material from other approved sources, and placing the material in certain designated sections of embankment, all in accordance with this special provision and in conformity with the lines, grades, and typical sections as shown on the plans.

Permits. Unless otherwise provided in the contract, the contractor shall, at his own expense, procure all necessary permits from proper authorities to operate dredges and other floating equipment in the waters of the Missouri River. He shall also obtain any necessary permits for the passage of a discharge pipe over private property. The Contractor shall also obtain all necessary permits for operating equipment and any other encroachment necessary on levee right of way.

Equipment. Any dredging and pumping equipment used by the contractor to dredge and pump hydraulic excavation shall be adequate to insure completion of the work within the time specified in the contract. All equipment shall be subject to approval by the engineer.

Material. Hydraulic excavation material shall consist of sand or a mixture of sand and other suitable materials, free from decayed matter, roots, stumps, logs, or other material considered by the engineer to be unfit for incorporation in roadway embankment, and meeting the following gradation:

Passing No. 200 Sieve - not more than 15 percent.

Passing No. 40 Sieve - not more than 50 percent.

Borrow excavation material shall be suitable embankment material meeting the approval of the engineer.

Identification of Earthwork Sections. For reference purposes, grading work between the Missouri River and the north end of the proposed bridge over the tracks of the GM & O and other railway companies, at Station 141+25.2, will be divided into four separate sections identified as follows:

Section	Station	To	Station
A	Missouri River	--	95+50±
B	95+50±	--	116+90±
C	116+90±	--	126+50±
D	126+50±	--	142+21.18

Embankments within the limits of Sections "A", "B", and "C" shall be formed from hydraulic-excavation material or material obtained from other sources furnished by the contractor and approved by the engineer.

Embankment within the limits of Section "D" shall be formed from borrow-excavation material obtained from a borrow area furnished by the Commission and located on the left at Station 155+00 as shown on Sheet No. 22 of the detail plans.

Dredging Operations. No hydraulic excavation material shall be obtained from sources closer than 300 feet upstream or 1500 feet downstream from the existing railroad bridge located about 0.75 mile downstream from the site of the proposed highway bridge over the Missouri River. Nor shall any hydraulic excavation material be obtained from sources closer than 300 feet up or down stream from said proposed highway bridge.

Construction Methods. If hydraulic excavation material is pumped directly into place in the embankment, the contractor shall begin at the centerline and deposit material in either direction or both directions toward the toes of the slopes and the discharge shall always be in the direction of, and along or parallel to the centerline, unless otherwise permitted by the engineer. If the engineer deems it necessary, splash-boards or dumping platforms of such size as may be required for proper reception of materials shall be furnished and used. If the discharge of material from the pipeline causes erosion or damage to existing work or adjacent property, the work shall be stopped until satisfactory methods of discharge are effected by the contractor to prevent such damage. Material shall be deposited in such manner as to maintain at all times a higher elevation at the center of the roadway than on either side. All existing drainage structures shall be kept open and free of clogging, and proper provisions shall be made by the contractor for satisfactory disposal of surplus water.

The Engineer shall have authority to reject any material considered by him to be unsuitable for use in the embankments. Any such materials excavated and the disposal of same shall be at the contractor's expense. Any soft and yielding spots in the embankments shall be removed and replaced with satisfactory material.

The Contractor will be required to assume all responsibility for compression, subsidence, displacements, or slides that may take place in hydraulic fill and no payment will be made for any additional material required to maintain the embankment in accordance with the typical sections as shown on the plans.

The discharge line shall be provided with a "Y" and diversion line properly equipped with shut-out valve or other suitable device so that unsuitable materials entering the discharge line can be diverted for disposal to a waste area. Any area required for the disposal of waste shall be furnished by the contractor at his own expense.

Compacting will not be required on embankments formed from hydraulic material pumped directly into place.

If hydraulic excavation material is temporarily stockpiled and subsequently removed and placed with conventional earthmoving equipment to form roadway embankments, the material shall be placed and compacted in accordance with Sections 21.10 and 21.20 of the Standard Specifications, except that the required density shall be at least 95 percent of maximum density as determined by the Standard Compaction Test.

Embankments formed from materials other than hydraulic excavation material shall be placed and compacted in accordance with Sections 21.10 and 21.20 of the Standard Specifications.

Embankment within the limits of Section "D" shall be formed wholly from material obtained from the aforementioned Commission-furnished borrow area, except as hereinafter provided:

At the contractor's option, a 50-foot section of embankment immediately adjacent to the north end of the proposed bridge at Station 141+25.2, may be formed from material obtained from within the right of way. However, any area within the right of way from which excavation is removed shall be backfilled with material obtained from the above said borrow area. No payment will be allowed for backfill material placed in undergraded areas.

The embankment shall be shaped to line and grade before the specified depth of capping material is placed and the final cross-section of the embankment shall be true to the lines, grades, and cross sections shown on the plans.

Method of Measurement. Final measurement of embankment in place will not be made except when errors are found in the original computations or ground elevations or when there has been an authorized change in grade or typical section. If the above exceptions are encountered, the plan quantities of embankment in place for those areas affected will be adjusted for any such change. No adjustment will be made for backfilling areas undergraded to obtain material for fill capping or for forming any portion of roadway embankment.

Basis of Payment. For the purpose of payment, all roadway embankment in Sections A, B, and C, except embankment capping, will be considered as hydraulic embankment regardless of the sources of the materials or method of construction used. Hydraulic embankment, complete in place, will be paid for at the contract price per cubic yard, which price shall constitute full compensation for furnishing any borrow pit areas, other than pits furnished by the Commission; all materials and all operations necessary to dredge and/or excavate, pump, haul, place and compact; and all equipment, tools, labor, and incidentals necessary to complete the item.

Payment for embankment in Section D, complete in place, will be made at the contract price per cubic yard which price shall be considered full compensation for all operations necessary to excavate, haul, place, and compact borrow excavation material in roadway embankment; and all equipment, tools, labor, and work incidental thereto.

Payment will be made under:

Item 21104: Embankment in Place (Borrow), per cubic yard.

Item 21105: Embankment in Place (Hydraulic), per cubic yard.

J. EMBANKMENT CAPPING.

This work shall consist of all of the operations necessary to the excavating, loading, hauling, placing, and compacting borrow-excavation material over the side slopes of hydraulic embankment to the lines, grades, and depths as shown on the cross sections of the plans. Benching for placement of capping material will not be required.

Material used for capping embankment shall be earth material meeting the approval of the engineer. Material for capping embankment may be obtained from the Commission furnished borrow pit on the left at Station 155+00; from sources furnished by the contractor; or from within the right of way by scalping between the toes of the proposed fills and stockpiling for subsequent use. If obtained by this latter method, the contractor shall backfill the scalped areas with suitable backfill material obtained from an approved source. No payment will be allowed for excavating, hauling, placing, and compacting backfill material in scalped area.

Material placed on embankment capping shall be placed and compacted in conformity with the applicable requirements of Section 21.10 and 21.20 of the Standard Specifications except that the specified density requirements shall be waived. The required density shall be that obtained from a reasonable compaction effort.

Plan quantities will be used as measurement of embankment capping, except when errors are found in the original computations or ground elevations or when there has been an authorized change in grade or typical section. If the above exceptions are encountered, the plan quantity for the areas or sections affected will be adjusted for the change.

Payment for the embankment capping, complete in place, will be made at the contract price per cubic yard, which price shall be full compensation for the excavating and hauling, placing, forming, compacting, and any incidental work necessary to complete the item.

Payment will be made under:

Item 21120: Embankment Capping, per cubic yard.

K. BORROW PIT

Land for a borrow pit will be furnished by the Commission. This land is located on the left at Station 155+00 as shown on Sheet No. 22 of the detail plans.

The Commission has entered into an agreement with the property owner whereby the necessary borrow-excavation material required to form 285,000 cubic yards of embankment in place may be obtained from the borrow area free of cost to the contractor. Any borrow excavation removed in addition to that required to form 285,000 cubic yards of embankment in place will be paid for by the contractor at the price of seven cents (7¢) per cubic yard, measured in place in embankment will be estimated monthly by the engineer and payment at the above agreed price of 7¢ per cubic yard shall be made by the contractor to the property owner on or before the 15th day of each month following the month in which borrow material is removed in excess of the amount required to form 285,000 cubic yards of embankment. At the conclusion of all operations for the removal of borrow from said borrow pit, final measurement of the borrow material removed in excess of the amount required to form 285,000 cubic yards of embankment will be made by the engineer and final payment shall be made by the contractor to the property owner, based upon said final measurement.

SECTION 217
HYDRAULIC EMBANKMENT

217.1 Description. This work shall consist of constructing embankment of materials transported in water. This work shall be performed in accordance with the specifications and in conformance with the lines, grades, thicknesses, and typical cross sections shown on the plans, or established by the engineer.

217.2 Materials. The embankment shall consist substantially of granular material. When tested by washing over a No. 200 sieve, not more than 10 percent of the material shall pass.

217.3 Construction requirements.

(217.3.1) The contractor may, subject to approval by the engineer, make his own arrangements for easements in addition to those shown on the plans for the handling of transporting lines and return water. He shall be responsible for the protection of adjacent property from the flow of water or the deposition of silt resulting from the movement of materials by hydraulic methods.

(217.3.3) Capping of the slopes and stabilization of the top portion of the embankment shall be performed as shown on the plans or specified in the contract.

(217.3.4) Any material disturbed after hydraulic deposition in the embankment shall be recompacted to not less than 95 percent of maximum density.

217.4 Stockpile. Material may, subject to approval by the engineer, be placed in stockpile by hydraulic methods for later placement in the embankment by other methods. When this procedure is used, the material shall be placed in the embankment and compacted in accordance with the applicable requirements of Sec. 203 except that the compaction to be obtained shall be not less than 95 percent of maximum density.

217.5 Method of measurement.

(217.5.1) Contract quantity payment: The quantity of hydraulic embankment for which payment will be made will be that shown in the contract, provided the hydraulic embankment is constructed essentially to the lines and grades shown on the plans. A partial check of existing ground elevations will be made at the time slope stakes are set, and of the finished work for deviations in the grade, width, or slope from the authorized grade or typical section. The contractor shall make his own estimate of anticipated subsidence under the embankment and no adjustment will be made because of deviations from his estimate. Contract quantities will be used for final payment of Hydraulic Embankment except when:

(a) errors are found in the original computations;
(b) an original cross section is found to have an average deviation from the true elevation in excess of one foot; or
(c) an authorized change in grade, slope, or typical section is made.

When the above conditions are encountered, the corrections or revisions will be computed and added to or deducted from the contract quantity. No measurement will be made of material placed outside the approved section.

(217.5.2) When the plans have been altered or when disagreement exists between the contractor and the engineer as to the accuracy of the plan quantities of any balance, or the entire project, either party shall have the right to request a recomputation of contract quantities or hydraulic embankment within any area by written notice to the other party. The written notice shall contain evidence that an error exists in the original ground line or in the original computations which will materially affect the final payment quantity. When such final measurement is required, it will be made from the latest available ground surface and the design section.

217.6 Basis of payment. Payment for hydraulic embankment complete in place will be made at the contract unit bid price per cubic yard which price shall be full compensation for the dredging and transporting of materials; placing and forming of embankments; disposal of surplus water and silt; finishing of embankment; recompaction of disturbed material; placement and compaction of embankment from intermediate stockpile; and any work noted on the plans to be included in the price bid for hydraulic embankment. No direct payment will be made for compaction in areas designated for hydraulic embankment. No payment will be made for any material used for purposes other than those designated except as approved by the engineer.

PERMEABILITY AND FLOW VALUES OF VARIOUS MATERIALS

By M.S.H.D. Lab.

Material	Permeability - Cm. per Sec.	Flow - Gal./Day f 1 Sq.Ft. Cross-Section Area & 3% Grade
Rolled Stone Base, St. Chas. Co.	(1) .00017 to .0026	.1084 to 1.658
Lead Mine Chat, St. Fran. Co. (14% - #200 Sieve)	.000594	.378
90% L.M. Chat, 10% Knox (Total 2% - #200 Sieve)	.000129*	.0822*
Knox Soil, Holt Co. (Natural State)	.00000133(2)	.00085(.11 oz
Knox Soil, Holt Co. (Compacted)	Too impermeable to measure.	
Mo.Rv. Sand, 2.5% - #200 Sieve	.017	10.84
Mo.Rv. Sand, 10% - #200 Sieve	.00062	.396 (50.5 oz.)
Mo.Rv. Sand, 15% - #200 Sieve	.00015	.0956(12.2 oz.)

*After specimen stood 24 hr.

By Other Agencies

Concrete Sand (Cal.)	.00282 to .0141	1.8 to 9.0
(3) Sand & Gravel (Cal.)	.00211 to .02116	1.35 to 13.5
Ottawa Sand (Lambe)	.029	18.49
Mo.Rv. Deposits (Terzaghi)	.02 to .20	12.8 to 127.6
Loess (Terzaghi)	.001±	.64±
Loess Loam (Terzaghi)	.0001±	.064±
Dune Sand (Terzaghi)	.1 to .3 (283 to 849'/day)	63.8 to 191.0
Till (Terzaghi)	Less than .0001	Less than .064
Clay (Terzaghi)	Less than .000001	Less than .000064

Degree of Permeability (Terzaghi)	Permeability, cm/sec.
High	over .10
Medium	.10 to .001
Low	.001 to .00001
Very Low	.00001 to .0000001
Practically impermeable	Less than .00000001

(1) Range in values of tests made in conjunction with drainage study on route 40 from 1952 through 1956.

(2) Tests on two other undisturbed samples of Knox showed much less permeability than this.

(3) Probably somewhat similar to Missouri's sand and gravel, open graded base.

AMPHIBIOUS DRILLING RIG

Duane A. Eversoll*

ABSTRACT

In November, 1967, an amphibious drilling rig was purchased by the Nebraska Department of Roads. The rig was the first of its type to be purchased for use by a state highway department. It is used to take soil samples and perform various physical soil tests in rivers, lakes, and swampy areas where it has been impractical or impossible to operate truck-mounted drills. The rig is also adaptable for soil sampling while completely afloat.

A 4-year study and research of all available amphibious types of equipment was undertaken by the Roads Department before deciding on the combination that was finally selected.

This equipment has proved highly successful in ability to obtain soil samples with a minimum use of manpower under tedious and difficult conditions. Numerous problems had been previously encountered in attempting to take soil samples in some of Nebraska's shallow-water impoundments containing basins of muck. Although this amphibious rig does not overcome all obstacles and problems, it is a definite improvement over the previously used methods and equipment.

Considerable interest has been shown by neighboring states as to the utility and practical usefulness of such a vehicle for use in their own programs. The Department of Roads has leased the rig to other state agenices; the results have been a minimum expense, in monetary value, and a monumental savings of both labor and precious time.

INTRODUCTION

For a number of years those of us involved in foundation exploration work in the Nebraska Department of Roads have realized that we needed an acceptable amphibious drilling rig. The drilling

*Bridge Geologist, Nebraska Department of Roads.

would be performed in and across rivers, lakes, and swamps whose beds may consist of quicksand, mud, muck, and other less desirable soils where it has been impractical or impossible to operate the truck-mounted drills. Soil sampling while completely afloat would constitute approximately 5 percent of the total use of the rig. With the help of other interested parties we studied the different makes and models of auger drills and all amphibious and semi-amphibious carriers. The limiting factor in selecting the drill rig model was ease of installation and weight balance. We hoped that the medium range of auger drills such as the CME 55, B-40H and the FA-100 had the necessary power and options needed to meet our drill requirements.

CHOICE OF EQUIPMENT

The review of the carrier units included both half- or full-tracked vehicles, war-surplus weasels, self-contained, trailer-mounted drills, a dual-cylinder screw-type amphibious carrier experimentally developed for the Navy by Chrysler, 4 X 4 wheel tractors, and the amphibious swamp-type buggies.

The carrier unit presented our first big problem. We had to decide whether or not we wanted to use a unit that was made primarily for semi-amphibious conditions and then design the necessary floatation tanks from scratch or to go to the amphibious unit best suited for floating but that would also navigate in quicksand, muck and swampy conditions. We narrowed the field to the tracked vehicles and the amphibious buggies after a considerable length of time spent with our equipment engineers. It was at this point that we decided to investigate three areas. First, to contact other states that we believed might have a use for or might possibly be using this type of rig at that time. Secondly, to contact states and private firms that were using the tracked vehicles, and thirdly, to contact states and private firms that were using the amphibious buggies. We had a fair response from all three areas, and from this additional information we made our decision to use the amphibious buggy. The investigations disclosed that the amphibious buggies were used to transport rotary-drills and core-drills in much the same manner as we had anticipated and in terrain similar to what we would be encountering. The conclusions favoring the amphibious buggy were twofold: 1) The Layne-Western Company had mounted a special CME-55 on a Gemco buggy and had found it highly successful in swamp and muddy conditions. The Layne-Western buggy was equipped with the 18.4 X 26 implement tires, and the drill was powered by a self-contained engine. 2) We had seen a movie demonstrating the Gemco buggy being used in swampy areas of the southern states.

Essentially the rig is made up of two components. A 4-wheel-drive amphibious carrier vehicle and a hydraulic-driven exploration drill. The 4-wheel-drive amphibious carrier vehicle is manufactured in Houston, Texas, by the Gemco Company. The vehicles are made on an assembly-line basis and are widely used in the southern swamps and overseas.

The carrier vehicles' standard power plant is the Ford 172 cu. in. gas industrial engine. (Options include the Ford 172 cu. in. diesel-industrial engine; the Wisconsin V461D and 154 cu. in. gas air-cooled engine, or the engine that was specified, the Ford 240 cu. in. gas engine that develops 124 h.p. at 2800 rpm.) The drive train consists of an automotive-type 11-inch clutch, a 4-speed forward--1-speed reverse transmission (435 New Process), a 1.5-to-1 transfer case with a front-axle disconnect unit, and a split-shaft power take-off to operate the drill hydraulic pumps utilizing the 4-speed transmission. The front and rear axles are equipped with planetary gears (optional) and the rear axles have the "no spin" differential. The axles are Rockwell-Standards with 9000 lb. capacity front axle and 15,000 lb. capacity rear axle. The tires are Goodyear "Super Terra Grip" size 66" X 43.5" X 25" and are 6 ply. The tire pressures vary, as we normally carry between 6 and 9 lbs. The implement tires are available if needed for work not requiring maximum floatation. The braking system is the mechanical band-type and is located between the transmission and the transfer case on the drive shaft.

The carrier is equipped with an orbitrol power-steering unit. This is a hydraulically operated system and is standard equipment.

Figure 1, SWAMP BUGGY OF PLATTE RIVER

The fuel tank has a 20-gallon capacity and is located under the rider's side of the carrier. Special equipment consists of an all-weather cab with a heater, defroster, windshield wipers, rider's seat, safety belts, headlights, taillights, clearance lights, directional lights, 4-way flasher unit, and an amber emergency warning lamp located on top of the cab. Other optionals include a front bumper and grill-brush guard, tow hooks, manually operated rear-mounted stabilizing jacks and four aluminum floatation tanks which are easily removed. The rear floatation tank is sheet steel and serves as an operator's platform while drilling in water. The all-steel body is complete with tool boxes, auger-storage racks, drill-rod storage racks, and gas can - hydraulic can storage. Standard vehicle system indicators such as oil-pressure gauge, water-temperature gauge, fuel gauge, ammeter and hour-meter are located inside the cab within eyesight of the driver. A 12,000-lb.-capacity winch is mounted on the front bumper complete with 150 feet of half-inch wire rope and a cable-roller assembly. The cable roller is an essential piece of equipment as it protects the cable from being damaged when pulling at an angle.

The completed rig is 22 feet long, weighs approximately 7 tons, is 11 feet 5 inches wide and stands 12 feet 9 inches tall with the tower in the folded-down position. The top speed on land is about 20 mph and 1 to 3 mph on the water. In deep water the churning tire ribs propel the rig. The pay load is designed at 1000 lbs.; however, we have safely exceeded this by 500 lbs. If more buoyancy is needed, additional floatation tanks can be installed under the main chassis.

The hydraulic exploration drill is powered by the carrier engine through the split shaft power take-off. The P.T.O. is used to avoid the additional weight and space required by a self-powered unit. Our drill rig is a Failing FA-100 which is manufactured by the Westinghouse Air Brake Company. Other suitable exploration drill rigs such as Mobile Drill's B-40H or B-50, or Central Mine's CME-55 or 65, and others could be mounted on the amphibious buggy. WABCO has successfully mounted their lightweight rotary drills on the Gemco Buggy.

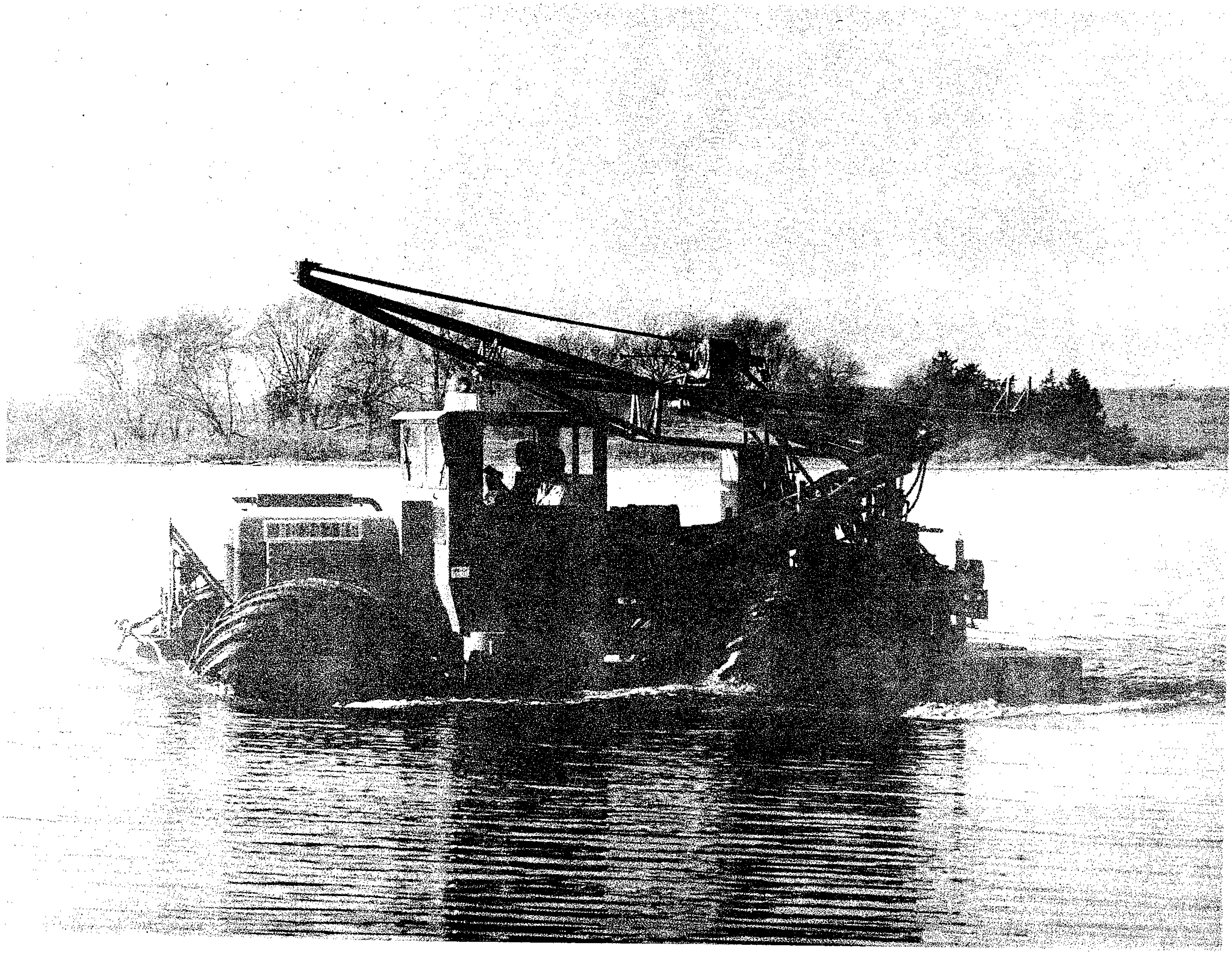

Figure 2. MOVING TO NEW DRILLING SITE

Specifications for the drill unit did not vary too much from the regular truck-mounting standards. We did specify that the drill tower would have 14 feet of clearance between the axis of the crown sheave and top of spindle when in lowest position. This clearance would allow us to pull 15 feet of A-rods at one time while taking standard penetration tests and thin-walled (Shelby tube) tests. The 200 lb. drill tower is easily detachable to provide clearance when the buggy is transported by truck to the drilling site.

We also specified an 18-inch sliding base to allow the rig to be moved from drilling position to another position nearer the cab for better weight balance and vehicle control while travelling. Other specified items included a hydraulic-motor-powered cathead and a control panel which can be easily reached and read by the driller.

FABRICATION

The standard procedure for the Nebraska Department of Roads when obtaining new drill rigs has been to take separate bids for the drill and carrier units. The drilling rig is then installed on the carrier by shop personnel at the maintenance shops. The procedure for obtaining this amphibious drilling rig was completely reversed. Our specifications called for a portable, hydraulic-driven exploration drill mounted on an amphibious carrier with all accessories, which was to be delivered to the Highway Department assembled, complete, and ready for satisfactory bridge and highway exploration operation. The completed unit was to be so constructed as to safely float the carrier, drilling unit, and 1000 lbs. of tools and accessories when equipped with the 66 x 43.5 x 25 Terra tires.

We further recommended that the unit as a whole would be tested and inspected at the factory by two representatives of the Nebraska Department of Roads. This inspection trip was very successful from the standpoint of both the manufacturer and the Department and is highly recommended. Several items were changed at the manufacturer's plant to our satisfaction. It also gave us a preview of what to expect from the rig and the manufacturer, and

Figure 3. METHOD OF TRANSPORTING SWAMP BUGGY

made for smoother relationships between the Department and the manufacturer.

OPERATION

The swamp buggy is presently transported to the site on a low-boy whenever possible. The Nebraska Department of Roads has divided the State into six field divisions. Each division is equipped with one or more transports capable of hauling the swamp buggy. It is fairly easy to make arrangements for transportation with the division where the site is located. When loaded onto the low-boys the tires hang over the truck bed with 22 inches on each side. Therefore, it was necessary to build special axle jacks which were designed to carry most of the weight of the buggy in order to prevent any damage to the tires during transportation.

CONCLUSIONS

In the 2 1/2 years that we have had the buggy, we are very well pleased with its performance. We have made minor adjustments and small additions, such as insulating the interior of the cab with a 1-inch polyfoam insulation layer held in place by perforated tempered masonite.

The "Swamp Buggy" has saved the Department of Roads considerable time and money and at the same time has allowed us to provide much more foundation information than was previously possible. Although it does not overcome all obstacles and problems, it is a definite improvement over the previously used methods and equipment.

ACKNOWLEDGEMENTS

I am greatly indebted to members of the Division of Materials and Tests, Nebraska Department of Roads, especially Otto B. Griess, Senior Geologist; and Lyle Nelson, Embankment Foundation Engineer, for their assistance in the preparation of this report. Thanks are also due to the members of the Bridge Sounding Crew and to the Bridge Department of the Nebraska Department of Roads.

UNDERGROUND TRANSPORTATION ROUTES AND QUARRY PRACTICES IN THE KANSAS CITY, MISSOURI-KANSAS AREA

John W. Whitfield* and James H. Williams**

ABSTRACT

The Bethany Falls Limestone (fig. 1) is quarried for aggregate by open-pit and underground methods in the Kansas City area. Over the years, excavation experience in these quarries has provided the methods for obtaining stable undergound transportation tunnels in the Bethany Falls Limestone.

Several arbitrary tunnel routes were selected to show geologic features that would influence tunneling operations. Procedures used in limestone mining that might relate to tunneling are noted in relationship to the illustrative tunnel routes.

INTRODUCTION

Early interest in underground transportation was shown when the Kansas City streetcar system constructed a tunnel under 8th Street, on top of the hill, west to the industrial district in the Kansas River floodplain. In 1950, a proposed two-tunnel system was considered to carry traffic from 13th to 6th Street in downtown Kansas City, Missouri. Exploratory borings were made but no further work was done. Renewed interest was shown by the *Kansas City Star* (1968), in a story discussing an underground freeway concept beneath 27th Street.

In order to relate general geologic features of mines to tunnels, we chose arbitrary tunnel routes for illustrative purposes. In selecting these alignments, trends of urban growth, transportation demands, or costs were not considered. The illustrative tunnel routes extend from the southern and eastern fringes of Kansas City to Kansas City International Airport (fig. 2).

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*Geologist, Engineering Geology, Missouri Geological Survey (Speaker).
**Geologist and Chief, Engineering Geology, Missouri Geological Survey.

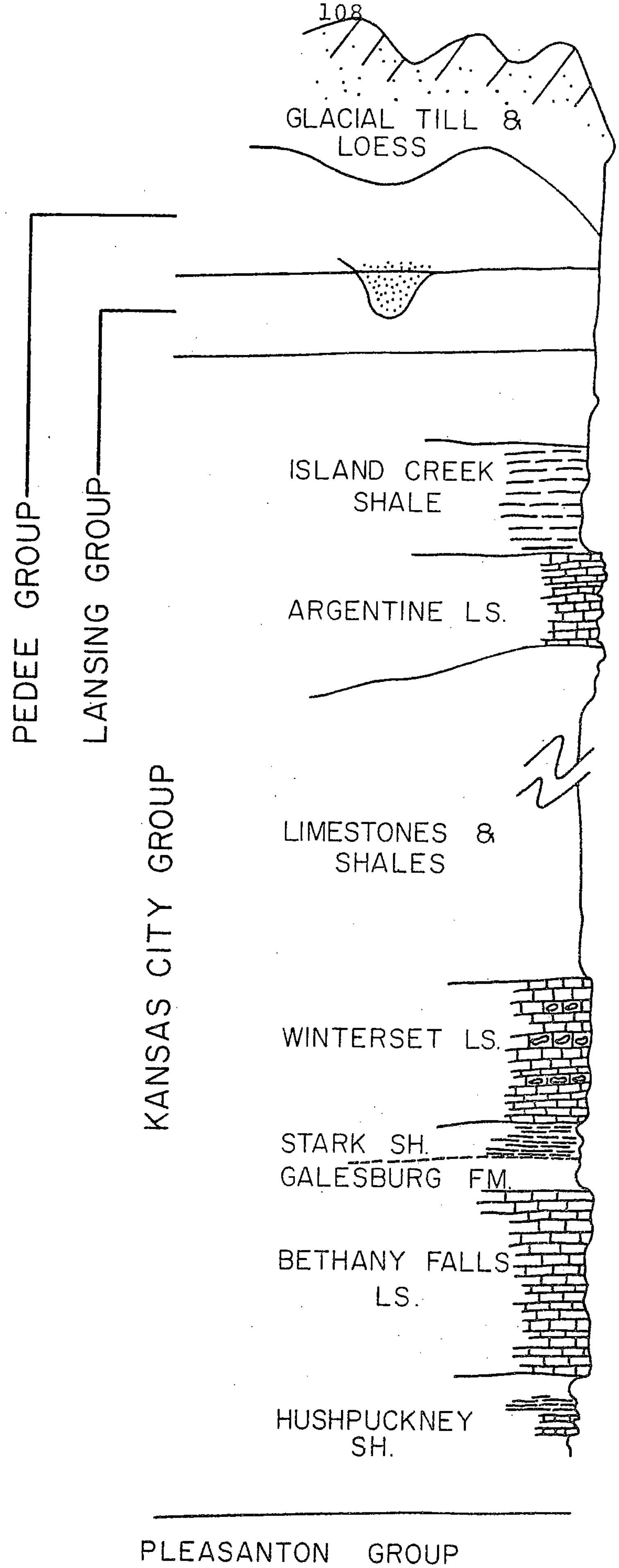
GLACIAL TILL & LOESS
PEDEE GROUP
LANSING GROUP
KANSAS CITY GROUP
ISLAND CREEK SHALE
ARGENTINE LS.
LIMESTONES & SHALES
WINTERSET LS.
STARK SH.
GALESBURG FM.
BETHANY FALLS LS.
HUSHPUCKNEY SH.
PLEASANTON GROUP

Figure 1.

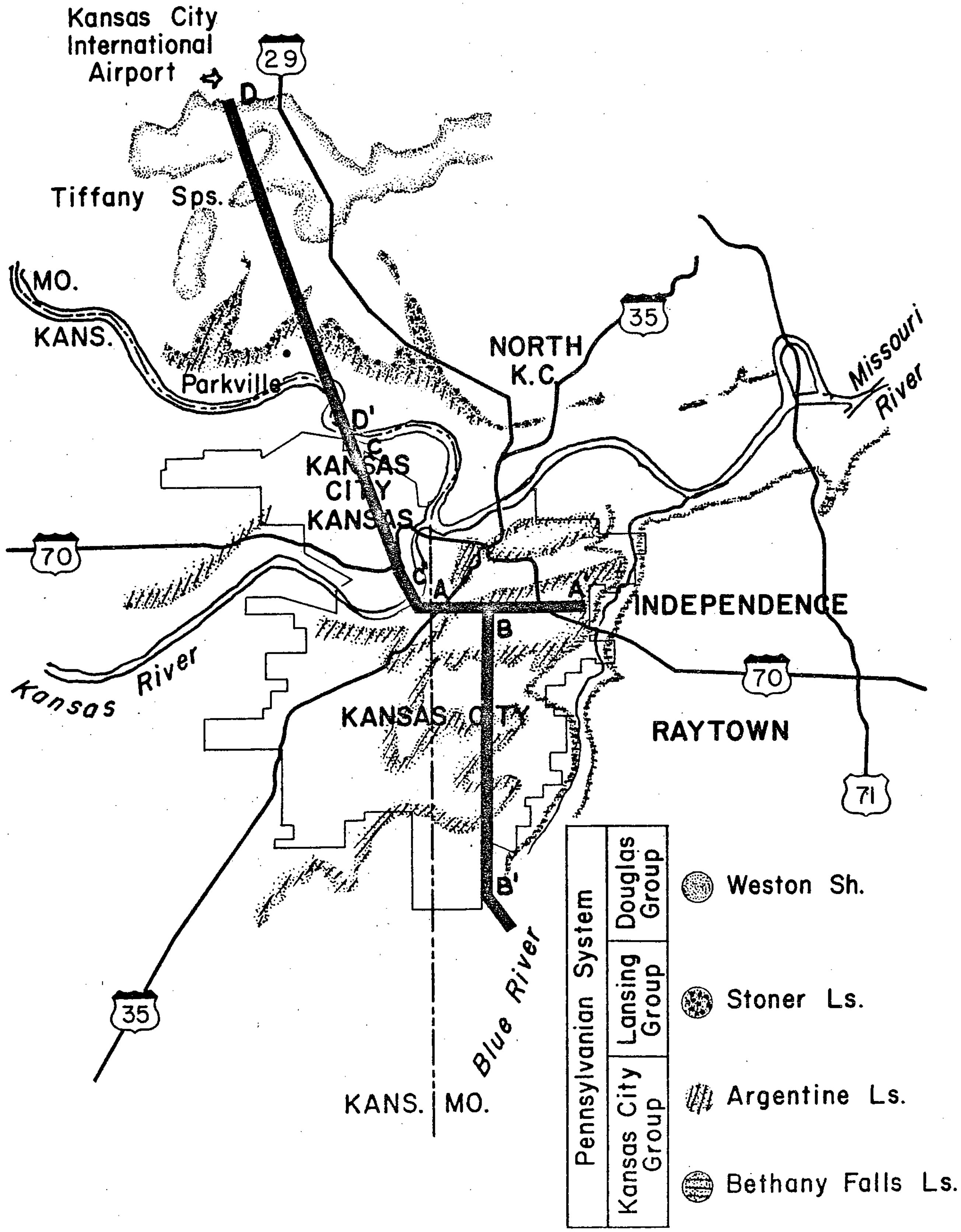
Kansas City International Airport
29
D
Tiffany Sps.
MO.
KANS.
Parkville
D'
C
KANSAS CITY KANSAS
NORTH K.C
35
Missouri River
70
C'
A
A'
INDEPENDENCE
B
Kansas River
KANSAS CITY
RAYTOWN
70
71
B'
35
Blue River
KANS.
MO.
Pennsylvanian System
Douglas Group
Lansing Group
Kansas City Group
Weston Sh.
Stoner Ls.
Argentine Ls.
Bethany Falls Ls.

Figure 2.

Figure 1 shows the geologic section that would influence tunneling.

Tunnels A-A' and B-B' (fig. 3), are in Kansas City, Missouri; tunnel C-C' is in Kansas City, Kansas (fig. 4); and tunnel D-D' is in southern Platte County, Missouri (fig. 4). Because most tunneling would be in the Bethany Falls Limestone, experience gained from underground quarrying practices in this limestone would be useful.

TUNNEL ALIGNMENTS

Tunnel A-A' has its east portal in the Bethany Falls Limestone outcrop in the Blue River Valley. The tunnel would follow under 27th Street and have its west portal in Turkey Creek Valley. The entire tunnel would be through relatively uniform Bethany Falls Limestone. Tunnel B-B' follows U.S. Highway 71. It would have its south portal in the Blue River Valley and would extend northward until it intersects tunnel A-A'. There would be no north surface portal for tunnel B-B'. This tunnel would also be excavated out of the Bethany Falls Limestone. Tunnel D-D', in southern Platte County, Missouri, would extend from its south portal in the Missouri River Valley to the north portal at Kansas City International Airport, a distance of approximately 7 miles. Because tunnels A-A', B-B', and C-C' are in the Bethany Falls, some of the geologic conditions should be discussed.

GEOLOGIC CONDITIONS - KANSAS CITY TUNNELS

Some of the geologic conditions that affect an underground mine or tunnel in the Bethany Falls Formation are:

1) Overburden - type and thickness of soil and bedrock. Neglect of these factors in planning has produced most of the problems in quarry operations.

2) Ground water - how much ground water can be expected, and what effect it will have.

3) Stability of the rock and how it affects columns, roof, and floor.

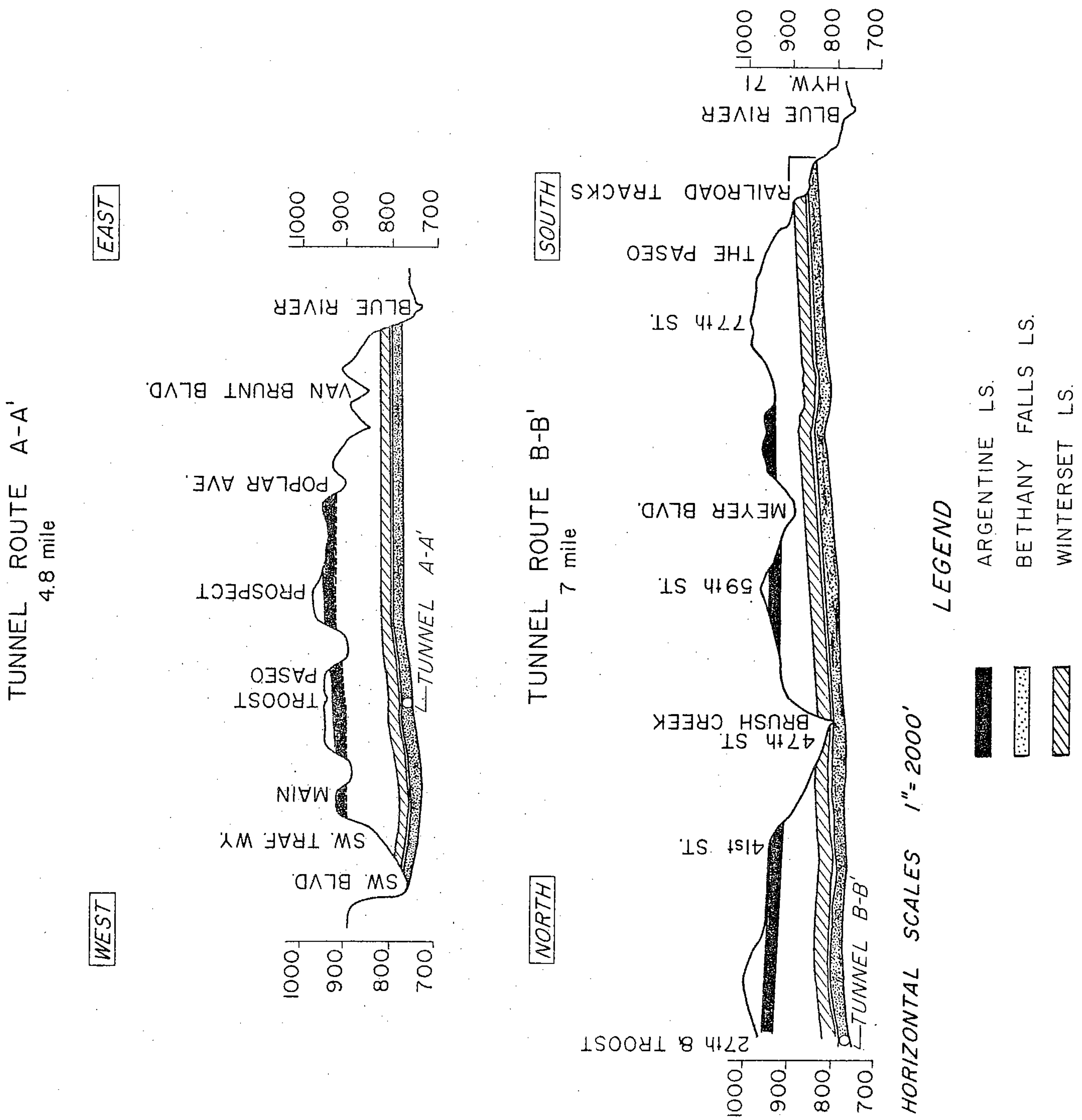
TUNNEL ROUTE A-A'
4.8 mile
WEST
EAST
1000
900
800
700
SW. BLVD.
SW. TRAF. WY.
MAIN
TROOST
PASEO
PROSPECT
POPLAR AVE.
VAN BRUNT BLVD.
BLUE RIVER
TUNNEL A-A'
TUNNEL ROUTE B-B'
7 mile
NORTH
SOUTH
27th & TROOST
41st ST.
47th ST.
BRUSH CREEK
59th ST.
MEYER BLVD.
77th ST.
THE PASEO
RAILROAD TRACKS
BLUE RIVER
HYW. 71
TUNNEL B-B'
HORIZONTAL SCALES 1"= 2000'
LEGEND
ARGENTINE LS.
BETHANY FALLS LS.
WINTERSET LS.

Figure 3.

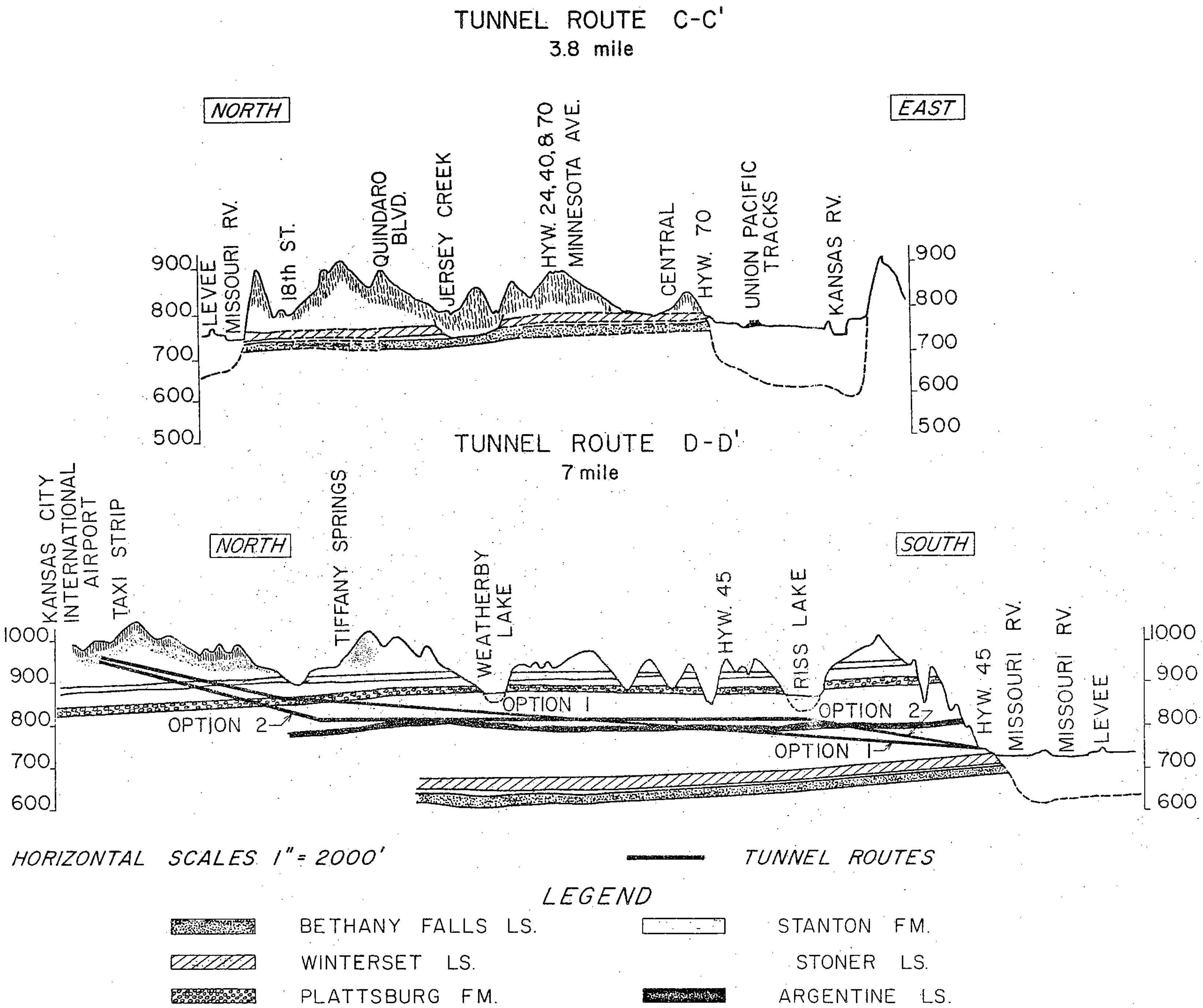
TUNNEL ROUTE C-C'
3.8 mile
NORTH
EAST
LEVEE
MISSOURI RV.
18th ST.
QUINDARO BLVD.
JERSEY CREEK
HYW. 24, 40, & 70
MINNESOTA AVE.
CENTRAL
HYW. 70
UNION PACIFIC TRACKS
KANSAS RV.
900
800
700
600
500
TUNNEL ROUTE D-D'
7 mile
KANSAS CITY INTERNATIONAL AIRPORT
TAXI STRIP
NORTH
TIFFANY SPRINGS
WEATHERBY LAKE
HYW. 45
RISS LAKE
SOUTH
OPTION 1
OPTION 2
HYW. 45
MISSOURI RV.
MISSOURI RV.
LEVEE
1000
900
800
700
600
HORIZONTAL SCALES 1" = 2000'
TUNNEL ROUTES
LEGEND
BETHANY FALLS LS.
WINTERSET LS.
PLATTSBURG FM.
STANTON FM.
STONER LS.
ARGENTINE LS.

Figure 4.

Overburden

The soil or bedrock cover governs the effects of weathering on the underlying rock that is to be excavated. As an example, the Bethany Falls will consist of hard, sound, unweathered limestone where it lies beneath a protective bedrock cover of Galesburg and Stark Shales and most of the Winterset Limestone (fig. 5). In contrast, where the overlying Winterset Limestone is severely weathered or is not present, roof collapse and surface cave-in of the Bethany Falls Limestone is a possibility.

Mine and tunnel entrances should be located where there is an adequate thickness of soil and bedrock overburden to assure stability in the rock ledge to be excavated. Thick bedrock overburden may also reduce water seepage resulting from percolation of surface water.

When tunnels pass beneath valleys that have eroded through the protective overburden, tunneling may be affected by clay-filled crevices and the possibility of roof-fall exists (fig. 6). Because some valley development is related to the Pleistocene, these valleys may be filled with permeable debris which could affect tunneling. Therefore, close attention should be given to exploration data as well as to physical changes in the limestone being excavated. Any softness, or seam or joint development may imply weathered and weak overburden. For example, Brush Creek Valley has been cut downward through the Winterset Limestone and possibly into the Bethany Falls Limestone near the middle part of tunnel B-B'; deterioration of the limestone and some ground water flow should be expected. Present subsurface data does not indicate buried Pleistocene valleys crossing tunnels A-A' and B-B', but further subsurface data should be collected.

Ground Water

Mining operations in the Bethany Falls Limestone have encountered ground-water seepage in the underlying black, fissile, Hushpuckney Shale. Also, the overlying Stark, a fissile shale, is a source of ground-water seepage. Water flow yields from wells drilled in these shales amounts to a few gallons per minute.

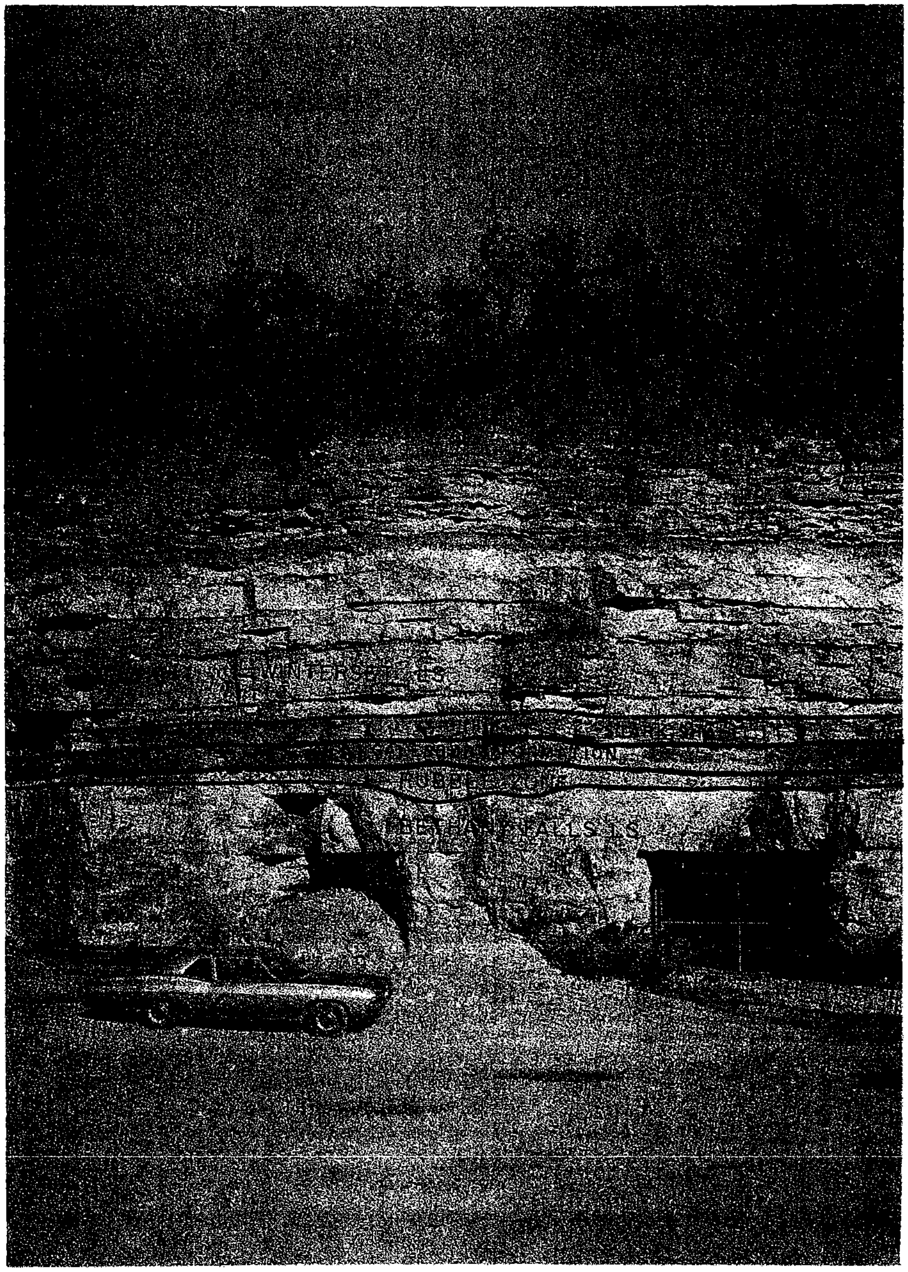

Figure 5. Bethany Falls roof protected by a thick overburden of shale and limestone. At least 5 feet of competent limestone is present in the roof beneath the rubble zone at the top of the Bethany Falls.

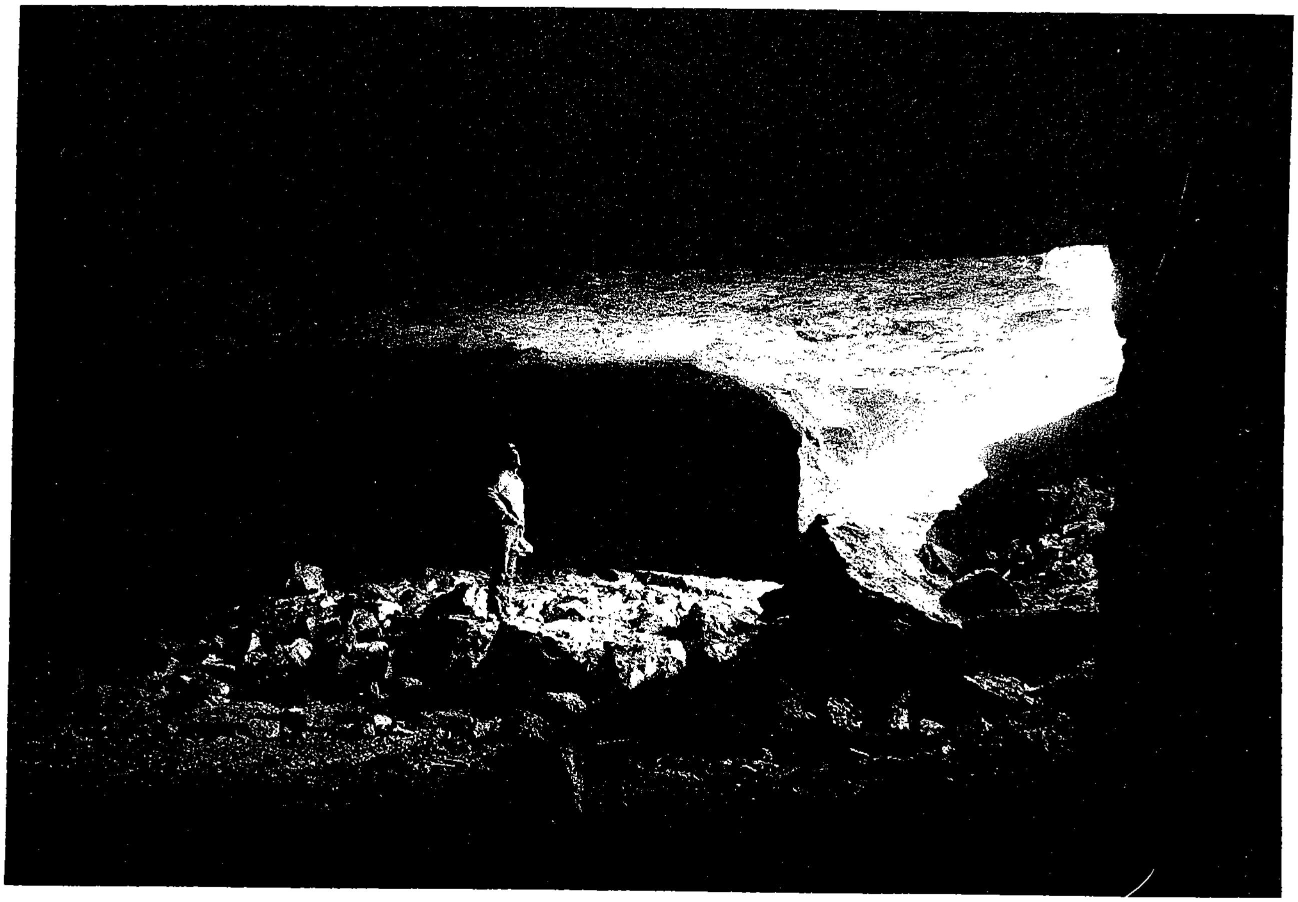

Figure 6. Collapse of mine roof in Bethany Falls where a stream valley had eroded a portion of the protective overburden.

Water in the lower Hushpuckney Shale will pond in the mine floor if the bedrock dip is such that it is difficult to provide drainage (fig. 7). As a result, pumps or other methods of water removal will be necessary. When water is left on the Hushpuckney Shale, some deterioration of the shale's surface can be expected.

Water in the overlying Stark Shale is in the same geologic setting as the water in the Hushpuckney. However, problems of water seepage from the Stark Shale are minor unless roof-robbing occurs in the tunnel or quarry. Ventilation shafts or roof bolts penetrating the shale can create flow paths for water. In these cases, grouting may be necessary to seal off seepage water.

Stability

This takes into consideration room sizes, pillars, and roof thicknesses in the Bethany Falls Limestone. Where unaffected by weathering, the Bethany Falls Limestone is a massively-bedded, uniformly hard limestone 15 to 20 feet thick.

So far, there has been only an empirical consideration of roof span width, pillar shape, pillar diameter, and roof thickness. It is general knowledge in mining operations that spans greater than 40 feet between pillars and roof thickness of less than 5 feet of sound bedrock eventually result in roof fall and possible surface subsidence. Pillar diameters vary from 22 to 25 feet in most mines.

Opinions have varied on what is considered to be a safe roof thickness in the Bethany Falls Limestone. Dean and others (1968), believe roof thicknesses should be 7 to 8 feet due to the rubbly zone present in the upper part of the Bethany Falls. The rubble zone varies in thickness and there is no way to predict where it will weaken the roof unless the area is perforated with exploration holes (fig. 8). The underground quarry operations that have left at least two thick beds of Bethany Falls (5 feet or more) <u>plus</u> the upper rubble zone, have had little trouble with roof collapse (fig. 8). Other quarry operators feel it is adequate to leave a total of only 5 to 6 feet of overlying limestone including the rubble zone in the roof.

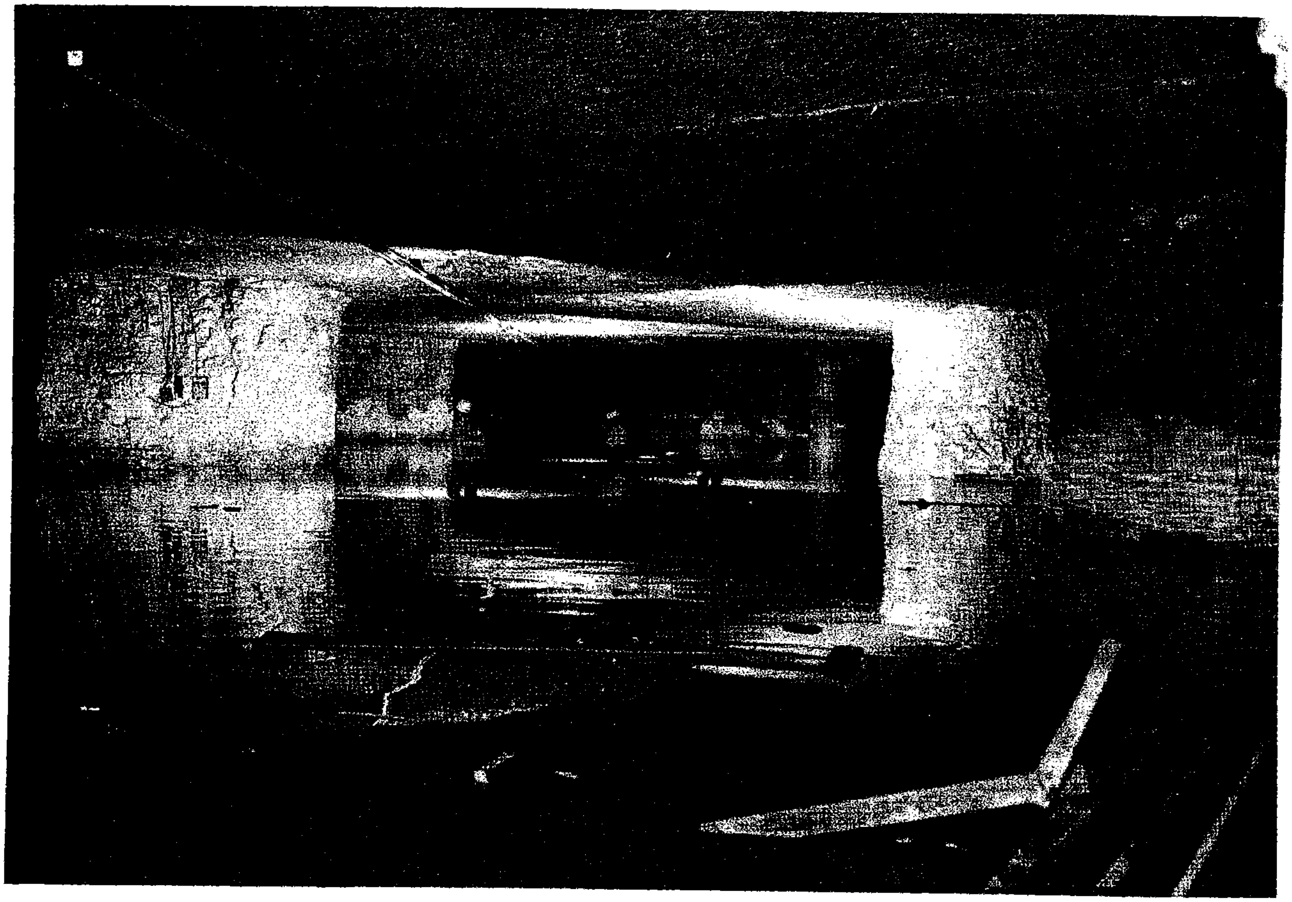

Figure 7. Ponding of water in an inadequately drained mine where bedrock dips toward the working face. (Photograph by Jerry D. Vineyar

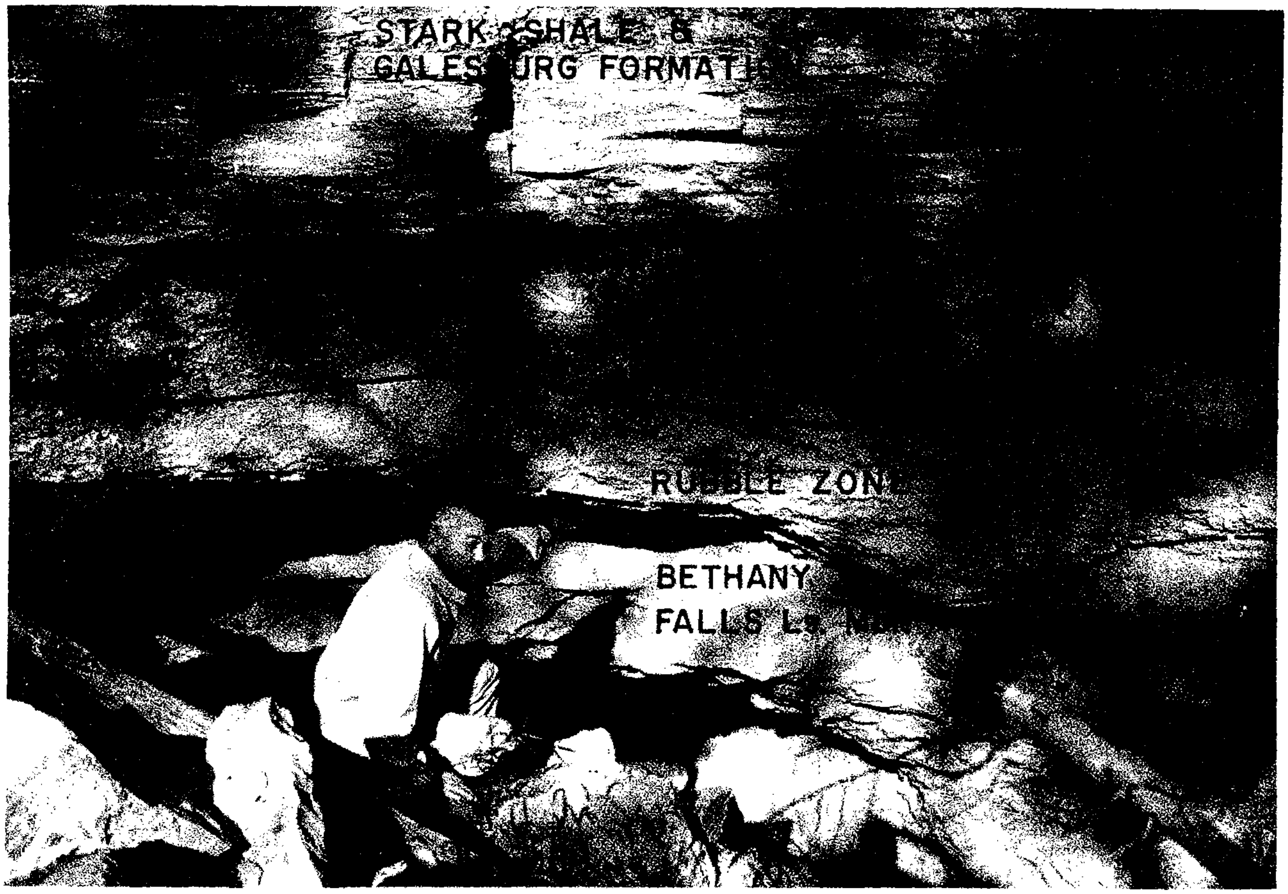

Figure 8. The rubble zone, upper Bethany Falls, can vary from 1 to almost 3 feet in thickness. A thick rubble zone reduces mine roof strength. (Photograph by Jerry D. Vineyard)

As in quarry operations, roof bolting may be necessary in unlined tunnels. Method of roof bolting will vary in the different formations that a tunnel will pass through. In the Kansas City area, bolting is generally done on 4- to 6-foot centers. This will vary where a tunnel roof may be a thin limestone. A complete lining of the tunnel may be necessary where it passes through the cyclic shale and limestone deposits of the Kansas City Group.

DISCUSSION OF TUNNEL ROUTES

Flood plains, Missouri and Kansas Rivers

No detailed consideration was given to tunnel boring in flood plains. However, if tunnels extend beneath the Missouri or Kansas Rivers, saturated gravels, sands, and silts can be expected. As an indication of water quantities, wells for municipal or industrial use produce 500 to several thousand gallons per minute (U.S.G.S. Report No. 273).

As to depth of bedrock in the flood plain, a test hole drilled by the U.S. Army Corps of Engineers in the Missouri River flood plain southeast of Parkville penetrated 162 feet of alluvium. Borings in the Kansas River flood plain have penetrated up to 183 feet of alluvium.

Geologic Setting - Kansas City International Airport, Tunnel D-D'

Due to the northwestward bedrock dip, the Bethany Falls can no longer be followed by the tunnel. If the tunnel follows a uniform angle beginning at the south portal near Highway 45, sloping upward at a uniform gradient until it reaches the surface at the airport, it would pass through various limestones such as the Westerville, the Cement City, the Raytown, and the thicker Argentine, as well as numerous shales. After the tunnel passes through the limestone and shale sequence, it would penetrate the thick Bonner Springs Shale, then the Lansing Group limestones and shales and the Pedee Group of shales and sandstones. It would finally exit through the glacial tills and loesses of southern Platte County.

An alternate possibility would be to steepen the gradient from the Missouri River portal at Highway 45 until the tunnel passes through the Argentine Limestone and enters the Island Creek Shale. The Argentine is thin- to medium-bedded limestone and has thin clay seams along bedding planes. Locally it is 20 to 40 feet thick south of the Missouri River, but thins north of the river. Because of its thinning, its use as a tunnel formation is questionable.

If the Argentine Limestone were used as a tunnel route, at least one-half of the tunnel face would be shale because of the thinning. Davis, Howe, and Heim's (1960) stratigraphic section 2 in the Riverside, Missouri, area shows 12 feet of Argentine Limestone present. Therefore, if the tunnel could be constructed in the Island Creek Shale above the Argentine Formation, a more uniform rock type could be followed for several miles and the Argentine Limestone could be used as the tunnel floor.

The Island Creek Shale ranges in thickness from about 17 feet in the Parkville area to 35 feet in southeastern Platte County. Further exploratory work should be done in the Island Creek Shale to determine its thickness at various points between the Missouri River and Kansas City International Airport.

Ground-water quantities may increase in the northern portion of tunnel D-D' due to the presence of the Tonganoxie Sandstone Member and permeable sands in Pleistocene glacial till. An examination of well-logs in this area shows that the Tonganoxie Sandstone is capable of producing 5 to 12 gallons per minute and the glacial till sands up to 60 gallons per minute. Surface water in the Brush Creek watershed may act as a recharge for groundwater in the sandstone. A tunnel passing through this area of recharge may encounter more water than other tunnel segments. Overall, limestones and shales produce very little water in this area. The tunnel would pass beneath several valleys and weathering below the valley bottoms may affect the strength of bedrock. An increase in ground-water quantities may also be expected below the valleys. As a result, lining of the tunnel may be necessary.

Another natural phenomenon that would hinder tunnel construction along route D-D' is limited amounts of natural gas in the bedrock. Past geologic reports show there has been gas production in the lower Kansas City Group and underlying Pleasanton Group.

SUMMARY

Mining practices in Kansas City have been unusually successful with only minor problems of stability in stone extraction. Only when techniques were such that common-sense practices were violated has roof fall and surface cave-in occurred (fig. 9). Tunneling could progress in almost any direction with favorable results provided that adequate attention is given to geologic conditions (fig. 10 and 11). Portions of tunnels that appear to present construction problem (such as where valley floors are close to the tunnel) could be used advantageously as ventilation-shaft sites.

Tunneling by conventional methods or with a boring machine (mole) should be routine from a geologic aspect. Where favorable conditions may exist for a mole (such as in the Bethany Falls Limestone) the tailings produced may not have an economic value comparable to the limestone aggregate extracted by conventional mining methods. The cutting action of the bits usually produces a limestone chat that is too small for concrete aggregate.

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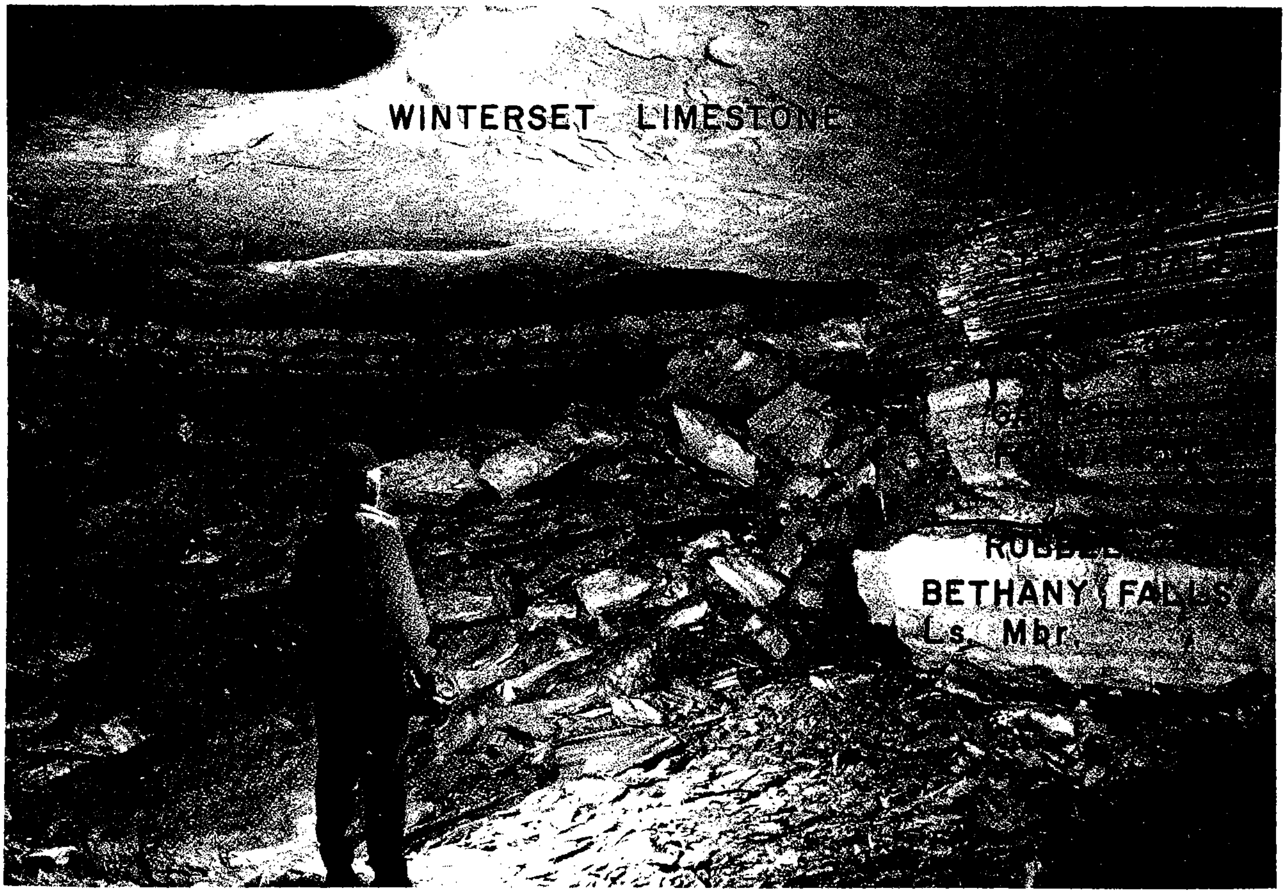

Figure 9. Roof collapse which will progress upward until surface subsidence occurs. This collapse was caused by overly wide spacing between pillars and robbing of limestone beds in the mine roof. (Photograph by Jerry D. Vineyard)

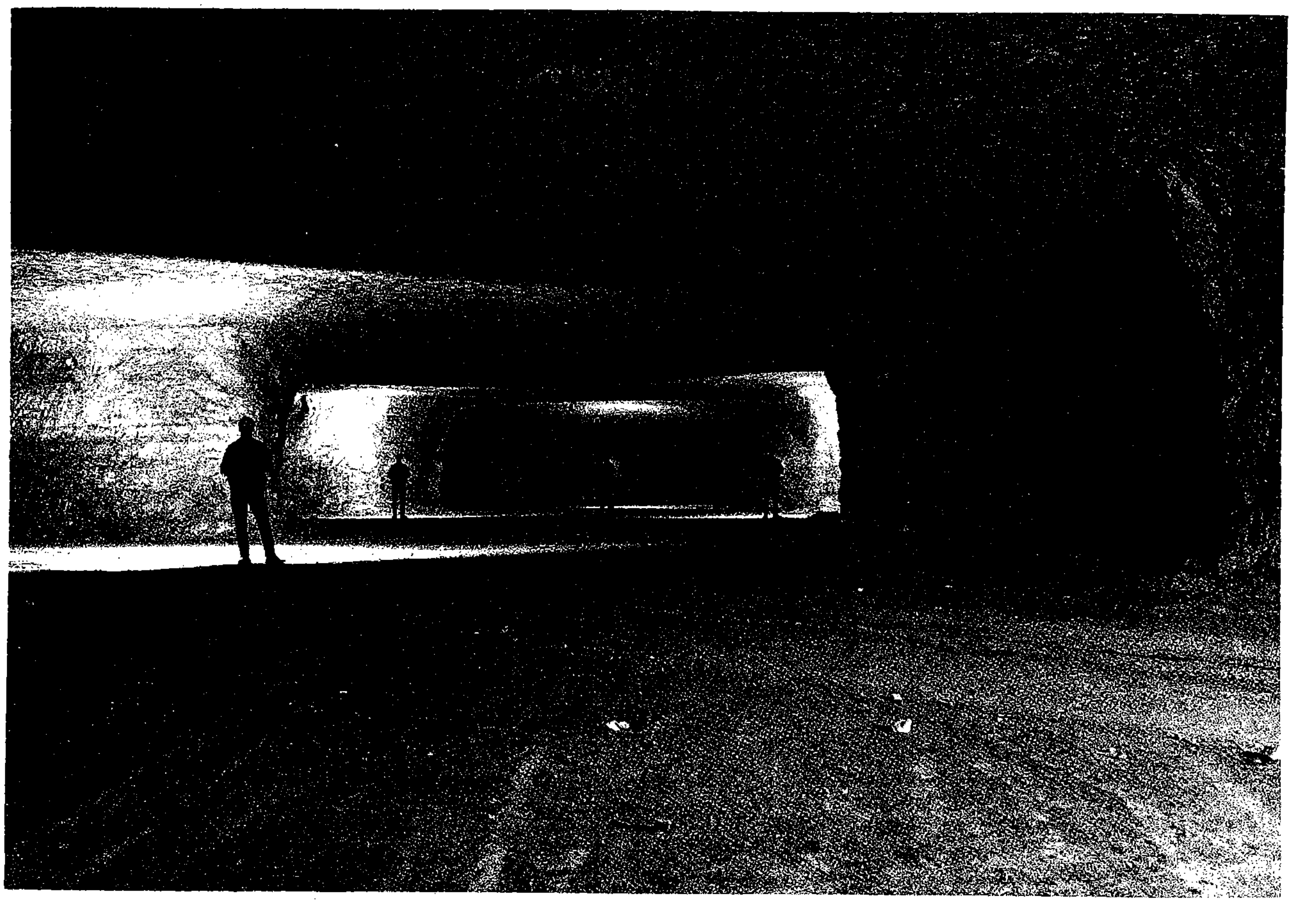

Figure 10. A planned mine with regularly spaced pillars. Here sequential use of the mine as office space, manufacturing or warehouse storage is possible.

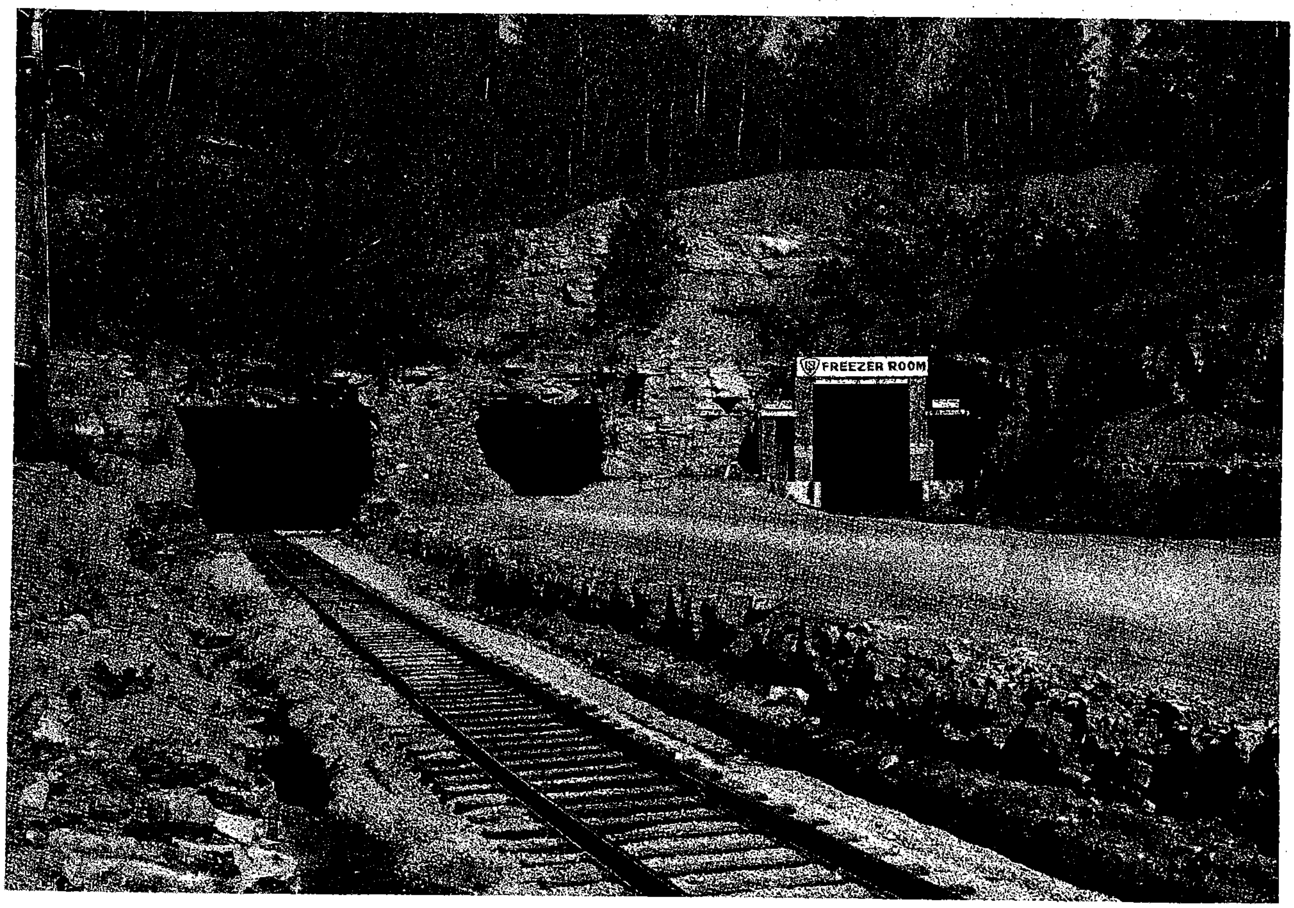

Figure 11. Trucks and trains enter and leave mines designed to accomodate such transportation facilities. The floor of the mine was deepened into the underlying shale to obtain clearance for railroad access.

APPLICATION OF REMOTE SENSING TO HIGHWAY LOCATIONS

Terry R. West*

INTRODUCTION

The title of this paper could easily be restated as, "The engineering mapping of soil and rock using remote sensing techniques, with special emphasis on highway location." Therefore, presented here is a brief review of work done in remote sensing on soil and rock mapping as it pertains to highway location. It is given with the hope that remote sensing, as a recently developed and rapidly growing specialty, will be of interest to geologists and engineers working in highway location and design.

Remote sensing is the identification and determination of characteristics of physical objects through the analysis of data obtained from a measuring device that does not come in contact with these objects. In most cases, these devices depend upon the measurement of electromagnetic radiation in that they record radiation intensities from selected portions of the electromagnetic spectrum. The measurement of gravity and magnetic force-fields may also be considered as remote sensing methods but they are included only in exhaustive lists of available techniques.

In remote sensing, reflected and emitted electromagnetic radiation from specific target areas is detected and recorded. The analysis phase which follows consists of the recognition and classification of the physical objects under study.

The following sections are reviewed and discussed: 1) the electromagnetic spectrum, 2) the use of remote sensing for mapping soil and rock material, and 3) an automatic soil- and rock-classification method based on multispectral, remote-sensing imagery.

The third subject consists of work being done at Purdue University by the Laboratory for the Application of Remote Sensing (LARS). Specializing originally in soil and vegetation studies

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for agricultural purposes, LARS has begun additional work in the related areas of hydrology, geology, geography, and land use. Mapping of soil and rock for engineering purposes comprises a portion of the geological study and thus supplies information for this paper. The author, who has recently become affiliated with LARS, will call on previous research findings for much of this discussion of the automatic classification system.

The author wishes to acknowledge the efforts of Dr. C. J. Johannsen who reviewed the manuscript and Mr. J. E. Halsema who assisted in preparing the figures.

ELECTROMAGNETIC SPECTRUM

The enormous expanse of electromagnetic radiation known as the electromagnetic spectrum is shown in Figure 1. The spectrum ranges from short-wavelength, high-frequency waves on the left to long-wavelength, low-frequency waves on the right. Frequency and wavelength are related by the equation $C = \gamma\lambda$ where C is the speed of light, γ the frequency and λ the wavelength. Since C is a constant approximately equal to 3.0×10^5 km/sec, γ and λ are directly proportional values.

On the extreme left side of the figure are seen in order, cosmic rays, gamma rays, and x-rays. To the right are found ultraviolet waves, the visible spectrum, infrared, radar, television, and finally, radio waves with their long wavelengths and low frequencies. Radar, television and radio waves are referred to as Hertzian Waves. In the enlarged section at the bottom of Figure 1, the visible spectrum and neighboring ultraviolet and infrared wavelengths are shown. The visible spectrum ranges from .40 microns or micrometers* (also 4,000 Å or 4×10^{-4} mm on the violet end to .72 microns on the red end of the scale. Reflective or near infrared wavelengths extend from .72 to 2.50 microns and thermal or far-infrared wavelengths range from approximately 2.5 to over 30 microns. Little information, however, is obtained above 14 microns.

*one micrometer = 10^{-6} meters

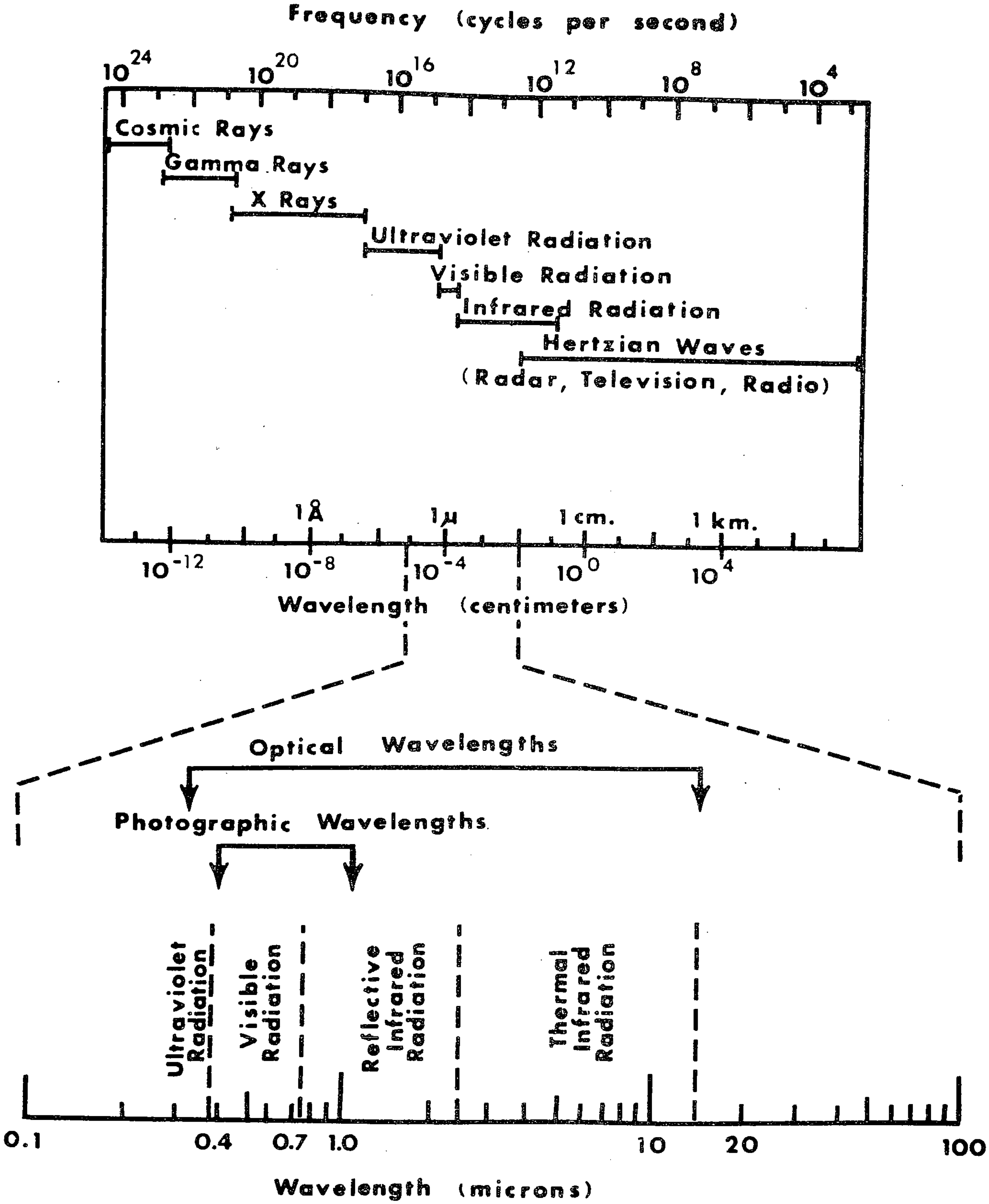

FIGURE 1 - The electromagnetic spectrum.

GEOLOGIC APPLICATIONS OF ELECTROMAGNETIC IMAGERY

Until quite recently much of the remote-sensing activities have fallen into the realm of aerial photography. Using various camera, film, and filter combinations much work has been done in the area of photo interpretation. A wealth of informtion is available on photo-interpretation but it is not the purpose of this paper to review it. Some of the more recent developments in aerial photography, however, are continuous-strip photography; color or black-and-white, large-scale photography (70 mm); low-altitude photography; color infrared photography; and multiband photography where two or more electromagnetic bands are photographed simultaneously. The subject of photography or any other specific technique will not be discussed in detail here, but instead, the portions of the electromagnetic spectrum which can be used to analyze geologic features will be considered individually.

In remote sensing there are two basic types of systems, active and passive. In an active system, energy is emitted by the instrument, interpreted by the object and a return pulse sent back to the detector. A passive system requires no energy emission but measures the energy from the object itself. Active systems are needed in that portion of the spectrum where insufficient or no natural radiation is available.

Individual remote sensors do not operate over a broad range of the electromagnetic spectrum. Each type of remote sensor reacts only to energy bands of specific frequency and wavelength. For example, radar receivers cannot detect visible light nor are microwaves detected by infrared scanners.

Figure 2 is a compilation of anticipated geologic applications of remote sensors and Figure 3 is a summary of the types of information and/or properties of materials that may be interpreted from observations of various parts of the electromagnetic spectrum. Together they show the considerable potential for measuring geologic materials. To date, not all the potential has been realized but considerable progress is being made. In the following paragraphs, portions of the electromagnetic spectrum will be discussed in turn giving some indication of the work accomplished in each region.

ANTICIPATED GEOLOGIC APPLICATIONS OF REMOTE SENSORS

COMPOSITION / STRUCTURE / STRATIGRAPHY SEDIMENTATION / MINERAL DEPOSITS / ENGINEERING / CRUSTAL / MANTLE STUDIES / OTHERS

	ROCK TYPES (GROSS)	LITHOLOGY	POROSITY	PERMEABILITY	FABRIC (GROSS)	FAULTS, FOLDS	SALT DOMES	CORAL REEFS	RIVER EFFLUENTS	NEAR SHORE & BEACH DEPOSITS	DUNES	STREAM, LAKE, BAY DEPOSITS FANS	FAULTS, FOLDS	ALTERATION, CONTACT ZONES	INTRUSIVE ROCKS	METALLOGENIC PROVINCES	OIL SHALE	SURFACE ROUGHNESS	SOIL MOISTURE DIST.	PERMAFROST	SLOPE STABILITY	COMPACTION	EARTHQUAKE ZONES	VOLCANOES, THERMAL WATERS	HEAT BALANCE, VARIATIONS	MAGNETIC FIELDS	GRAVITY GRADIENTS	INTERNAL CONSTITUTION	ISOSTASY	CONTINENTAL DRIFT	GLACIATION (CONTINENTAL, VALLEY)	
1. METRIC CAMERA	X	X		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	
2. PANORAMIC CAMERA	X	X		X	X	X	X	X	X	X	X	X	X			X	X	X		X			X	X	X					X	X	
3. ULTRA-HIGH RES. CAMERA	X	X	X	X		X	X		X	X			X	X	X	X	X	X			X	X	X	X	X						X	
4. MULTIBAND SYNOPTIC CAMERA	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X				X	X	X	X						X	
5. RADAR IMAGER	X		X	X	X	X	X			X	X	X	X	X	X	X		X	X		X	X									X	
6. RADAR SCATTEROMETER	?			?	?						?			?	?	X		X														
7. INFRARED IMAGER	X		X	X	X	X	X	X	X	X	X	X	X	X		X	X		X	X		X	X	X	X			X				
8. IR RADIOMETER/SPECTROMETER	X	X	X					X	X		X	X			X				X	X				X	X			X				
9. MICROWAVE IMAGER	X				X		?	X	X		X	?	X			X			?	?		X	X	X	X						X	
10. MICROWAVE RADIOMETER	?	?	X	X			?		X		X								X	?	X	X	X	X	X						X	
11. LASER ALTIMETER						X	X	X					X			X		X										X	X	X	X	
12. MAGNETOMETER	X														X	X	X									X		X	X	X		
13. GRAVITY GRADIOMETER						?									X												X	X	X	X		
14. ABSORPTION SPECTROSCOPY		X												?	?	X	X	X	X			X		X								
15. RADIO FREQUENCY REFLEC.	?																	X	X												X	
16. ULTRAVIOLET	X	X	?	?		?	?	X	X	X	X	X	?	X	X	X	X					X			?							
17. VIEW FINDER								X	X			X												X							X	

COMPILED BY THE U.S. GEOLOGICAL SURVEY

FIGURE 2 - Instrument types needed by geologic users.

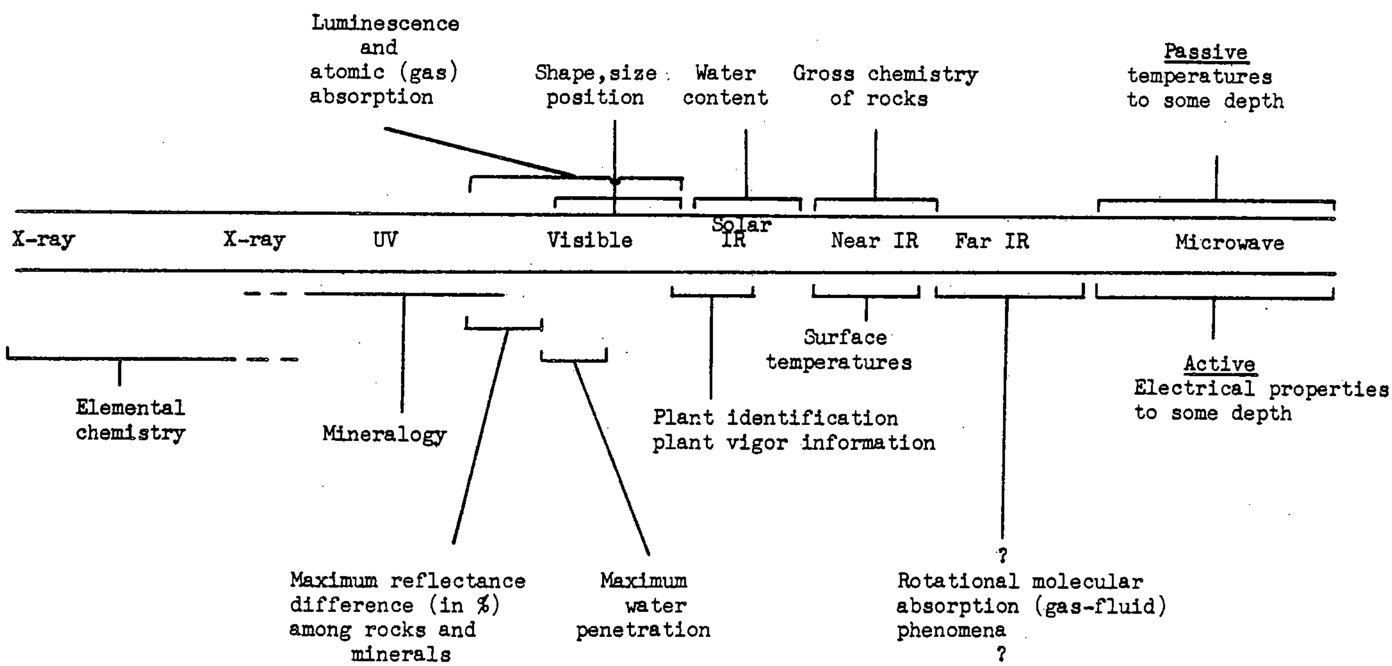

FIGURE 3 - Information interpreted in various parts of the electromagnetic spectrum.

Gamma Rays

Naturally-occurring gamma rays are emitted by terrain materials and can be measured by passive airborne detectors. In addition, specific gamma wavelengths are traceable to emitting isotopes which in turn can be used to identify the naturally-occurring materials. Unfortunately, most gamma-ray detectors are total-scintillation counters (measuring a broad band of high-frequency waves) so that mineral identification is dependent on a previous knowledge of what minerals are likely to occur in the area of study.

Gamma-ray spectrometers have been developed recently which, by means of filters, are able to measure the radiation counts within a series of narrow bands. Airborne gamma-ray spectrometers were used only after it was established that selective attenuation of gamma-ray does not occur in the earth's atmosphere. The full potential of gamma-ray remote-sensing devices has not been realized yet but they possess obvious utility in geologic studies. Several major instrumentation problems exist which must be solved before they become truly operational.

X-ray

The next major electromagnetic radiation band is that of x-rays. There are two reasons why this band is of little interest for geological applications of remote sensing: 1) The probability is very low that x-rays from natural radiation decay or solar-induced re-radiation will be available from terrain materials. (This rules out a passive system.) 2) Presently it is infeasible to equip an airplane with an active x-ray system because of the considerable power requirements necessary for electron acceleration.

Ultraviolet

Film and filter combinations which are so effective for the visible and near-infrared spectra have not been developed for the far ultraviolet. In the higher ultraviolet frequencies, solar radiation is the energy source but scanning systems employing photomultiplier and filter combinations are used. Throughout the ultraviolet region there is only limited reflectance of energy because of atmospheric absorption. The ozone layer in the atmosphere attenuates much of the ultraviolet radiation and limits the usefulness of these wavelengths. Also, scattered radiation adds noise to the measuring system and a poor record is commonly obtained. Potential does exist for the application of the ultraviolet spectra but it has not been developed to date.

Visible

Some general comments can be made concerning near ultraviolet, visible, and near infrared since they are all commonly viewed using selective film-filter combinations. Photographic film may collect energy from different spectral ranges, for example, from .30 to .70 microns (which includes near ultraviolet and visible light) or .31 to .38 microns (entirely in ultraviolet), .31 to .50 microns (includes ultraviolet and some visible), .60 to .70 microns (entirely in the upper visible range) or .70 to .90 microns (in the near infrared).

Mechanical-optical scanning devices, which will be discussed later, measure narrow bands of radiation within the .3 to 14 microns range and yield more selective response in these individual bands than can be obtained from wide-band photography.

Obviously the things most readily obtained in the visible range are those observed by the naked eye, that is, patterns due to geometric form and color or photographic tone. Advantages of photography are the large areas covered and presented on workable scales, a three-dimensional view, a historical record, and a uniform record obtained in an economical and speedy manner.

Infrared

Near infrared, extending from .7 to 3 microns, contains the photographic IR portion of the spectrum. In fact, most exposures are made in a band ranging from .7 to .9 microns. Film-filter combinations are used to obtain color and black-and-white photographs in these wavelengths.

On infrared black-and-white film, the gray tones result from the reflectiveness of an object in infrared and not from its true color. Broad-leaved vegetation is highly reflective and appears in light tones whereas conifers absorb infrared radiation to yield much darker tones. Bodies of water absorb a high degree of infrared radiation and will register a dark tone unless the water is heavily laden with silt. Infrared photography generally penetrates haze or moisture-laden clouds.

Infrared color or camouflage-detection film is a false-color, reversed film that was originally designed to detect the difference between healthy vegetation and artificial objects made to simulate the color of foliage (camouflage). Natural deciduous foliage appears magenta or red, while false foliage is purple or blue. This film has also proved useful in the early detection of plant diseases and insect outbreaks in forest areas.

In visual color the difference between deciduous trees and conifers is not obvious, but as healthy deciduous trees have a much higher infrared reflectivity than healthy conifers, there is an obvious difference in color between these trees when compared on infrared color film. During spring and summer, healthy deciduous trees appear magenta or red and healthy evergreens bluish-purple. Dead deciduous leaves or dead evergreen needles usually photograph bright green as they have lost the infrared reflectivity. In the fall, healthy deciduous trees whose leaves have turned red or yellow retain some infrared reflectivity and therefore red leaves photograph yellow and yellow ones white.

Infrared color film is also useful in conducting studies on water pollution. The infrared reflectivity of the water is influenced by the suspended solids so that considerable detail on water circulation, discharge of effluent, and so on can be observed.

In general it can be said that color-infrared film is most useful in the study of vegetation and of water bodies. Normally it is used in combination with panchromatic film and interpretation is made by comparing the information in the visual and in the photographic infrared regions.

The thermal-infrared portion of the spectrum extends from 3 to 15 microns. In the visible region most energy is reflected solar radiation, but in the middle and far infrared, most of the energy is from natural emissions of the objects themselves. Infrared energy is emitted by any substance which has a temperature above absolute zero (-273°C) due to the vibration and rotation of atoms and molecules which make up the substance.

The wavelength at which the energy peaks is generally controlled by thermodynamic temperature but the level or amount of the energy is controlled by the emissivity of the substance. Emissivity, which is determined primarily by the surface conditions of an object, is expressed as a ratio of the energy radiated from an object in relation to a "black body." By definition, a black body is an object that completely absorbs all radiation incident upon it and hence the emissivity of a black body is unity. A highly polished surface is an extremely poor radiator and absorber, with an emissivity close to zero. Most surfaces in thermal mapping lie between these two extremes.

Natural terrain materials generally peak between 9 and 10 microns because terrain temperatures are about 300°K. It is a fortunate phenomenon that these wavelengths are easily transmitted through the atmosphere, because attenuation over this range is minimal. Atmospheric transmission characteristics must be considered when working with the electromagnetic spectrum. Figure 4 indicates the transmission characteristics of the atmosphere. The major cause of attenuation is water vapor, but CO_2, O_2, and O_3 have absorption spectra as well. Most infrared sensing systems are designed to operate in the spectral bands corresponding to "infrared windows" (non-shaded areas of figure 4) within the approximate ranges of 2-5 and 8-14 microns.

As previously indicated, the shorter infrared wavelengths (.7-1 microns) can be recorded by conventional photography. For wavelengths greater than 1 micron however, highly specialized sensing devices are required. These function somewhat like a television receiver by producing a nearly continuous image from a series of scan lines. As the terrain is not photographed directly the term "infrared imagery" is used to describe the final image printed on photographic film.

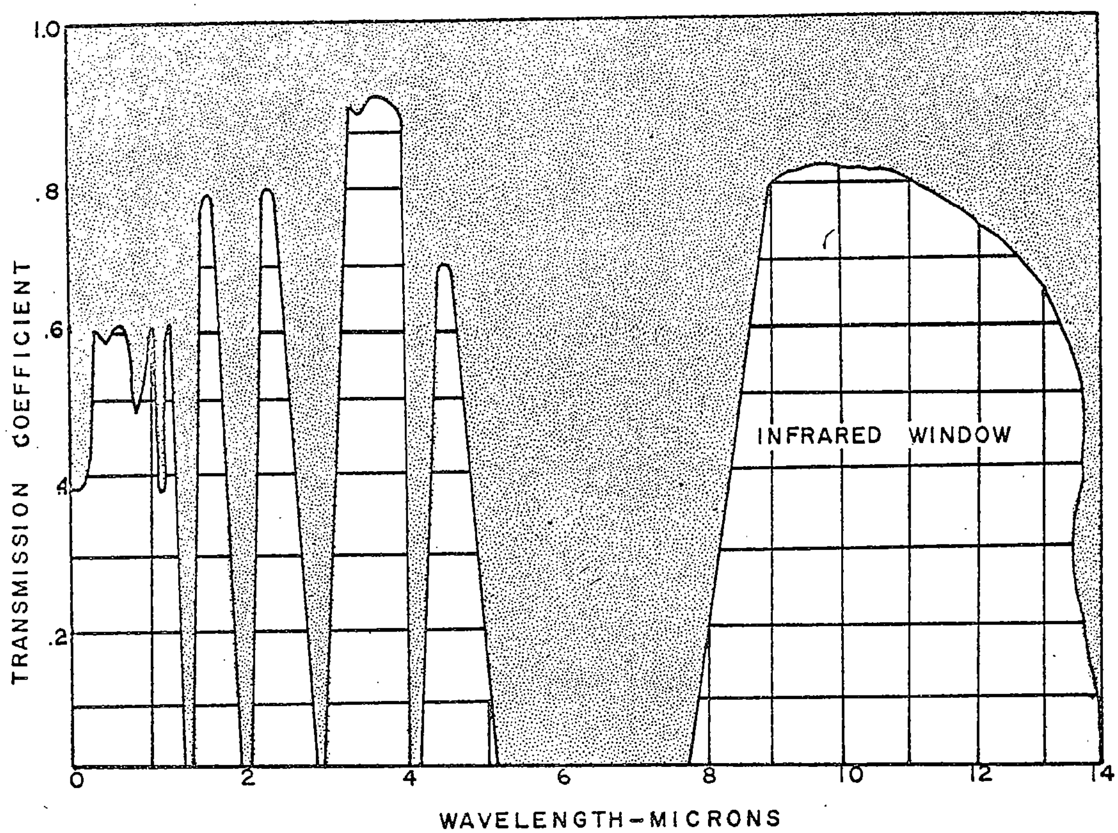

FIGURE 4 - Transmission spectra of the atmosphere - radiation transmitted in non-shaded areas.

The applications of thermal imagery are numerous. This method is ideally suited for the detection and mapping of forest fires. It can be used at night and has the ability to penetrate smoke. In agricultural studies, thermal imagery is used to identify crop species, soil types, detecting crop diseases, making animal censuses, and determining relative moisture content of various soils. In hydrology problems, studies can determine where hot effluent is being discharged into streams or where cool ground-water is emerging. Fresh-water supplies have been found in Hawaii by detailed thermal surveys. Military applications are varied and include, among others, detection of man's activities in a forested or remote area.

Geological applications of thermal imagery constitute a growing list. Included are the mapping of faults and other geologic structures, distinguishing major rock types in an area (such as basalt, pyroclastics, and alluvium), conducting petroleum-exploration surveys, maintaining surveillance of active volcanoes, locating subsurface burning areas in coal fields, and finding hot subsurface water in volcanic areas such as Yellowstone Park.

Submillimeter

The region between the micron wavelengths (infrared) and the millimeter wavelength (radar) is known as the submillimeter range. Operational airborne sensors to date have not been flown for this region of the spectrum but the sensors are being developed. New instrumentation is required because there are no standard sources of calibrating equipment over this band.

These wavelengths are of interest because photoconductive and microwave devices are bound to meet in this region and currently it is not known which will prove more applicable. The geological potential in this realm is speculative because no information has been collected to test its significance.

Microwave

Radar or microwave systems operate in the approximate wave-length from 1 centimeter to 3 meters, and are active systems.

Radar systems, which were developed for wartime needs, can obtain terrain information under the cover of darkness and during all weather conditions. On radar imagery one can resolve field patterns and tonal effects related to vegetation, drainage and shoreline features. Also, some subsurface information is obtained as it is a rough equivalent of a photo mosaic but from a single image. This is accomplished at the cost of a reduced resolution (as compared to most types of aerial photography) and a forced dependence on data from other sources if complete analyses are expected. This results from the fact that snow, ice, and some vegetation may be penetrated by the radar, yielding an image dissimilar to the visual one. Therefore, radar photographs are a supplement rather than a replacement for conventional aerial photographs.

The list of radar capabilities is impressive. These include determination of some subsurface compositions and conditions; penetration of vegetation, snow, and ice; resolution of soil textures down to small gravel; moisture-content determinations, if temperature data are available; the reverse, surface temperature, if moisture content is known; metallic content of some surface and near-surface features; and the general rule that radar imagery can be obtained whenever aircraft can be sent aloft.

The two principal types of radar displays which have been used are the "plan position indicator" (PPI) and the "side-looking airborne radar" (SLAR). The PPI has the circular-sweep type display which is used so commonly for weather forecasting, commercial aircraft control, and air defense.

The SLAR equipment yields a continuous strip image of the terrain recorded on photographic film. Energy is transmitted from both sides of the aircraft and the reflected energy produces two continuous images, one for each side of the plane. These are photographed side-by-side on the same film to present a continuous presentation of the collected data. As previously stated, the resolution of the radar image is decidedly inferior to conventional photography, but in many cases the all-weather capability of radar more than offsets the lower resolution of terrain features.

Communication

Below the microwave region of the spectrum is a large segment used for communication. Only at the lower end of the radio portion are additional remote sensing devices applied.

Sub-Communication

In the frequency range from 10,000 to 100 cycles per second (approximately 20 Km to 20,000 Km wavelength) both active and passive sensors are being used. Active sensors rely on the basic conductivity characteristics of the terrain materials, as these characteristics dictate the flow of eddy currents when the material is pulsed with electromagnetic energy. The eddy currents produce an associated field whose rate of decay is a function of the conductivity of the material. An appropriate receiver, one which measures these eddy currents as a function of time, can yield data from which conductivity contrasts are obtained.

Passive sensors depend upon natural electromagnetic radiation, such as lightning discharges, as their primary source, but they actually measure polarization declinations caused by conductive bodies. Both active and passive devices have been used successfully for some time in airborne geophysical exploration work.

The primary advantage of these systems is their ability to penetrate overburden (even shallow water in some cases) to detect mineral deposits or conductive bodies at depth. The primary disadvantages are the towed "bird" configuration which house the receivers, the lack of predictable source signals for passive devices, and their relatively poor resolution.

AUTOMATIC SOIL AND ROCK CLASSIFICATION USING MULTISPECTRAL REMOTE-SENSING IMAGERY

This final discussion will be limited to a small portion of the electromagnetic spectrum, the visible through thermal infrared. The subject of automatic classification of soil and rock materials, utilizing high speed computers, will be discussed employing the LARS classification technique as an example. The LARS system is currently applied to the visible through infrared regions.

Perhaps the first question to answer in this regard is why use an automatic classifying system at all? Why not rely on the current method of mapping soil and rock for geological and other purposes. An obvious answer is that computers can make computations much faster and more accurately than people can, so a division of labor whereby calculations are performed by computers and the decision making is left to the scientist, should result in greater accomplishments. This, of course, does not remove the scientist from the operation, it makes him more important. Accurate field work is as necessary as before, if not more so, but less field effort is required to cover the same size area. Accurate data are needed to train the computer for proper classification of an area.

Research at the Laboratory for the Application of Remote Sensing (LARS) concerns the analysis by pattern recognition techniques of remote-sensing data collected by ground-based, airborne, and (ultimately) satellite-borne instruments. Figure 5 shows the three locations at which remote sensors can be placed. The diagram as a whole is called the energy flow profile. Just above the field is an elevated platform with an aircraft aloft, and finally an earth-orbiting satellite in space. In this paper the discussion will be limited to remote sensing from the airplane-based sensor.

Currently, multispectral imagery of the terrain is obtained for LARS by the Institute of Science and Technology, University of Michigan, using an optical-mechanical scanner. Such an airborne scanning device is illustrated in Figure 6. As the plane follows the flight path the energy radiated by a specific ground resolution element passes through the scanner optics, is divided according to its spectral wavelength, and directed to an appropriate detector. The output of all such detectors are simultaneously recorded by a multiband instrumentation recorder. The transverse motion provided by the rotating mirror and the forward motion of the aircraft cause a continuous signal to be recorded for each spectral band of the scanner output. The information is stored in analog form by the tape-recording system.

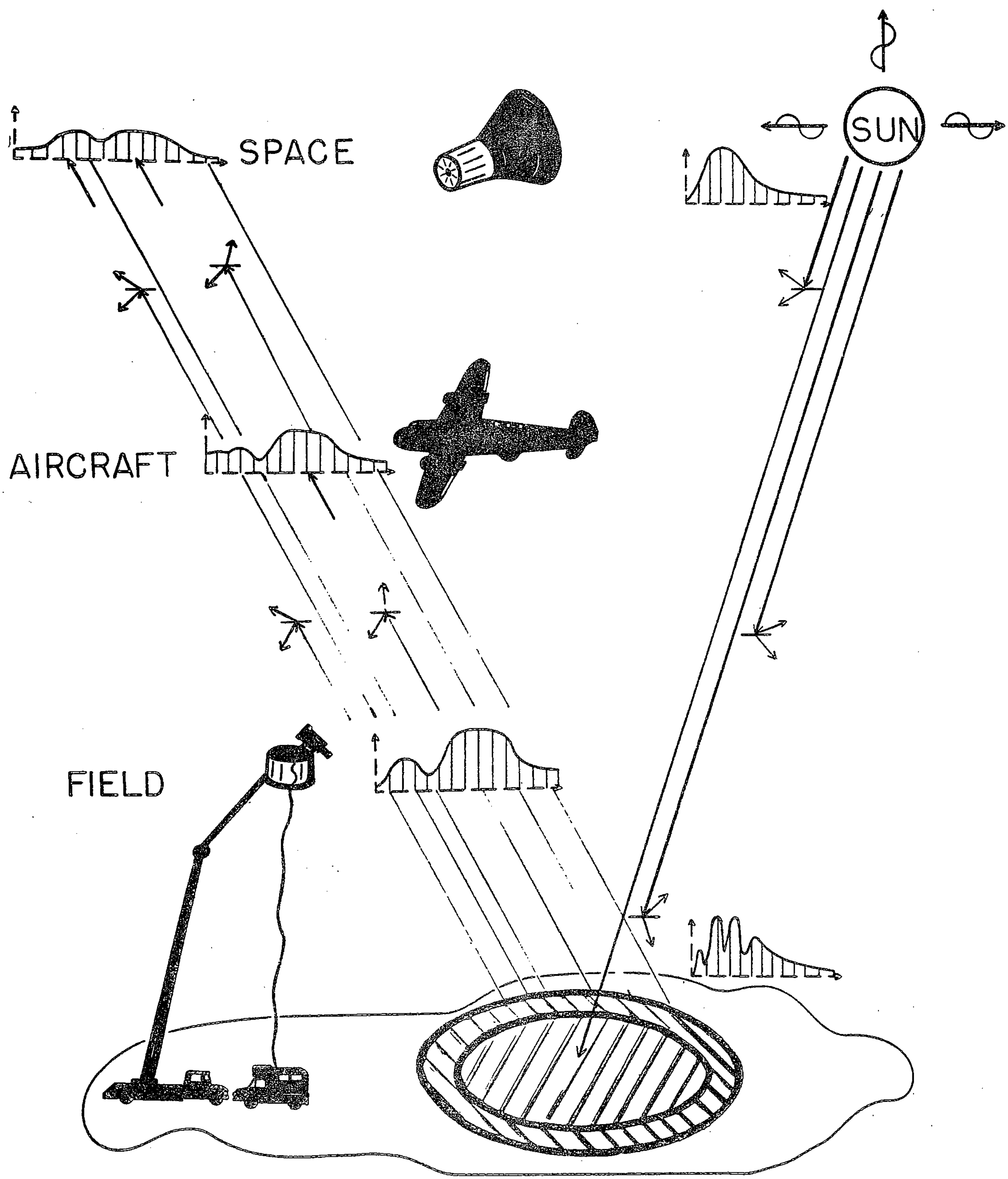

FIGURE 5 - Energy flow profile.

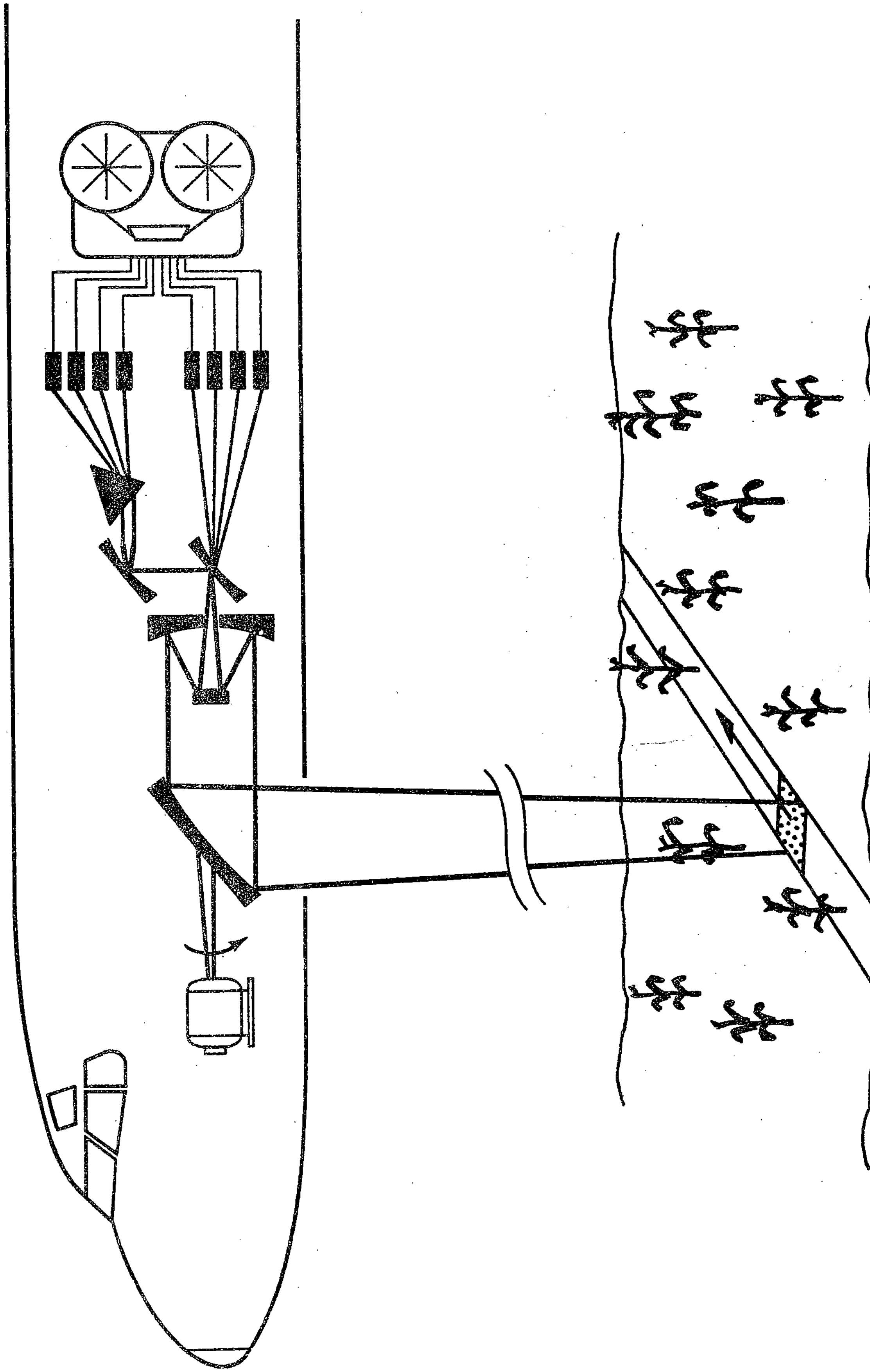

FIGURE 6 - Mechanical-optical scanning system.

The specific multispectral bands recorded on the aircraft storage tapes can differ according to the requests of the researcher. In the past, 18 channels have been recorded. Typically, 12 channels of data are processed, 10 from one end of the scanning device and the remaining two from the other end. Twelve channels have been used in the past to reduce the data-handling requirements for the LARS systems although more channels can be processed if the need arises. Typically, 10 channels are obtained from the .4- to 1.0-micron range plus two channels, 1.0-1.4 and 2.0-2.6 microns.

Under the LARS data-handling system, the analog data are first converted to a digital form. The digital approach is used because this system has greater speed and convenience for the researcher and has greater flexibility for data handling and analysis when using a general-purpose computer. An analog-to-digital conversion unit is used to obtain digital data and an IBM 360 Model 44 is used for the computation and analysis.

The following discussion is presented in a simplified fashion to give the reader a general idea of the analysis approach. It is by no means to be considered as a statistical justification for the method. More complete information can be obtained from references in the bibliography. Described here are those techniques which were available in the Spring of 1970 and are subject to change and improvement as research continues.

The first step in the analytical technique is to obtain grey-scale printouts of the data for several channels. Grey-scale printouts are digital displays of the spectral response of the area in question but are limited to a single band width of the spectrum, for example Channel 6, .58-.62 microns. They resemble a crude photograph of the area. Alpha numeric symbols (letters of the alphabet, typing symbols and single-digit numbers) are assigned to the radiance levels such that high intensity, light areas receive symbols which cover a low percentage of the paper, example (-), and dark areas receive symbols with a high percentage of coverage, example (M). The symbols are not simply assigned so that each represents the same amount of radiance because the data are not equally distributed between these end points. Instead, bins or segments are assigned to give the maximum contrast between the radiance levels where the points are concentrated.

Figure 7 shows several grey-scale printouts and the corresponding aerial photo. Tonal differences can be observed from one grey-scale printout to the other showing a difference in response in the spectral bands. Grey-scale printouts are used to locate field boundaries and other geographic features, that is, to find the line and column numbers (the address) of these terrain features in the digitized data. This information is relayed to the computer for training purposes.

Areas of known materials are manually located on the grey scale printouts by the researcher. This information is referred to as "ground truth" and is obtained by examining these areas in the field. In agricultural studies, these may consist of locations of various crops such as corn, wheat, oats, soy beans, and the like; in forestry studies, conifers and deciduous trees or individual tree species; and in geology, specific bedrock and soil types. A portion of the ground-truth areas is used to train the computer on the spectral characteristics of the classes of interest. The remaining areas are saved to test the accuracy of the computer classification. The computer is given training-field locations for each class using punch-card information. Boundary corners are indicated by column and line number which limits the fields to retangular shapes.

The actual field locations are checked by running another set of grey-scale printouts with the training fields outlined on them. This allows corrections to be made before the classification is begun.

The first step in the classification is to obtain histograms of the classes of material. An example of classes for a geologic study area might be alluvium, limestone soil, shale soil, trees, mixed crops and water. The histogram of each class shows the distribution of reflectance intensity for each channel designated. Histograms may be obtained for each channel at the discretion of the researcher. A unimodal distribution on the histogram suggests that it is a

FIGURE 7 - Computer printouts of three wavelength bands of multispectral data (.40-.44μ, .62-.66μ, .80-1.0μ).

single class spectrally. Bimodal and trimodal distributions are subdivided manually into separate unimodal classes by the researcher. Individual fields may be histogrammed to check for a multimodal situation within them.

The next step is to use a divergence analysis to indicate the best channels for classification. This simply tells which channels will give the best classification. All 12 channels are not used because it requires an excessive amount of computer time to consider them all and the accuracy may not meaningfully increase in the process. Four to six channels are generally selected. The divergence analysis also indicates how separable the designated classes will be. Quite often needed changes in the individual classes are indicated by this analysis and some of the previous steps are repeated.

Following this operation, the training fields are classified by the computer. For each data point a decision is made concerning which class it should be designated (if the answer is not unique, a blank space is left). The computer then calculates how accurately it classified the training fields by comparing its classification of each point to what it was designated by the researcher. A high percentage of accuracy at this juncture means that the training-field classification is consistent with what the researcher trained the computer to recognize, but most important, consistent when all training fields and classes are being considered.

The final steps are to print out a computer classification for the whole area and to determine how well the test fields were classified. Recalling that the test fields are areas of known material from ground-truth studies, this indicates how well the classification works for areas other than training fields. If test fields and training fields show a high degree of accuracy, the classification is a good one (assuming the test fields are representative of the entire area). If the accuracy is low, some reworking of the classes may be in order or the researcher may decide that the classes selected are not separable.

Figure 8 is a computer classification of an area. Included in this classification are soil types, vegetation and water. For

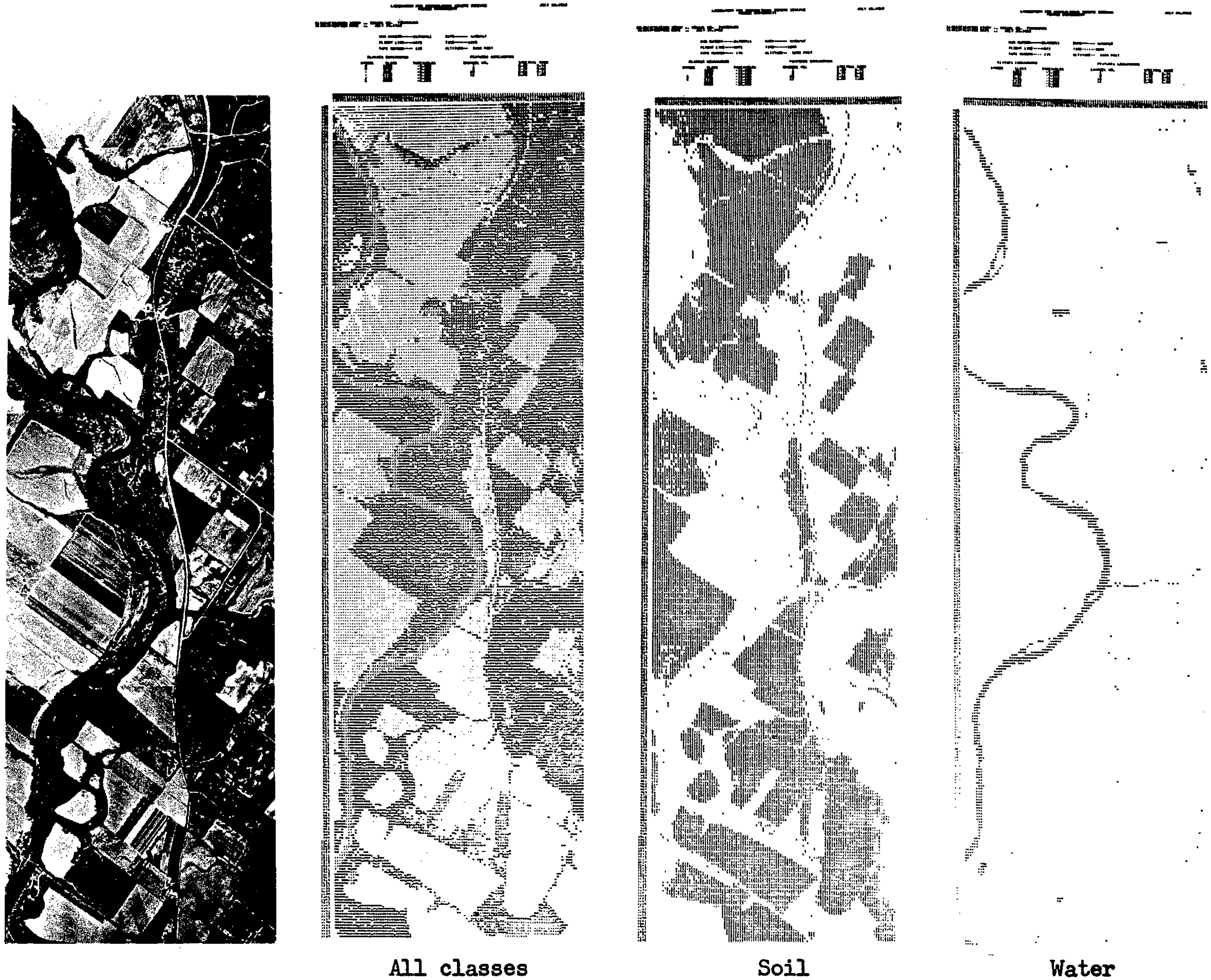

FIGURE 8 Aerial photograph and computer printouts showing automatic identification of trees, green vegetation, soil, and water.

better visualization, computer classifications may be colored, which accentuates the different areas classified.

As a final note, the following are presented as examples of materials which have been mapped or classified using the LARS automatic-classification technique.

1. Determination of crop types in Tippecanoe County, Indiana, and in the Midwest corn belt. (3,4,7,8,11)*
2. Detailed soils mapping of Indiana farms to determine moisture content, clay content, and so forth. (7)
3. Engineering soils-mapping of an area south of Indianapolis, Indiana, in a study for Indiana Highway 37. (16)
4. Geologic reconnaissance mapping in mountainous terrain. (17)
5. Detailed geologic and agricultural soils-mapping from Apollo 9 photographs using three-spectral-band photography. (10)
6. Forest-vegetation type studies. (3,4)
7. Hydrology studies (1)

In the future, LARS will strive to develop further the technology of remote sensing and its application to the classification of natural materials. LARS looks forward to receiving remotely sensed data from NASA's first experimental Earth Resources satellites.

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REMARKS ON KANSAS HIGHWAY RESEARCH PROJECT IN REMOTE SENSING FOR SOILS AND GEOLOGIC MAPPING

Alvis H. Stallard*

The State Highway Commission of Kansas, in cooperation with the Bureau of Public Roads, is currently conducting research using remote sensors for soils investigations. The main objective of the project is to map engineering soil groups using black-and-white, color, color-infrared, and narrow-band photography, as well as data from remote sensors.

A 25-mile by 1-mile corridor of land in Jefferson County, Kansas was selected as a test site. The selection was based on the presence of a large number of different engineering soil types, the availability of geologic data, the availability of Soil Conservation Service data, and the proximity of the site to Topeka.

Planning photography of the test site was obtained in March, 1969 and additional photography was procured during the summer, 1969. Table I is a "recap" of the types of photography procurred during 1969. All photography was taken by the State Highway Commission of Kansas.

TABLE I

Photography	Date	Scale
Black and White Panchromatic (Planning Photography)	3-17-69	1:18,000
Color Ektachrome	7-28-69	1: 2,000
Color Ektachrome	7-28-69	1:10,000
Color Infrared	9-05-69	1:12,000

In June, 1969, the scope of the project was increased to include three pavement-test sites in Shawnee, Franklin, and Johnson Counties, and a mined-out area in Wyandotte County. Remote Sensing, Inc., Houston, Texas, was engaged by the State Highway Commission of Kansas to procure the remote-sensing data and imagery. Their capabilities included an RS-14 dual-channel infrared scanner (3.5-5

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and 8-14 micron range), 13.3 GHZ scatterometer, 13.7 GHZ microwave radiometer, four-camera cluster of Hasselblad 70 mm cameras (narrow-band photography), and a Wild RC-8 camera. The University of Michigan, Willow Run Laboratories, was engaged by the Bureau of Public Roads to conduct a multisensor flight over the Jefferson County (soil) site, the Wyandotte County (mined-out) site, and the Johnson County (pavement) site. Their capabilities included an 18-channel multisensor system which gathers data between .32 and 14 microns.

During the fall of 1969, the initial landform classification was completed on the planning photography for the Jefferson County site. Landforms were used as the basis for planning the collection of ground-truth data. Soils in most landforms were evaluated as to color (Munsell color notations) and grain size. As a result, ground-truth collection stations were selected at 111 different sites within the test area.

On March 14 and 15, ground-reflectance readings were taken at 12 stations which represented engineering soil groups most difficult to differentiate on aerial photography. As a result of the reflectance tests, we found that the greatest difference in energy reflected between two or more given soils occurred in the near-infrared region and the red band in the visible range (.62 to one micron). These data were used to select the film-filter combination for the Hasselblad cameras (see Table II).

TABLE II

Camera	Film	Filter	Band Photographed
Hasselblad Camera #1	Panchromatic Plus X No. 2402	HF-3 (Haze)	.4-.72 Microns
Hasselblad Camera #2	Panchromatic Plus X No. 2402	25A	.58-.72 Microns
Hasselblad Camera #3	Infrared Aerographic No. 2424	15	.52-.9 Microns
Hasselblad Camera #4	Infrared Aerographic No. 2424	87C	.81-.9 Microns

The remote-sensing flights were flown on March 23, 24, and 27 and on April 4, 1970. Table III recaps the time of flights and systems used:

TABLE III

Contractor	Time & Date	Test Site Cov.	Systems Used
Remote Sensing, Inc.	Daytime, March 23, 1970	All sites.	All systems
Remote Sensing, Inc.	Nighttime, March 24, 1970	Jefferson and Wyandotte county sites.	All systems but cameras.
Univ. of Michigan	Nighttime, March 27, 1970	Jefferson and Wyandotte county sites.	Only infrared channels
Univ. of Michigan	Daytime, April 4, 1970	Jefferson, Wyandotte and Johnson county sites.	All channels

Table IV shows data obtained for each site.

TABLE IV

Jefferson County (Soils-test site)

1. Two sets - Color Ektachrome. Scales 1:2,000 and 1:8,000. Procured by State Highway Commission of Kansas.
2. Two sets - Color Infrared. Scales 1:2,000 and 1:8,000. Procured by State Highway Commission of Kansas.
3. Two sets - Black and White Panchromatic Plus X. Scales 1:2,000. and 1:8,000.
   Remote Sensing, Inc.
4. Two sets - 70 mm Black and White Panchromatic Plus X (.4-.72 microns). Scales 1:8,000 and 1:32,000.
   Remote Sensing, Inc.
5. Two sets - 70 mm Black and White Panchromatic Plus X (.58-.72 microns). Scales 1:8,000 and 1:32,000.
   Remote Sensing, Inc.
6. Two sets - 70 mm Infrared Aerographic (.52-.9 microns). Scales 1:8,000 and 1:32,000.
   Remote Sensing, Inc.

TABLE IV
continued

7. Two sets - 70 mm Infrared Aerographic (.81-.9 microns). Scales 1:8,000 and 1:32,000.
   Remote Sensing, Inc.
8. Four sets - Infrared Imagery (3.5-5 microns) day and night at altitudes of 1,000 and 4,000 feet.
   Remote Sensing, Inc.
9. Four sets - Infrared Imagery (8-14 microns) day and night at altitudes of 1,000 and 4,000 feet.
   Remote Sensing, Inc.
10. Four sets - 13.3 GHZ Scatterometer data. Day and night at altitudes of 1,000 and 4,000 feet.
    Remote Sensing, Inc.
11. Four sets - 13.7 GHZ Microwave data. Day and night at altitudes of 1,000 and 4,000 feet.
    Remote Sensing, Inc.
12. Two sets of data for each of 18 channels using the multisensor system at 1,000 and 4,000 feet altitudes - daytime.
    University of Michigan.
13. Two sets of data for four infrared channels of the multisensor system at 1,000 and 4,000 feet altitudes - nighttime.
    University of Michigan.

<u>A total of 74 different sets of data.</u>

<u>Wyandotte County</u> (mined-out area), not associated with soils investigation.

1. One set of Black and White Panchromatic Plus X photography. Scale 1:5,000.
   Remote Sensing, Inc.
2. Two sets each Infrared Imagery 3.5-5 microns and 8-14 microns range. Day and night at an altitude of 1,500 feet.
   Remote Sensing, Inc.
3. 13.3 GHA Scatterometer data - nighttime at an altitude of 1,500 feet.
   Remote Sensing, Inc.

TABLE IV
continued

4. 13.7 Microwave Radiometer data - nighttime at an altitude of 1,500 feet.
   Remote Sensing, Inc.
5. One set of data, 18-channel multisensor system at an altitude of 1,500 feet.
   University of Michigan.
6. One set of data of four infrared channels of the multisensor system at an altitude of 1,500 feet.
   University of Michigan.

A total of 29 different set of data.

Johnson County (pavement-test site), not associated with the soils investigation.

1. One set of Black and White Panchromatic Plus X photography. Scale of 1:3,000.
   Remote Sensing, Inc.
2. One set each of infrared imagery 3.5-5 micron and 8-14 micron range - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.
3. One set - 13.3 GHZ Scatterometer data - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.
4. One set - 13.7 GHZ Microwave Radiometer data - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.
5. One set each of the 18-channel multisensor system - daytime at 1,500 feet altitude.
   University of Michigan.
6. One set each of four infrared channels of the multisensor system - nighttime at 1,500 feet altitude.
   University of Michigan.

A total of 27 different sets of data.

TABLE IV
continued

Franklin County (pavement-test site), not associated with soils investigation

1. One set of Black and White Panchromatic Plus X photography. Scale 1:3,000.
   Remote Sensing, Inc.
2. One set each of infrared imagery 3.5-5 micron and 8-14 micron range - daytime taken at 1,500 feet altitude.
   Remote Sensing, Inc.
3. One set - 13.3 GHZ Scatterometer data - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.
4. One set - 13.7 GHZ Microwave Radiometer data - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.

A total of 5 different sets of data.

Shawnee County (pavement-test site), not associated with soils investigation.

1. One set of Black and White Panchromatic Plus X photography. Scale 1:3,000.
   Remote Sensing, Inc.
2. One set each of infrared imagery 3.5-5 micron and 8-14 micron range - daytime taken at 1,500 feet altitude.
   Remote Sensing, Inc.
3. One set - 13.3 GHZ Scatterometer data - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.
4. One set - 13.7 GHZ Microwave Radiometer data - daytime at 1,500 feet altitude.
   Remote Sensing, Inc.

A total of 5 different sets of data.

To date the data are being developed and processed. No analyses have been completed. The visual analysis, to be conducted by the

State Highway Commission of Kansas, should be completed by June, 1971. The automatic interpretation, to be conducted by the Bureau of Public Roads, should be completed in approximately 2 years.

THE INFLUENCE OF GEOLOGICAL FACTORS UPON THE MECHANICAL PROPERTIES OF ROAD SURFACING AGGREGATES (WITH PARTICULAR REFERENCE TO BRITISH CONDITIONS AND PRACTICE)

Alan Hartley*

INTRODUCTION

Because of the ever-increasing expansion of highway and other forms of construction throughout the world, the supply of suitable aggregate materials must also necessarily increase. Please and Pike[1] have forecast a doubling of the demand for road aggregates in Britain in the next 10 years and have indicated that there is likely to be a similar increase in demand for all constructional aggregates in that time. It is also evident that problems are likely to occur in the provision of high quality aggregates used for surfacing as these are the ones most in demand for the manufacture of high-strength concretes. (A 60-percent increased demand was forecast for bituminous mixtures and surface dressings[1].) Sensible selection of aggregates for all types of construction work is therefore an economic necessity if natural aggregate resources are not be wantonly squandered.

In the U. K. the selection of aggregate is generally based on three main criteria: i) experience, ii) reputation, and iii) the results of mechanical property tests. The author believes that these criteria need to be supplemented with a more detailed knowledge of the effect of physical characteristics upon the mechanical properties of aggregates. In the last two decades, a great deal of work has gone into the study of the polishing of roadstones (the phenomenon was, of course, noted much earlier) and it was this characteristic which first interested the author in the geological factors affecting the behaviour of aggregates. Not surprisingly, therefore, this paper will be largely concerned with the relationship between the petrology and the polishing of

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natural rock aggregates, although other mechanical properties and other forms of aggregate will be discussed.

REQUIREMENTS OF A PAVEMENT SURFACE

Although the surfacing, particularly the wearing course, represents only a small proportion of the total depth, it accounts for a much higher proportion of the total cost of the pavement. An ideal road surface should be:

a) durable, hardwearing, and unaffected by weathering;
b) impermeable, and of sufficient strength to prevent failure during emplacement and service;
c) of low tractive resistance, yet not liable to cause skidding;
d) quiet running;
e) capable of being easily cleaned;
f) easy to excavate and reinstate; and
g) economic to construct and maintain.

In addition, a light colour may assist night visibility, yet care must be taken to ensure that there is no glare under bright-light conditions. Some of these factors influence the choice of roadstone but since it is impossible to fulfill all these conditions, the final selection is a matter of compromise.

The requirements of the surfacing aggregate must therefore be similar to the overall properties quoted, i.e., it must be of sufficient strength, durable, chemically inert, clean, relatively impermeable, unaffected by frost, of suitable surface texture and grading, and economical to quarry and transport.

Because the surface is that part of the pavement which requires the greatest maintenance and relatively frequent replacement, the durability factor is of considerable economic importance. The term "durability" used here is as defined by Hosking[(2)]: "in the sense of 'long lasting' and includes resistance to wear, resistance to weathering and resistance to any other factor that might impair long life in a road surface."

DURABILITY

In accordance with the definition given above, durability includes all those factors likely to impair the long life of a road surface. Thus an aggregate may be considered to have failed if it:

a) disintegrates because of mechanical crushing, by the action of frost, or by chemical weathering;

b) strips away from the binding medium because of lack of adhesion;

c) is rapidly worn away by the action of traffic and/or weather; or

d) reaches a state of polish such that it is "skid-prone."

The failure may be caused by a combination of these various factors. (Unfortunately it is sometimes apparent that failure because of (d) alone is thought of as an insufficient reason for having the road surface remade.)

These various ways of failure are largely dependent upon the petrological characteristics of the stone which are themselves caused by the effects of various geological environments which have existed during the lifetime of the rock. The author has described the effects of geological environment upon the characteristics of aggregates in a recent paper[3] and this aspect will not be discussed further. However, it would seem appropriate at this point to restate the nomenclature system used for roadstones in the U. K. because it was derived from a mixture of geological and mechanical property factors.

NOMENCLATURE USED FOR DESCRIBING ROADSTONES IN THE U. K.

The innumerable subdivisions of the three broad geological rock types (igneous, sedimentary, and metamorphic) based on a variety of considerations leads to a mass of rock terms which, while necessary to the geologist, is far too cumbersome for general use in highway engineering. For this reason, both engineers and those persons connected with the stone-producing industry adopted, and still use, general descriptive terms for the materials which they are handling, e.g., "granite" for any

Andesite	Gabbro	Limestone
"Artificial"	Granite	Porphyry
Basalt	Grit	Quartzite
Flint	Hornfels	Schist

TABLE 1. Trade Group Classification For Roadstones, 1913.(4)

"ARTIFICIAL" GROUP		**GRITSTONE GROUP**	
Slag		Agglomerate	Sed.
		Arkose	Sed. Si.
BASALT GROUP		Breccia	Sed.
Andesite	Ig. Int. F.	Conglomerate	Sed.
Basalt	Ig. Bas. F.	Greywacke	Sed. Si.
Basic porphyrites	Ig. Int. M.	Grit	Sed. Si.
Diabase	Ig. Bas. M.	Sandstone	Sed. Si.
Dolerite	Ig. Bas. M.	Tuff	Sed.
Epidiorite	Met. R.		
Hornblende-schist	Met. R.	**HORNFELS GROUP**	
Lamprophyre	Ig. Bas.	Contact-altered rocks of all kinds except marble	Met. Th.
Quartz-dolerite	Ig. Bas. M.		
Spilite	Ig. Bas. F.	**LIMESTONE GROUP**	
Teschenite	Ig. Bas.	Dolomite	Sed. Ca.
Theralite	Ig. Bas.	Limestone	Sed. Ca.
		Marble	Met. Th.
FLINT GROUP			
Chert	Sed. Si.	**PORPHYRY GROUP**	
Flint	Sed. Si.	Aplite	—
		Dacite	Ig. Ac. F.
GABBRO GROUP		Felsite	Ig. Ac. F.
Basic diorite	Ig. Int. C.	Granophyre	Ig. Ac. M.
Basic gneiss	Met. R.	Keratophyre	Ig. Int. F.
Gabbro	Ig. Bas. C.	Microgranite	Ig. Ac. M.
Hornblende rock	Ig. U.	Porphyry	Ig. Int. M.
Norite	Ig. Bas. C.	Quartz-porphyrite	Ig. Int. M.
Peridotite	Ig. U.	Rhyolite	Ig. Ac. F.
Picrite	Ig. U.	Trachyte	Ig. Int. F.
Serpentine	—		
		QUARTZITE GROUP	
GRANITE GROUP		Ganister	—
Gneiss	Met. R.	Quartzitic sandstone	Sed. Si.
Granite	Ig. Ac. C.	Recrystallized quartzite	Met. Th.
Granodiorite	Ig. Ac. C.		
Granulite	Met. R.	**SCHIST GROUP**	
Pegmatite	—	Phyllite	Met. R.
Quartz-diorite	Ig. Int. C.	Schist	Met. R.
Syenite	Ig. Int. C.	Slate	Met. R.

KEY

Ig. = Igneous
Sed. = Sedimentary
Met. = Metamorphic

Ac. = Acid
Int. = Intermediate
Bas. = Basic
U. = Ultrabasic

C. = Coarse-grained
M. = Medium-grained
F. = Fine-grained

Ca. = Calcareous
Si. = Siliceous

Th. = Thermal
R. = Regional

NOTE. It is recognised that traditional names are in use for describing certain rocks. If such terms are used, the rock in question should also be described by the appropriate trade group name from the list above.

TABLE 2.

TRADE GROUPS OF ROADMAKING AGGREGATES (6) hard rock, or "marble" for a part or fully altered limestone. In order to overcome this difficulty and to establish a standard system of nomenclature, the British Standards Institution introduced a Trade Group Classification for roadstones in 1913 containing twelve groups[4], Table 1. In 1943 the number of groups was reduced to eleven[5] by combining "Andesite" into the Basalt group. Table 2[6], indicates the distribution of well known rock types within the groups.

According to Phemister and others[7] the groups have been compiled "with a view to associating in the same trade group rocks which may be expected to behave similarly as roadstone." The author is of the opinion that the range of test results obtained from "similar" roadstones is such that the assumption that they can be placed in the same group is completely wrong, a matter which will be referred to later in this paper.

This Trade Group Classification has been described in some detail because the bulk of research work carried out in the U. K. uses the system for nomenclature purposes.

a) <u>Type of aggregate used in wearing surface</u>.
b) The relationship between aggregate and binder.
c) The type and degree of trafficking.
d) <u>Climatic conditions prevailing (average and seasonal variations)</u>.
e) <u>Surface conditions prevailing, i.e., influence detritus and degree of moisture retention</u>.
f) Geometric design of the surface.

TABLE 3. Factors affecting the durability of a road surface.

GENERAL FACTORS AFFECTING THE DURABILITY OF ROADSTONES

Table 3 lists the factors affecting the durability of a road surface. These factors are interrelated to some extent, as is shown in Figure 1[8]. As this paper is concerned with the geological and mechanical properties of the aggregate itself, only items a), d), and e) underlined in Table 3, will be discussed in depth although comments will be made upon the other factors where appropriate.

GEOLOGICAL FACTORS AFFECTING THE DURABILITY OF ROADSTONES

As was stated earlier, the geological factors affecting natural rock masses, and hence the quarrying of aggregates, are not included in this paper. The geological factors affecting the performance of aggregated material are therefore mainly petrographic and related to the mineral content of the rock and the petrofabric texture.

As naturally occurring rock materials are an aggregation of mineral particles, it follows that the properties of the individual minerals, their relative proportions, and their relationship one to another are to some extent reflected in the properties of the rock as a whole. Three important mineralogical characteristics affecting durability are hardness, cleavage, and alteration.

Whilst the hardness of a mineral is denoted quantitatively by various means, it is itself a quality which is difficult to define in precise terms. Moh's scale of hardness, which is probably the best known among geologists, refers to ability of one mineral to scratch the flat surface of another. More familiar to engineers are the Vickers and Brinell techniques which measure the ability of a surface to resist indentation. Young and Millman have shown that there is virtually a linear relationship between $\log^{M}$ and $\log^{HV}$ (where M is the Moh's Hardness number, and HV is the Vickers Hardness value using microscopic measurement of a diamond indentation). Their studies have also indicated the influence of mineral anisotropy upon microhardness values[9]. It should be noted here that the Knoop Hardness value, which is sometimes quoted, is measured in a manner similar to the Vickers Microhardness, the two being directly related.

The cleavage facility is the ability of a mineral to break along a fixed crystal plane. It varies from "perfect," as illustrated by the single cleavage of mica, to "none," as found in quartz which has no cleavage preference. Both hardness and cleavage are vector properties determined by the atomic arrangement within the crystal lattice. There is, as yet, no information to indicate a relationship between these two properties.

The chemical alteration of a mineral generally results in the production of softer minerals. Thus pyroxenes (H=5-7) are often converted to chlorites (H=2-2.5), orthoclase felspar (H=6-6.5), to kaolinite (H=2-2.5), and biotite mica (H=2.5-3) to chlorites (H=2-2.5) (see Table 4). A greater number of smaller crystals is usually formed and the process may be accompanied by an increase in stress due to the lower specific gravity of the

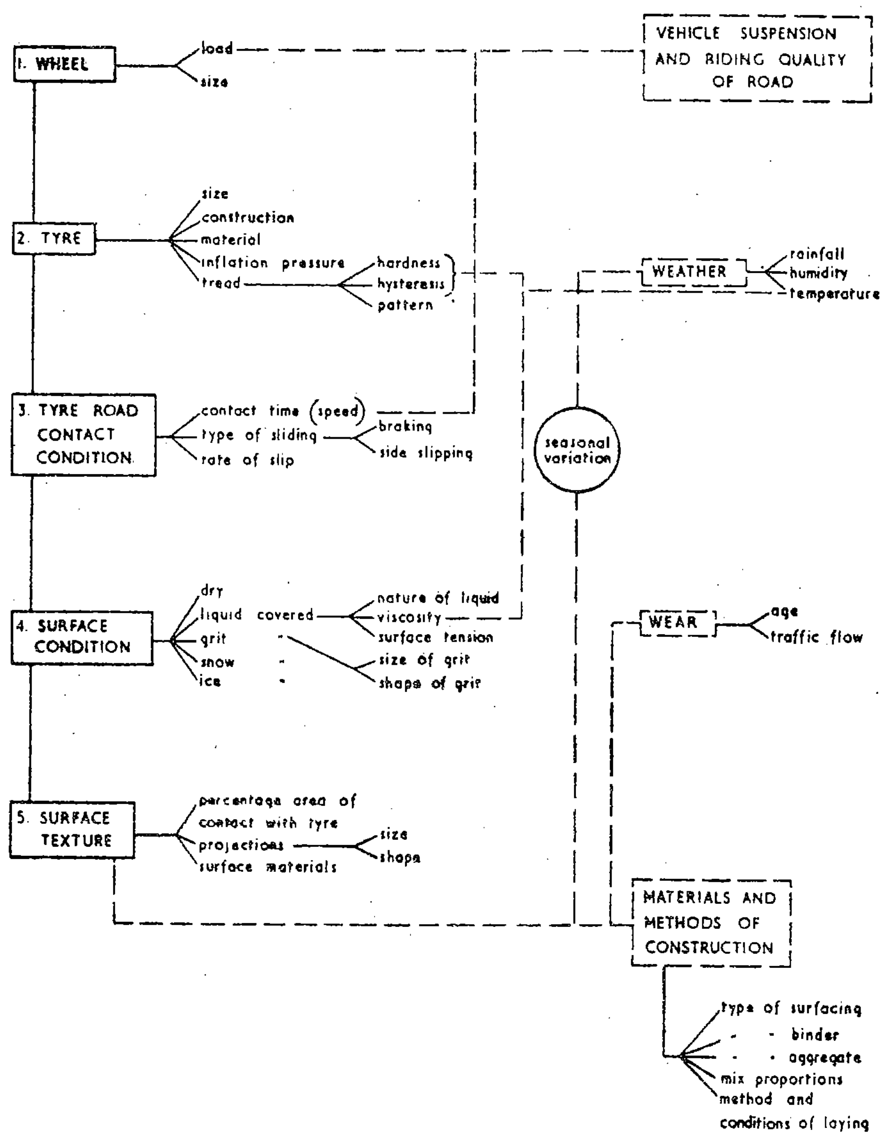

FIGURE 1. Factors affecting the durability of road surfacings[8].

new minerals. The way in which alteration proceeds is also important. Knight and Knight[10] noted that alteration may occur on the perimeter, within a crystal (kernel alteration), or throughout the whole crystal, and that the type of alteration can have a pronounced effect upon the durability - a point which will be referred to later. Cleavage and fracture also have an effect upon the form and extent of alteration of a mineral, as is shown in Plate 1.

The broken and ground-off minerals become part of the abrasive material acting on the remaining road surface. This, together with imported detritus and abraded rubber, acts on the surface until such time as it is removed from the road by heavy rain[11] (fig. 2). Even so, a certain amount of detritus remains embedded in vehicle tyres and continues the wearing process[12].

Table 4 lists the hardness, cleavage, and alteration characteristics of minerals commonly found in natural roadstones. It also indicates the rock types of which the minerals are the principal constituent.

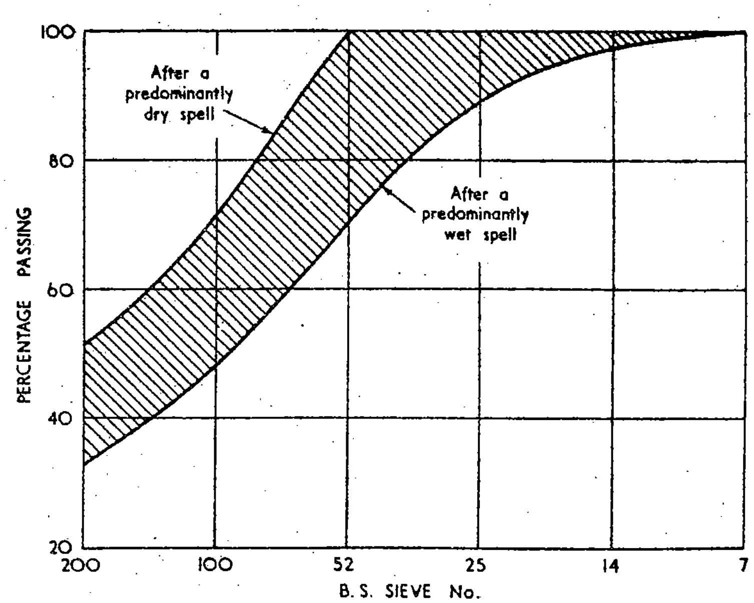

Figure 2. Range covered by gradings of samples of detritus recovered from the road surface under various weather conditions[11].

The petrofabric texture plays a major part in determining the durability of a roadstone[13]. Important features are:

a) the variety of minerals present and their relative proportions;

b) the size and shape of the mineral grains and their mode of combination;

c) the relative orientation of the minerals;

Plate 1. Biotite mica crystal showing perfect single cleavage (001) and alteration to chlorites around perimeter and along cleavages. (PPL x 8.5)

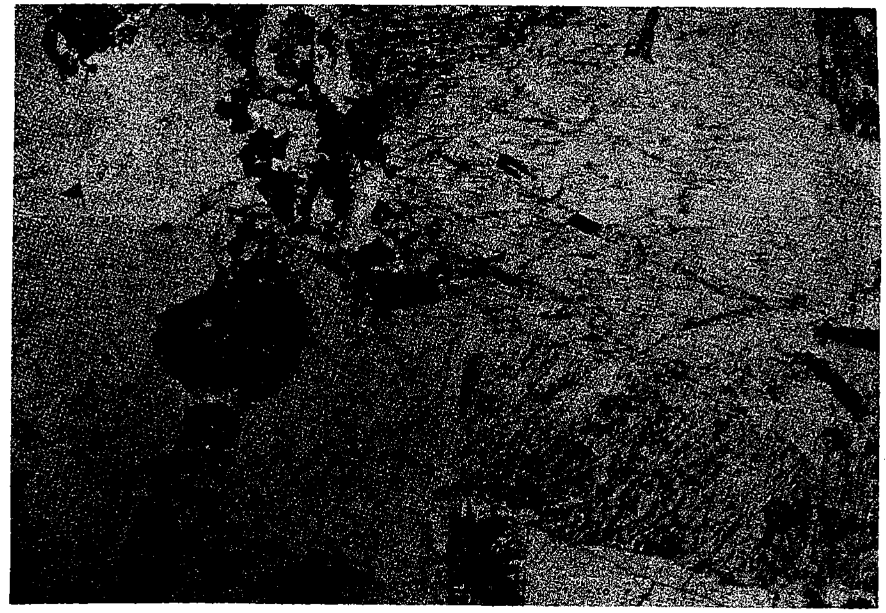

Plate 2. Granite (Lands End). Coarse interlocked grains of highly altered plagioclase, partially altered orthoclase, unaltered quartz and dark mica. (PPL x 8.5)

Plate 3. Gabbro. Coarse interlocking grains of calcic felspar, olivine and poikolitic pyroxene. (x 8.5)

Plate 4. Dolerite/Gabbro. Medium grained interlocking grains of felspar and pyroxene showing distinct orientation (foliation) with possible banding. (x 8.5)

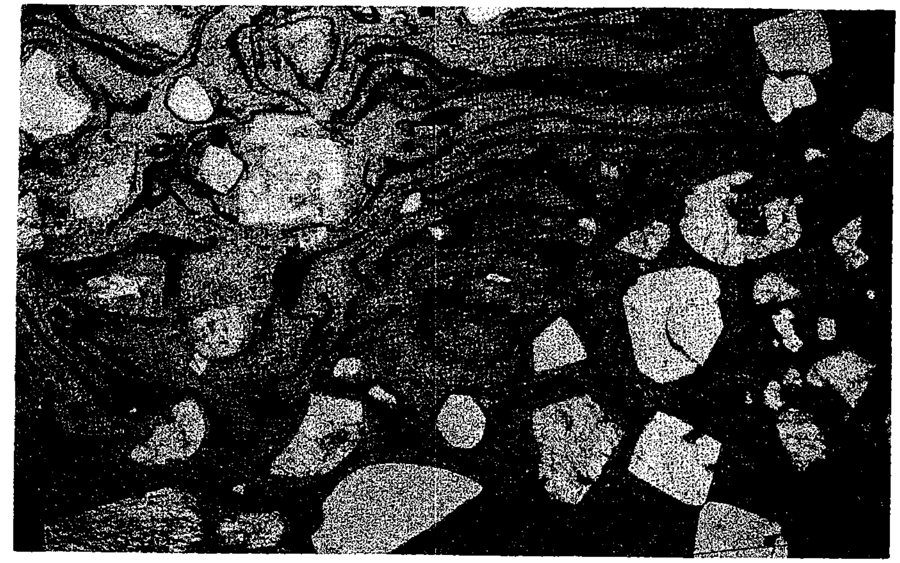

Plate 5. Quartz Porphyry (Dun Dubh, Arran). Phenocrysts of quartz and felspar set in a cryptocrystalline ground mass which clearly shows flow structure. (PPL x 8.5)

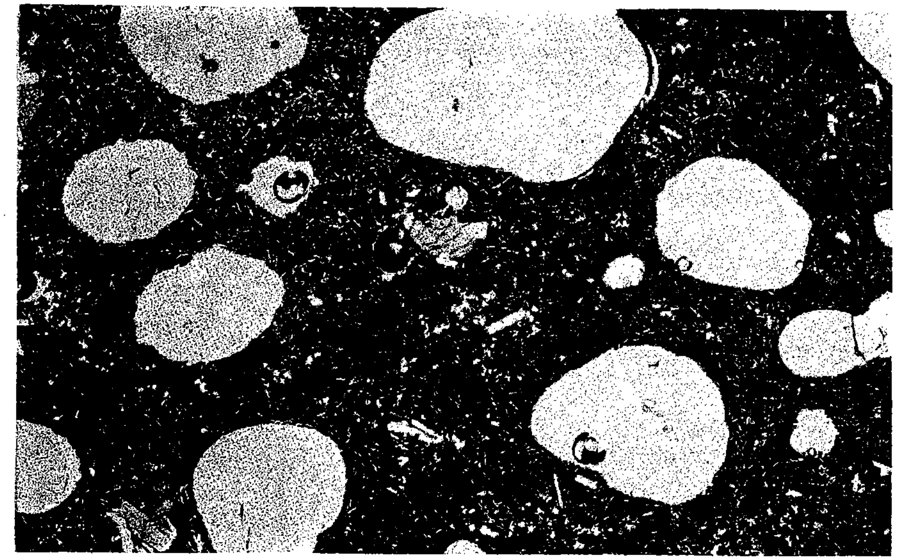

Plate 6. Vesicular Basalt (Crater Bory). Fine grained interlocking ground mass of interlocking felspars and pyroxene with relatively large well dispersed gas cavities. (PPL x 8.5)

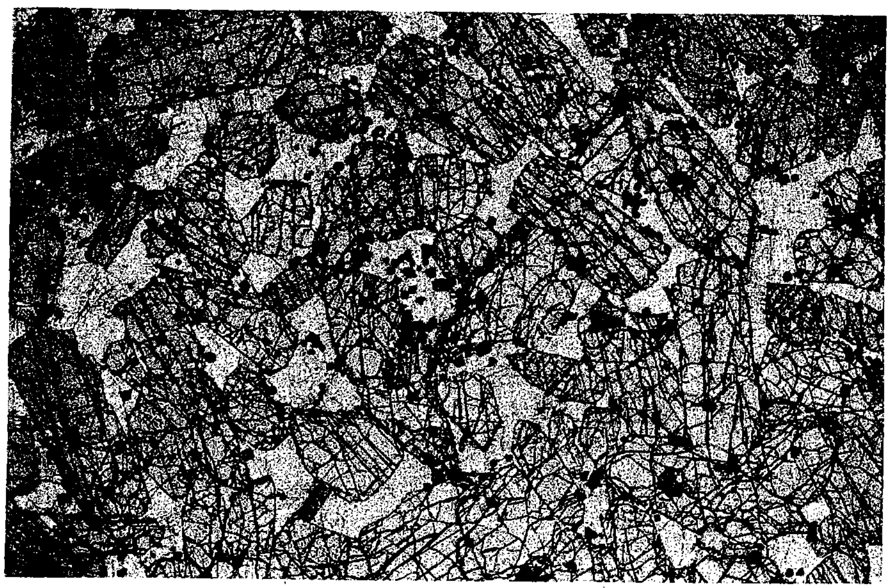

Plate 7. Black specks of finely disseminated metallic ore (Chrome Spinel) occurring in coarse-grained, olivine-rich ultrabasic rock. (PPL x 8.5)

Plate 8. Recrystallized Sandstone (Penrith). Small wind rounded quartz grains cemented together by additional growth of silica from percolating solutions. The quartz jacket is clearly visible. (# x 50)

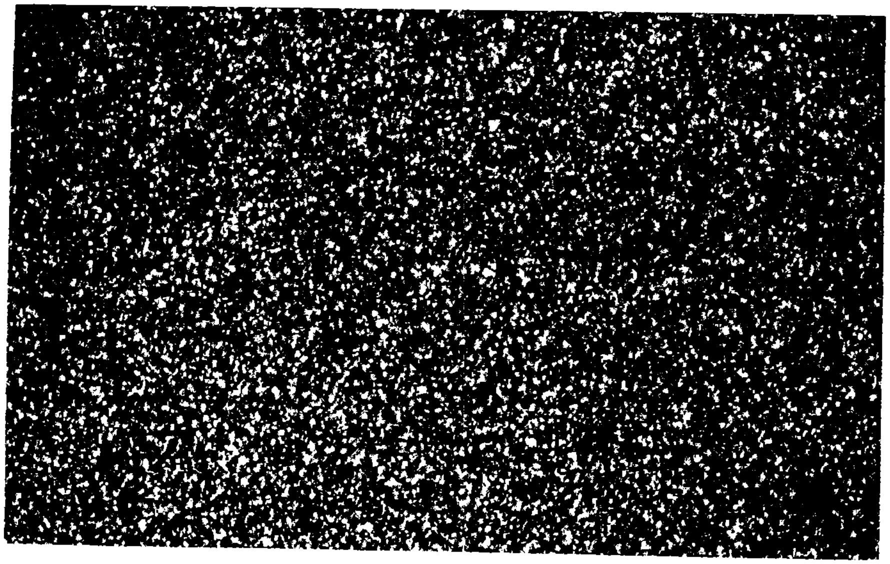

Plate 9. Siltstone (Heys Britannia, Lancs). Very fine grained quartz with occasional felspar and mica crystals interlocked by induration. (PPL x 8.5)

Plate 10. Saccharoidal Limestone (Tomintoul). Recrystallization of calcite due to metamorphism, resulting in granular texture (marble). The cleavage is clearly apparent. (PPL x 8.5)

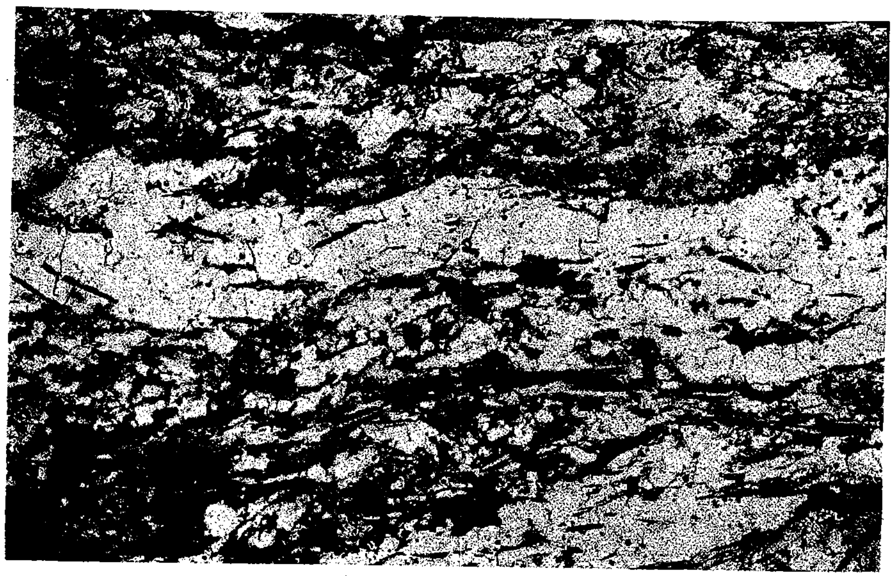

Plate 11. Quartz-mica Schist (Perthshire). Metamorphism has resulted in the banding of large quartz, felspar and mica crystals to produce a distinctly foliated rock. (PPL x 8.5)

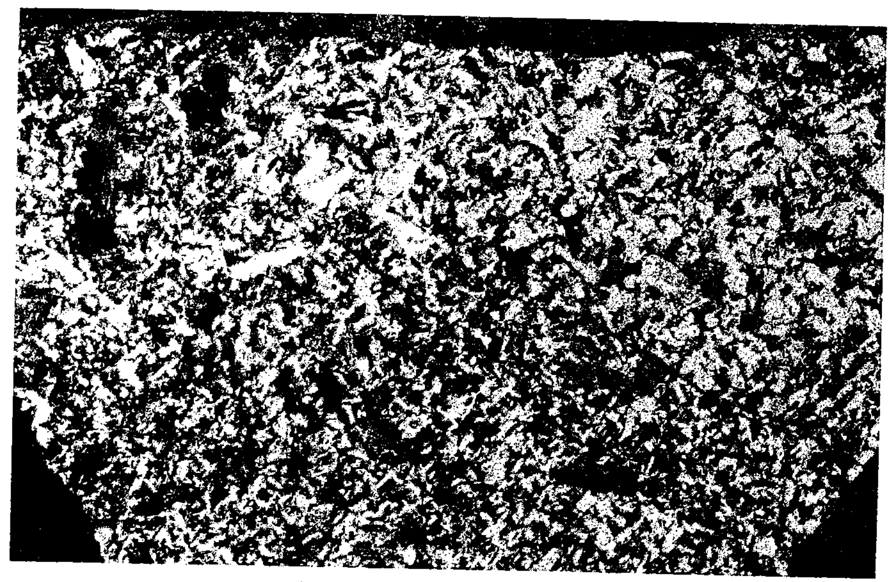

Plate 12. Hornfels (Penlee). A metamorphosed dolerite showing perfect interlocking fine-grained crystals of felspar and pyroxene randomly orientated throughout the specimen. (PPL x 8.5)

Mineral	Hardness (Moh)	Cleavage (Millers Indices)	Alteration products	Remarks and Rocks of which mineral is an important constituent
Quartz	7	-	-	Acid igneious rocks, sand-stones, quartzite, schists gneisses.
Felspars Orthoclase and Plagioclase	6 - 6.5	001 and 010 good, basal perfect	Orthoclase to Kaolinite	Most igneious rocks, arkoses, common in metamorphic rocks.
Muscovite Mica	2.5 - 3	001 perfect	-	Acid igneous, micaceous sandstones, schists and igneous gneisses.
Biotite Mica	2.5 - 3	001 perfect	Chlorite	Acid and intermediate igneous; schists and gneisses.
Amphiboles (Hornblende etc)	4 - 6.5	Good prismatic cleavage	Chlorite	Intermediate and basic igneous. Certain schists and gneisses.
Pyroxenes (Augite etc)	5 - 7	Good prismatic cleavage	Chlorite	Intermediate and basic igneous.
Olivine group	6.5 - 7	010 imperfect	Serpentine	Basic and ultrabasic igneous rocks.
Calcite	3	$10\bar{1}1$ perfect	Readily attacked by acidic solutions.	Limestone and chalk.
Dolomite	3.5 - 4	$10\bar{1}1$ perfect	-	Dolomite and dolomitic limestones.
Kaolinite	2 - 2.5	001 perfect	-	Alteration product.
Chlorite group	2 - 2.5	001 perfect	-	Alteration product.
Zeolites	3.5 - 5.5	fibrous or columnar	-	Alteration and replacement products occuring in igneous, sedimentary and metamorphic rocks.
Chalcedony	6	-	-	Cryptocrystalline quartz. Flints and cherts.
Epidote group	6 - 7	001 perfect	-	Alteration product of plagioclase. Schists and gneisses.
Garnet group	6.5 - 7.5	-	-	Typically metamorphic.
Cordierite	7 - 7.5	010 imperfect, 100 and 001 poor.	Chlorite and muscovite.	Schists, gneisses and hornfels.
Sillimanite	7.5	010 perfect	-	Schists and gneisses.

Table 4. Characteristics of minerals commonly found in roadstones[14].

d) the relative hardness, mode of cleavage, and/or fracture of the minerals;

e) the type and degree of alteration and the proportion of decomposing minerals;

f) the occurrence and distribution of fossil and metallic ore included within the rock; and

g) the occurrence of soluble and insoluble mineral mixtures.

These factors are a direct result of the mode of genesis of the rock and the various environmental states to which it has since been subjected.

The type and proportion of minerals present is dependent, to some extent, upon the composition of the originating material, be it a magma, sediment, or preformed rock material about to undergo metamorphosis. The various genetic processes can produce different rocks from similar chemical ingredients.

The size of the mineral particles and the manner in which they are aggregated within a rock is also affected by the mode of formation. Generally speaking, the mineral grains of an igneous rock interlock during solidification whilst in sedimentary rocks the grains abut against one another during deposition and are subsequently linked by later lithification processes. The type of bond produced by lithification has a profound effect upon the durability characteristics of the resultant rock. High-grade metamorphic rocks generally have interlocking grains whilst certain low-grade metamorphics may still retain the granular characteristics of their original sedimentary parent. It should be noted here that the lower the porosity the less likely is the aggregate to be affected by the disruptive effect of ice and salt crystallization within the voids. Similarly, the vesicular and/or amygdaloidal texture of some volcanic rocks is likely to affect durability.

The importance of the relative orientation of minerals is most pronounced with respect to the metamorphic rocks whilst the effect of relative hardness, cleavage, and/or fracture is reflected in the behaviour of all rock types. The subject of chemical weathering is highly complex and should be considered in two ways; firstly, with respect to the alteration which has taken place up to the time of quarrying; and secondly, as to whether deterioration is likely to occur or continue once the aggregate is in service. The effect of chemical alteration is not always detrimental to the mechanical properties. A small amount of alteration to the minerals within a rock can often improve its resistance to polishing because an assemblage of hard and soft minerals can increase the degree of roughness of the stone surface[13, 14]. If the proportion becomes too high, however, the surface irregularities may become filled with clay minerals and thus decrease the polish resistance. The abrasion resistance decreases progressively with the increase in altered minerals. In the extreme case, the strength may be reduced such that the roadstone disintegrates. As was mentioned previously, Knight and Knight[10] have commented on the fact that alteration of minerals can occur in different ways. Perimeter weathering may reduce the bonding strength between grains to an extent that they are easily plucked out, whilst a small amount of kernel alteration may have but little effect upon the durability.

In the U. K., the chemical weathering of rock-forming-minerals is usually a long-term process and is not, therefore, considered important in relation to the relatively short period that a stone is in service. In other countries, and particularly those subject to tropical and sub-tropical climates, this phenomenon can be a major problem to road engineers who wish to ensure the stability of their roadmaking materials following emplacement[15-20 incl.]. It is evident that basic igneous rocks are those most influenced by chemical weathering and the author is aware of instances in the U. K. where freshly quarried dolerite and teschenite have failed prematurely as a result of deterioration both whilst stockpiled and in service[19,20,21].

Metallic ore and fossil inclusions can have different effects upon the durability depending upon their proportion, size, and distribution within the rock. Their influence is somewhat similar to that produced by a mixture of hard and soft minerals and also to mixtures of soluble and insoluble minerals.

INFLUENCE OF PETROGRAPHY UPON DURABILITY

A number of researchers have attempted to relate the petrographic characteristics of roadstones to their durability, with varying degrees of success[2,7,10,13,19,22-31 incl., 54]. The result of some of this work is summarized in the following discussion in which the different aspects of durability are dealt with in turn.

Strength

Deere and Miller[32] have devised an engineering-classification system for rock material which can be tested in the laboratory (intact rock) based on the two mechanical properties--uniaxial compressive strength, and the tangent modulus of elasticity. Their results are plotted in terms of uniaxial compressive strength against modulus ratio (E/σ) and are related to various petrological groups of rock (fig. 3-5[33].) Few rocks have strength in excess of 32,000 lb/in^2 and it is noticeable that these are almost entirely non-porous rocks possessing an interlocking texture, i.e., quartzites, dolerites, and dense basalts. The rocks of high strength (16,000 - 32,000 lb/in^2) include the majority of the igneous rocks, the stronger metamorphics, well-cemented sandstones and indurated shales, and about 40 percent of the limestones and dolomites tested. The medium-strength range includes poorly bonded porous rocks and foliated rocks. Rocks in the low- and very-low-strength ranges are generally very porous, weakly bonded sedimentary rocks, highly foliated igneous and metamorphics, or chemically altered rocks of any lithology.

Thus, the main factors causing a reduction in the strength of igneous rocks are: an increase in porosity, grain size, the proportion of soft minerals, and the development of foliation due to flow structure. Chemical alteration generally increases the proportion of soft minerals and perimeter alteration may result in dislocation of the intergranular bonding. Vesicular texture generally decreases the strength, whilst the effect of amygdales depends upon the nature of the occupying mineral.

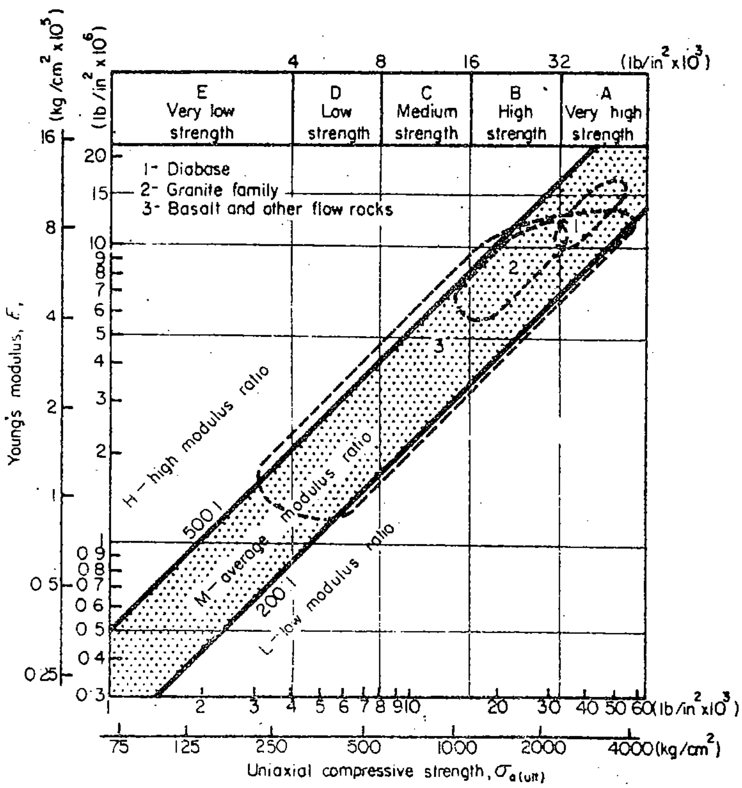

Figure 3. Engineering classification for intact rock—summary plot for igneous rocks (176 specimens, 75% of points) (33)
E_t = tangent modulus at 50% ultimate strength.
Classify rock as AM, BH, BL, etc.

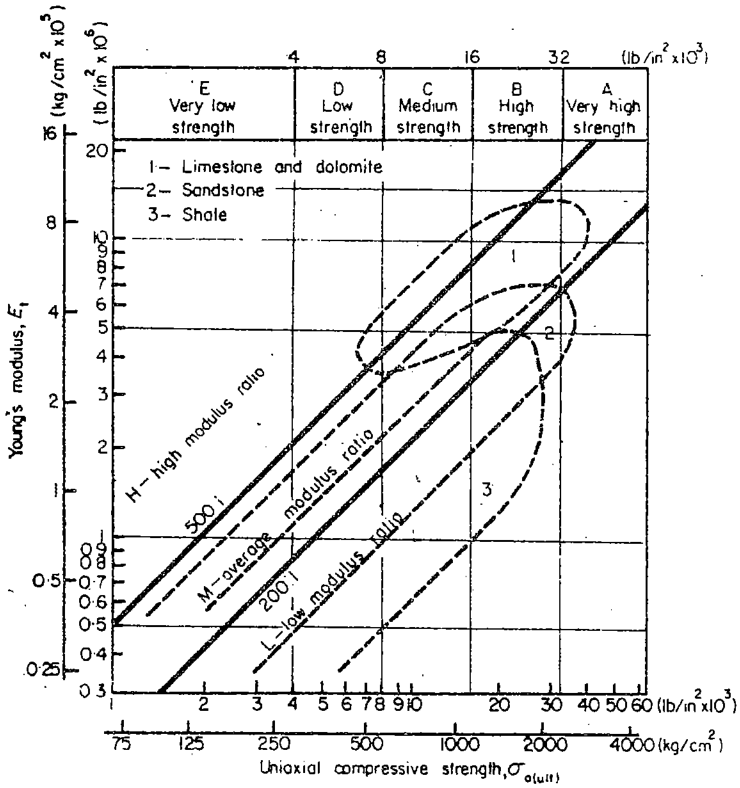

Figure 4. Engineering classification for intact rock—summary plot for sedimentary rocks (193 specimens. 75% of points) (33)
E_t = tangent modulus at 50% ultimate strength.
Classify rock as AM, BH, BL, etc.

With regard to sedimentary rocks, the interlocking fabric and mineralogy of limestones and dolomites appears to be responsible for their relatively high strength range (and modular ratio). The main factors influencing the strength of sandstones and shales are the intergranular bonding and porosity. Chemical alteration effects are the same as those occurring in similar igneous rocks. Reversion, as occurs in non-indurated shales, is not important in relation to road surfacings because such material would not normally be used unless stabilized. (The anisotropy caused by sedimentation causes variation in the modular ratio according to whether the load axis is placed perpendicular to, or across the plane of the laminations.)

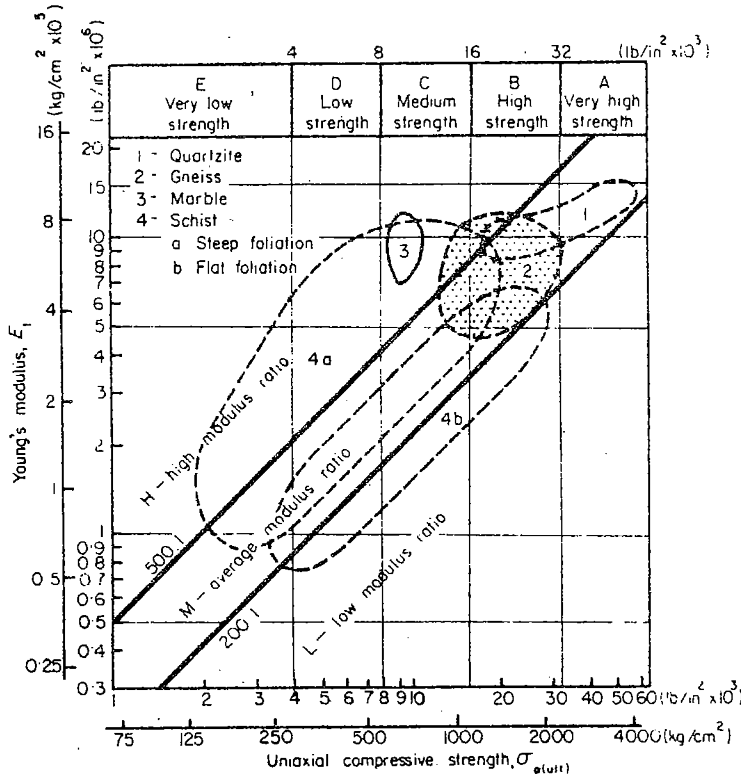

Figure 5. Engineering classification for intact rock—summary plot for metamorphic rocks (167 specimens, 75% of points) (33)
E_t = tangent modulus at 50% ultimate strength.
Classify rock as AM, BH, BL, etc.

The principal factors causing a reduction in the strength of metamorphic rocks are increase in grain size, porosity, proportion of soft minerals and, in particular, an increase in schistosity. The extent of the intergranular bond is also highly important. The effect of chemical alteration is similar to that noted for igneous and sedimentary rocks. (Note: In order to reduce the anisotropic effects previously indicated, fundamental strength tests are not now used in the U. K. as a method of assessing the potential of aggregates. Arbitrary specimens using a number of pieces of aggregate are used instead. The petrographical characteristics listed nevertheless control the behaviour of the individual aggregate particles.)

Stripping

Although some materials are more liable to stripping than others, the author believes that this phenomenon occurs more often from faulty preparation and laying techniques than from the use of inadequate stone. However, porosity, degree of alteration, and liability to chemical decomposition are important petrological characteristics. Vesicular textures are generally advantageous.

Abrasion

The term "abrasion" is used here in the sense of "wear of the surface by attrition." The polishing of a roadstone also involves wear of the surface but this aspect will be dealt with in the following section.

Petrographical characteristics of importance are the proportion of hard minerals and the degree of their hardness, the preponderance of cleaved minerals and the degree of alignment of these minerals, the grain size, the nature of the intergranular bond, and the degree of, or liability to, chemical alteration of the mineral content.

Of the fresh igneous rocks, those containing a high free-silica content tend to resist abrasion better than the basic rocks which have a high ferromagnesian content, the quartz being harder and without cleavage. Chemical decomposition, particularly a high degree of perimeter alteration (as this can destroy the intergranular bond and allow the harder cores to be plucked from the rock surface) results in increased abrasion. Vesicular texture generally reduces the resistance to abrasion whilst the nature of the mineral occupant determines the effect of amygdales. Among the sedimentary rocks, limestones and dolomites are subject to rapid wear because of the softness and cleavage facility of their relatively monomineralic composition. The abrasion resistance of quartzitic clastic rocks is almost entirely dependent upon the nature of the intergranular bond. Thus flint is highly resistant, whilst poorly cemented sandstones and gritstones are soon abraded because of plucking of the grains. Mixed-mineral sedimentary rocks, such as greywackes, tend to have poor abrasion resistance.

Of the metamorphic rocks used in road surfacing, gneisses tend to behave similarly to igneous rocks of the same mineralogy, whilst hornfels and quartzites have a high abrasion resistance because of their hard mineral content and dense interlocking texture.

Polishing

As was mentioned in the previous section, polishing may be considered as a special form of wear. Skid-resistance is provided by roughness of the road surface and a skid-resistant surface is one which is able to retain a high degree of roughness whilst in service. It should be noted, however, that the influence of surface texture upon skidding varies with the vehicular speed. At slow speeds (30 MPH) micro-texture predominates, whilst at high speeds the macro-texture becomes the major factor[34,35]. Micro-texture is mainly controlled by the

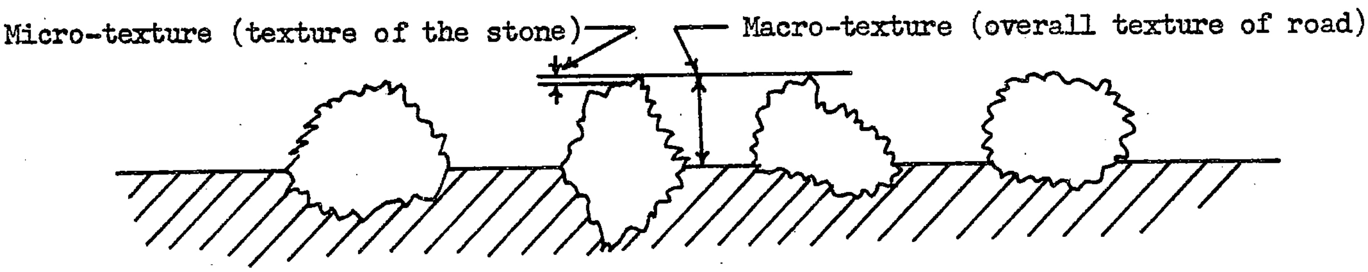

Figure 6. Micro- and macro-texture of a road surface[2].

roadstone itself whereas macro-texture is largely dependent upon the aggregate-binder relationship (fig. 6[2]). For the purposes of this paper, only micro-texture will be discussed.

Igneous rocks containing a small proportion of soft minerals tend to have reasonably high polish resistance as do those containing fractured grains of reasonably large size. Grain size seems to have but little effect although there is a small increase in polish resistance with larger grains. Fine-grained, fresh igneous rocks (i.e., those with minerals of comparable hardness) tend to accept a high polish. Foliation of the fine-grained groundmass of basalts increases the polishing resistance but the existence of an ophitic texture appears unimportant unless there is some alteration of the groundmass. Secondary minerals only increase the resistance to polishing if their hardness differs from the bulk mineral content. Similarly, metallic ore only increases the polish resistance if it is finely disseminated throughout the rock. A well-dispersed system of vesicles generally increases the resistance to polishing. The effect of amygdales depends upon their size and distribution throughout the stone, the relative hardnesses of the secondary occupant, and the enclosing original minerals.

Among the sedimentary rocks, clastic gritstones of variable mineral content and certain sedimentary quartzites tend to be polish resistant. The plucking of the grains from the matrix results in a continual renewal of the rough surface. In contrast, other monomineralic, well-bonded orthoquartzites and cryptocrystalline flints take a high degree of polish. Monomineralic limestones and those containing a small amount of insoluble residue tend to have a low resistance to polishing. The resistance to polishing increases with an increase in the amount and size of the insoluble residue, especially if the material is quartz. Similarly, the presence of fossil organisms and patches of medium- and coarse-grained carbonate increases resistance if they differ in hardness or solubility from the matrix material.

Of the metamorphic rocks, only gneisses, hornfels, and metaquartzites are considered as possible road surfacings. Gneisses behave in a similar manner to the equivalent igneous rock whilst the hornfels and quartzites tend to take a high polish because they generally have a fine-grained, interlocking texture and are monomineralic or composed of relatively hard minerals (H 5).

ASSESSMENT OF AGGREGATE DURABILITY

The main laboratory tests used in the U. K. to determine surfacing aggregate durability are as follows:

a) specific gravity and water absorption[36];

b) sodium sulphate soundness test[37,38,19];

c) those determining mechanical properties such as:
   1) aggregate crushing value (ACV) 10 percent fines values[36];
   2) crushing value[36];
   3) aggregate impact value (AIV)[36];

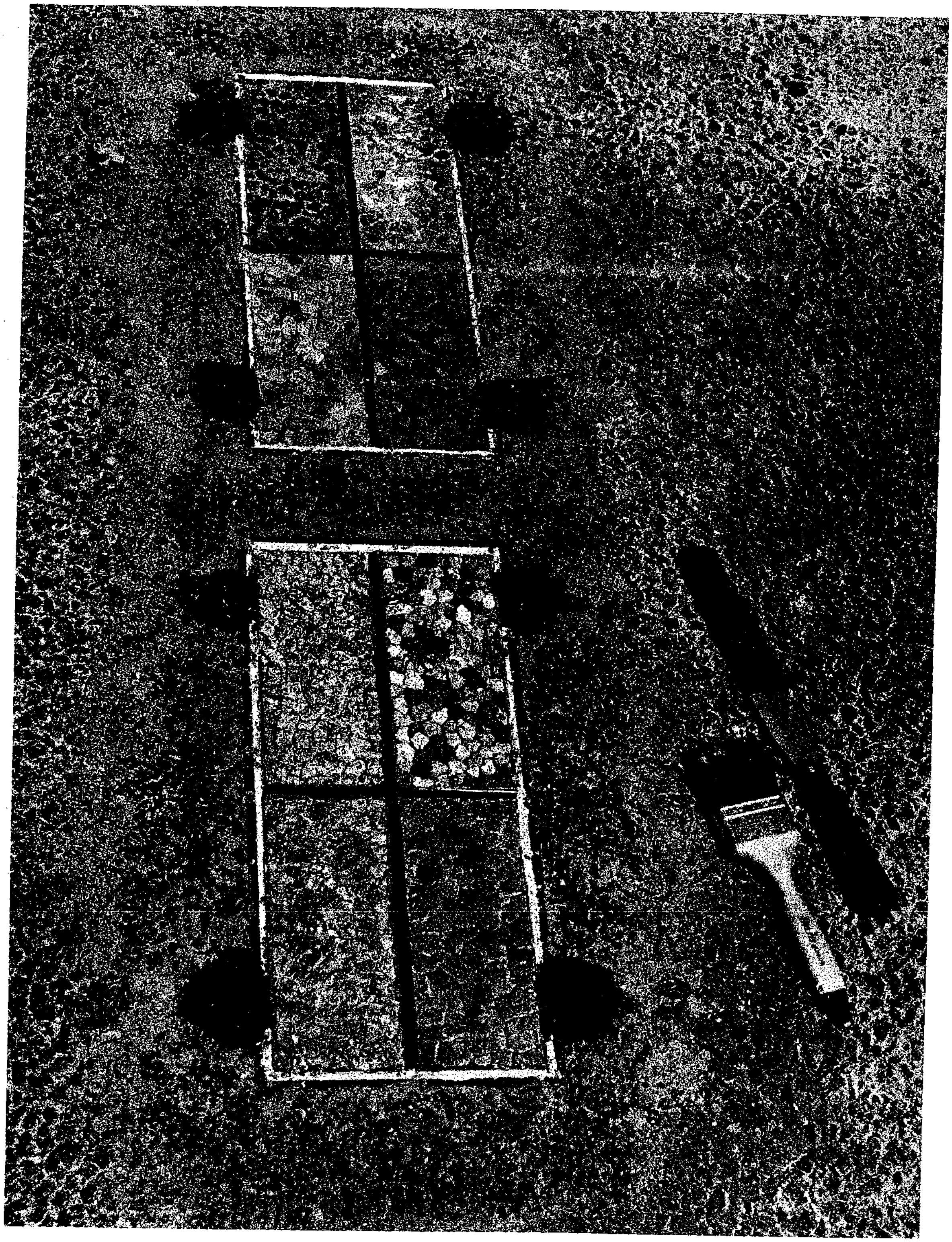

Plate 13. Small-scale road experiment using panels of chippings set in an existing road surface[2].

4) aggregate abrasion value (AAV)[36]; and
5) polished-stone value (PSV)[36].

In addition, the Los Angeles abrasion/attrition test[39] which is in common use in other countries, is sometimes specified. Table 5 lists the results of a number of mechanical-strength tests[40] and indicates the wide range of results for a particular test obtainable with any trade group.*

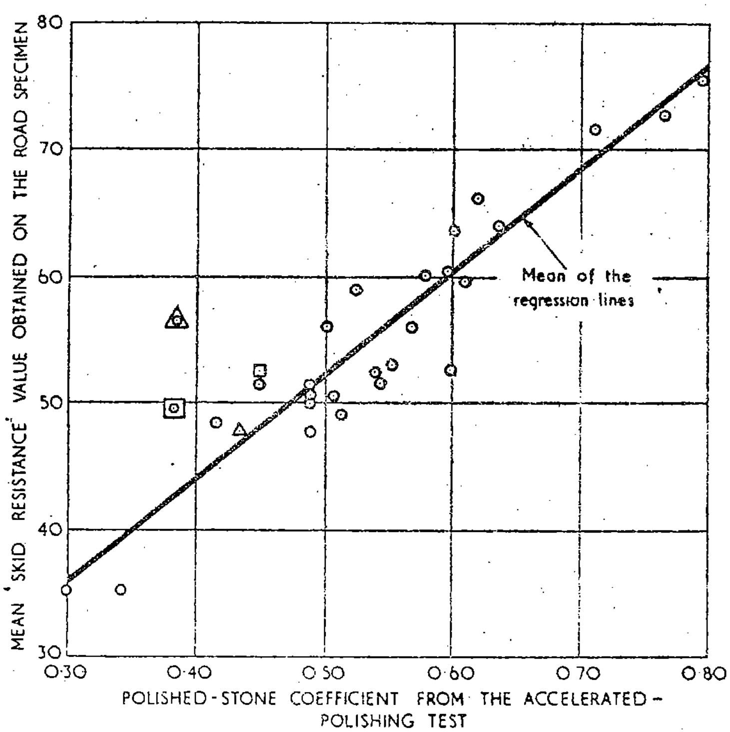

Figure 7. Relation between PSC and skid-resistance values obtained from small-scale road experiments(11).

The degree of this variation is perhaps better shown by using a distribution plot for one type of test only. Thus Figure 8[41] shows the distribution of PSV test results for a range of stones which have been plotted in terms of their Trade Group Classification. From this it is quite evident that some stones placed in the same group do not possess similar polishing characteristics. Further, it is found that stones which possess similar results for one test yield quite different results when subjected to another test[40]. For these reasons such a classification system can be misleading. The author considers that the present system should be discontinued and that it should be replaced with a simple classification based upon well-defined petrological terms rather than hypothetical mechanical properties. A system such as that shown

*Note. The polishing characteristic is given in terms of polished-stone coefficient (PSC). This measure became obsolete in 1965 when B.S. 812 was revised. The equivalent PSV's are approximately 10 percent lower than PSC's.

Group classification B.S. 812 : 1960		Aggregate crushing value*	Aggregate impact value*	Aggregate abrasion value*	Water absorption* (per cent)	Specific gravity	Ten per cent fines aggregate crushing value	Polished -stone coefficient
Artificial	Mean	28	27	8.3	0.7	2.68	–	0.59
	Range	(15-39)	(17-33)	(3-15)	(0·2-1·8)	(2·8-2·6)		(0·35-0·74)
	No. of samples	55	21	18	18	18		33
Basalt	Mean	14	15	6·1	1·1	2·80	–	0·62
	Range	(7-25)	(7-25)	(2-12)	(0·0-2·3)	(3·0-2·6)		(0·45-0·81)
	No. of samples	123	79	65	68	68		70
Flint	Mean	18	23	1·1	1·0	2·54	–	0·39
	Range	(7-25)	(19-27)	(1-2)	(0·3-2·4)	(2·6-2·4)		(0·30-0·53)
	No. of samples	63	32	45	24	24		7
Granite	Mean	20	19	4·8	0·4	2·69	–	0·59
	Range	(9-35)	(9-35)	(3-9)	(0·2-0·9)	(3·0-2·6)		(0·40-0·70)
	No. of samples	41	32	28	16	16		23
Gritstone	Mean	17	19	7·0	0·6	2·69	–	0·72
	Range	(7-29)	(9-35)	(2-16)	(0·1-1·6)	(2·9-2·6)		(0·60-0·82)
	No. of samples	81	45	31	33	33		32
Hornfels	Mean	13	12	2·2	0·4	2·82	–	0·45
	Range	(5-15)	(9-17)	(1-4)	(0·2-0·8)	(3·0-2·7)		(0·40-0·50)
	No. of samples	28	24	13	15	15		4
Limestone	Mean	24	23	13·7	1·0	2·66	–	0·43
	Range	(11-37)	(17-33)	(7-26)	(0·2-2·9)	(2·8-2·5)		(0·30-0·75)
	No. of samples	164	61	34	42	42		51
Porphyry	Mean	14	14	3·7	0·6	2·73	–	0·56
	Range	(9-29)	(9-23)	(2-9)	(0·4-1·1)	(2·9-2·6)		(0·43-0·71)
	No. of samples	62	29	23	30	30		23
Quartzite	Mean	16	21	3·0	0·7	2·62	–	0·58
	Range	(9-25)	(11-33)	(2-6)	(0·3-1·3)	(2·7-2·6)		(0·45-0·67)
	No. of samples	57	37	29	21	21		20
All groups†	Mean	19	19	5·7	0·7	2·68	17	0·58
	Range	(5-39)	(7-35)	(1-26)	(0·0-3·7)	(3·0-2·3)	(1-37)	(0·30-0·83)
	No. of samples	724	370	311	312	312	69	292

*In these tests, a numerically lower result indicates a higher resistance in the test
†Including results from unclassified samples

Table 5
Summary of means and range of values for roadstone tests in each rock-group (40)

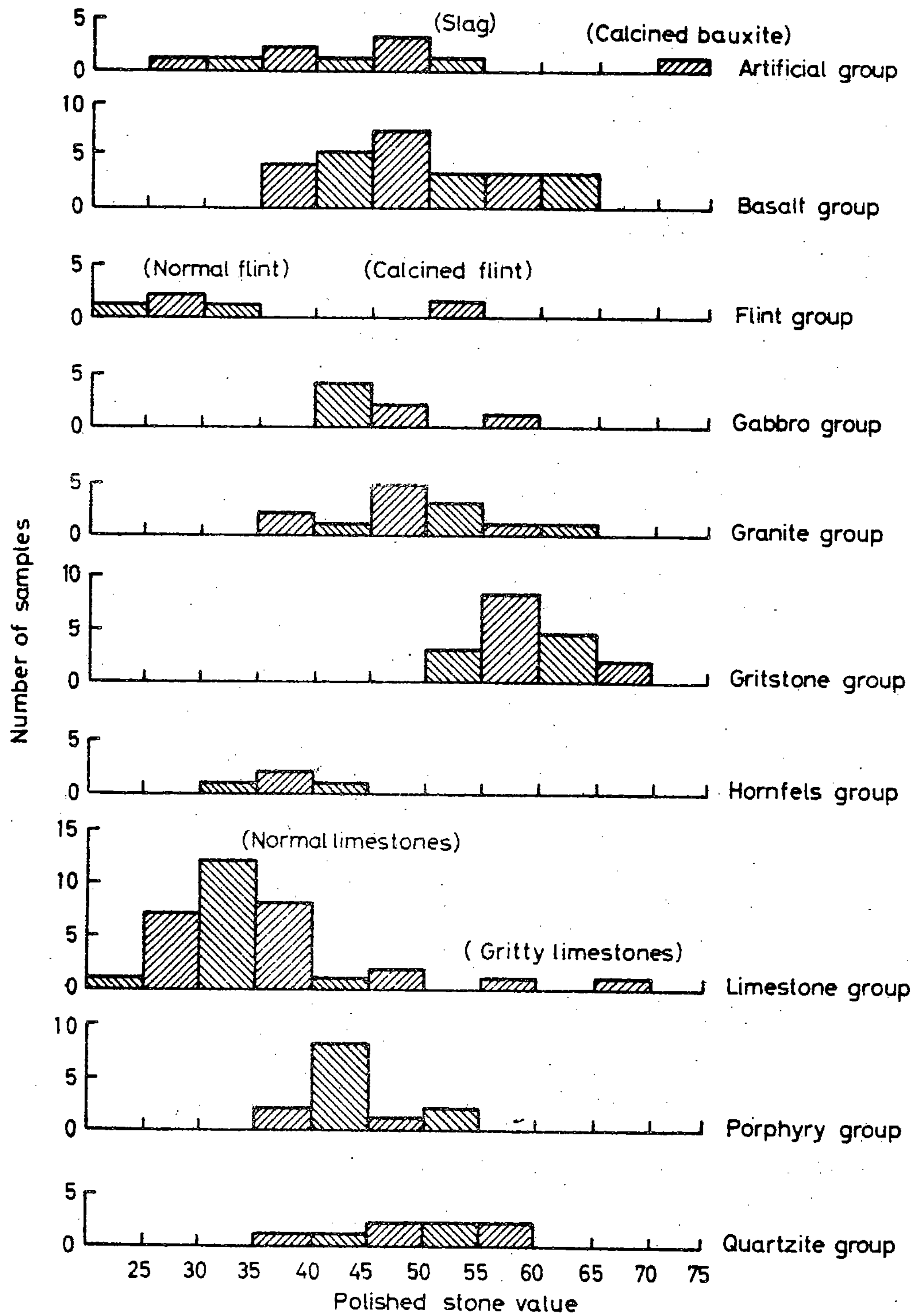

Fig.8 DISTRIBUTION OF POLISHED STONE VALUES IN DIFFERENT GROUPS OF ROCK (41)

Note: this table is largely based on Fig. 14 of Technical Paper 43 published in 1958. Since 1965, however, the method of determining the P.S.V. has been modified and values obtained by the new method are 10 per cent lower than formerly.

in Table 6, which is used by Lees (discussion to Ref. 14) would be appropriate.

Maclean and Shergold[11] were of the opinion that the accelerated-polishing test yielded a degree of polishing comparable to that which roadstones would attain in service. The results of their work using a small-scale road experiment is shown in Figure 7. Hosking[2] has extended this work by using actual road surfaces and small-scale experimental panels on a number of sites within the U. K. (Plate 13). Whilst his results generally confirm the good correlation between the PSV of a roadstone and the skid-resistance of surfaces consisting largely of that stone, they do indicate consistent departures by certain rock types. In particular, the quartzites which were tested had higher skid resistances than were anticipated from their PSV's and the igneous rocks yielded lower values than were expected. Full-scale experiments have been carried out to find a relation between texture depth and AAV[21,44,45,46]. The results were then used to establish minimum allowable values of AAV. However, in the author's experience, there are similar anomalies in the wearing performances of certain types of gritstones from those forecast from British aggregate abrasion tests. Hosking believes that the PSV test does not simulate actual traffic polishing conditions as closely as is desirable and suggests that test specimens should be subjected to a more severe polishing procedure. The author agrees with Hosking's conclusion but would draw attention to one factor involving petrography which appears to have been overlooked. Both the PSV and AAV tests require the use of abrasives and both tests employ materials (emery and silica sand respectively) of standard grading. Using indurated gritstones and siltstones of similar mineralogy and intergranular bonding (as far as this can be ascertained), the author has found that the mean grain size of the specimens under test appears to affect the aggregate abrasion values obtained. Because at least a part of the abrading material is derived from the aggregate itself, the mineralogy and grain size of that abraded material must necessarily influence the degree of attrition and polishing which takes place. The Los Angeles test,

IGNEOUS

(Ic) Coarse
- Granite
- Syenite
- Diorite
- Gabbro

(Im) Medium
- Micrognanite
- Microsyenite
- Microdiorite
- Dolerite

(If) Fine
- Rhyolite
- Trachyte
- Andesite
- Basalt

SEDIMENTARY

(Sc) Coarse
- Conglomerate
- Breccia
- Gravel
- Scree

(Sm) Medium
- Sandstone
- Sand

(Sf) Fine
- Shale
- Mudstone
- Clay

(Sca) Calcareous
- Limestone
- Dolomite

METAMORPHIC

(Mc) Coarse
- Gneiss

(Mm) Medium
- Quartzite

(Mf) Fine
- Schist
- Slate
- Hornfels

ARTIFICIAL STONES

(A_v) Various
- Slag
- Calcined bauxite
- Calcined flint
- Synopal
- Pulverised Fuel Ash

TABLE 6

Suggested Classification of "Roadstones" according to Petrological Characters. (after Lees)[14] because it uses self-abraded material, is in this respect superior to the British test.

PRESENT AND FUTURE RESEARCH IN THE U. K.

In 1968 the author[14] suggested a number of lines of research leading to a fuller understanding of the mechanism of roadstone polishing. Much of the work suggested is equally applicable to understanding durability of stone in general.

At the present time, research is continuing at the British Road Research Laboratory[19,51,52,etc.], the Cement and Concrete Association[42], and at Birmingham University[31,42] into the performance of roadstones under simulated and actual traffic conditions. A particular aspect of this work concerns the interaction of tyres and the road surface. At Salford University, the research is biased toward the basic mechanism involved in the wear of minerals. Attempts are being made to determine the influence of individual mineral properties such as hardness, cleavage, and crystallographic orientation upon the attrition and polishing of mineral surfaces. It is also hoped that the research can be extended to cover the nature of different abrasive materials and their mode of action. It should be noted here that the development of the stereoscan electron microscope has provided an invaluable tool for the examination of roadstones[31] (and indeed other materials[43]) (Plate 14).

A search for new sources of high-PSV stone is at present being carried out by the Road Research Laboratory in collaboration with the Institute of Geological Sciences[52].

DEVELOPMENT OF HIGH QUALITY SYNTHETIC AGGREGATES

It has become evident in recent years that there are certain road locations which require a higher degree of skid-resistance than can be provided by natural roadstones. The reasons may be twofold; firstly, these sites are skid-prone, and secondly, urban conditions can make regular maintenance closures uneconomic[47].

There are few sources of natural stone having PSV's in excess of 62 in the U. K.[46], the highest consistent recorded value

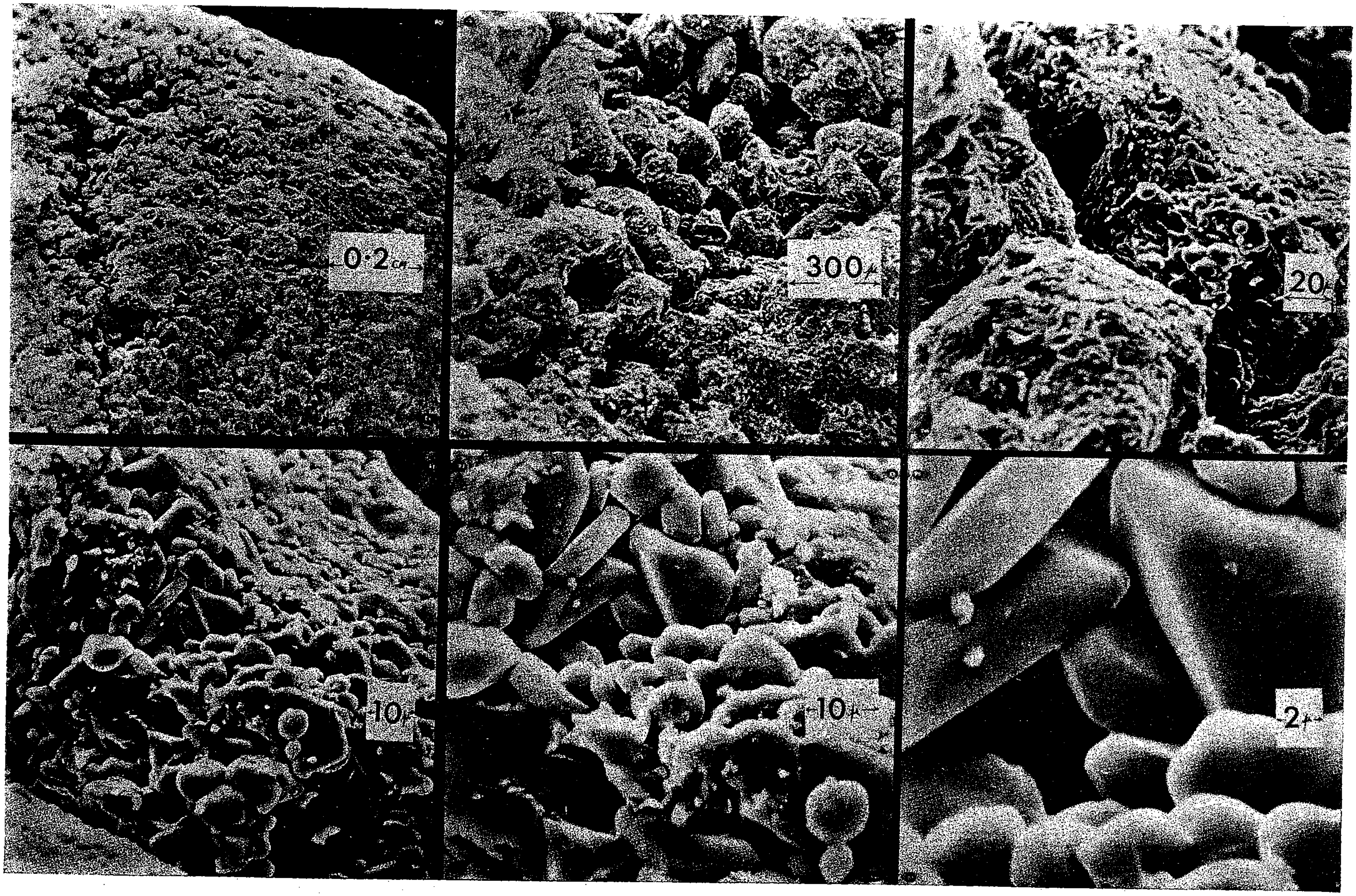

Plate 14. Topography of sample of calcined bauxite (after Williams and Lees)[31].

being 72. However, the abrasion resistance of this stone is somewhat suspect. For this reason synthetic aggregates have been, and are being, developed with a view to producing a polish-resistant material which is also highly durable.

Undoubtedly the best material used so far is calcined bauxite which is laid in conjunction with an epoxy-resin binder or epoxy-resin extended bitumen (Shellgrip). Hosking(46), and Hatherley and others(48), have described the historical development and use of these materials. The success of these materials is due to the semi-rigid nature of the binder which holds the aggregate firmly in position, thus ensuring a good texture depth, and the high durability of the aggregate. The high temperature of calcination of the bauxite (about 1600°C) results in a welded vesicular structure(31) composed mainly of the angular crystals, corundum and mullite (which is of relatively rare natural occurrence) (Plate 14). A consideration of the characteristics of these minerals indicates why this assemblage results in such high durability (Tables 7 and 8). The poor aggregate crushing and impact values of the bauxite are to some extent offset by the use of small chippings (1.2 - 2.8 mm) firmly set in the non-thermoplastic binder. Though very successful, the high cost of

	CORUNDUM	MULLITE
Hardness	9	6 - 7
Cleavage (Miller Indices)	None but partings on $\{0001\}$ and $\{10\bar{1}1\}$	$\{010\}$ distinct
Alteration products	None	None

Table 7. Mineralogical characteristics of corundum and mullite.

laying calcined bauxite-resin surface dressings has limited their use to exceptionally accident-prone locations. For this reason a number of commercial, university, and government research laboratories in the U. K. and elsewhere are endeavouring to develop other synthetic materials of high durability[19,46,49,50]. The research involves using materials such as devitrified glasses, metallurgical slags, treated power-station waste (PFA)[53], and various grits bonded in matrixes such as blast-furnace slag, fused rock, and ceramics. The author is conducting research at Salford into the use of grits such as calcined bauxite fused in an engineering brick matrix[50]. The work is not far advanced but Table 8 gives some indication of the results obtained thus far.

	A.C.V.	A.I.V.	A.A.V.	P.S.V.
Calcined Bauxite[19,21]	33	31	3	75
Accrington Red Bricks[19]	34	26	5	55
Accrington Brick/Calc. Bauxite[50]	—	30	6	70
Natural Aggregate[19,40] (best value)	5 (Hornfels)	7 (Basalt)	1 (Flint and Hornfels)	72 (Gritstone and Siltstone)

Table 8. Mechanical characteristics of various aggregated materials.

CONCLUSIONS

The probable future shortage of high-quality aggregates combined with the high cost of accidents (assessed as about £1000, i.e. about $2,400, per personal injury accident - notwithstanding the human suffering involved) renders the selection and use of surfacing materials a most important factor in highway construction. The author believes that monies spent on carrying out research to further our knowledge in this field is likely to prove a truly worthwhile investment.

ACKNOWLEDGEMENTS

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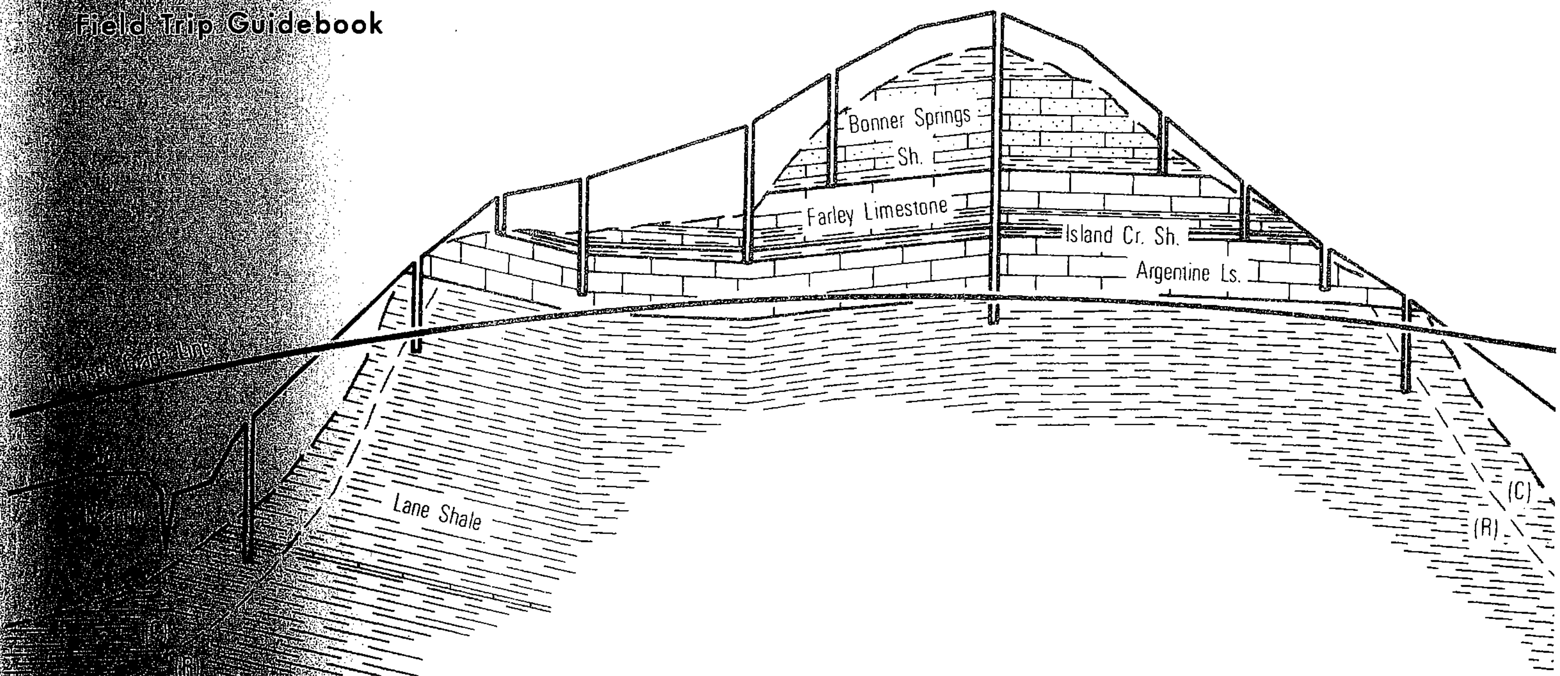

GEO ENGINEERING IN NORTHEASTERN KANSAS

GREATER KANSAS CITY AREA

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21ST ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

FIELD TRIP

APRIL 23, 1970

GEO-ENGINEERING IN NORTHEASTERN KANSAS

Field Trip Leaders
Geologists From The
Sponsoring Organizations

Authors: Virgil A. Burgat
Lowell W. Fowler
Walter F. Fredericksen
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Alvis H. Stallard

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FOREWORD

On behalf of the National Steering Committee, the sponsors of this Twenty-first Highway Geology Symposium bid you welcome to Kansas.

On the first day of this Symposium, you are invited on a field trip that covers the route shown in this Guide Book. The field trip does not exhibit any special category of Highway Geology problems, rather, it covers problems of a general nature found within a field trip area that is limited because of time and distance.

Observations dealing with a variety of highway design and construction problems will be made along the route. There are certain problems described within this sedimentary rock environment that deal with alternating beds of limestone, shale and sandstone. There are also other problems dealing with glacial outwash sediments and those dealing with environmental geology.

Highway Geology attention will be focused on cut slope stability, classification of excavation, subgrade and embankment stability, bridge foundations and other related highway problems.

The field trip covers new and older sections of highway construction in order to compare current design standards with former methods of design.

This year's Symposium is conducted jointly by the State Highway Commission and the State Geological Survey of Kansas. We express our appreciation to each one having a part in its preparation.

We hope that you will find the Symposium stimulating and that your knowledge in Highway Geology is broadened through your participation.

Virgil A. Burgat, Local Chairman
1970 Highway Geology Symposium
Lawrence, Kansas
April 23, 1970

Guide Map For Field Trip

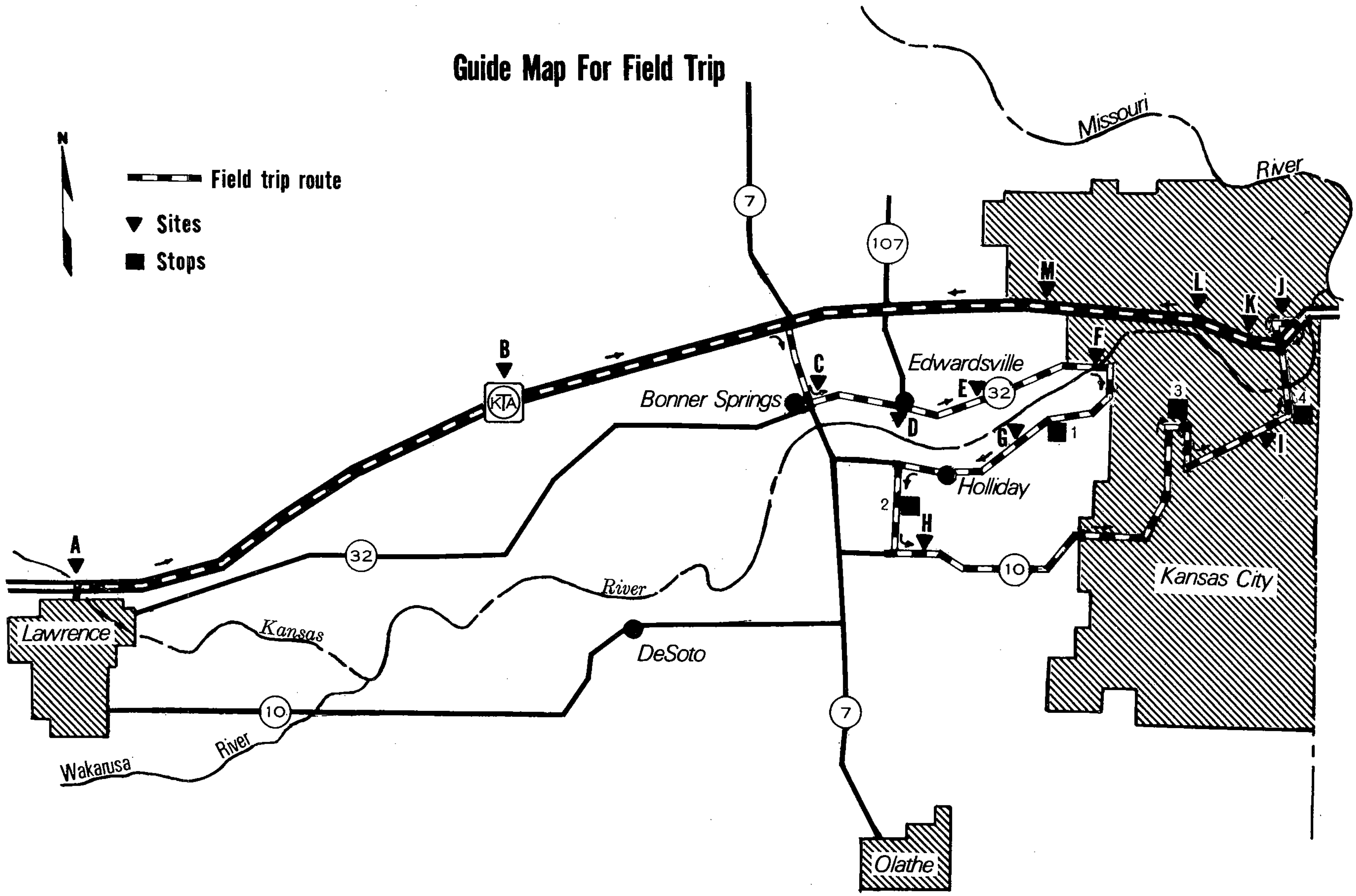
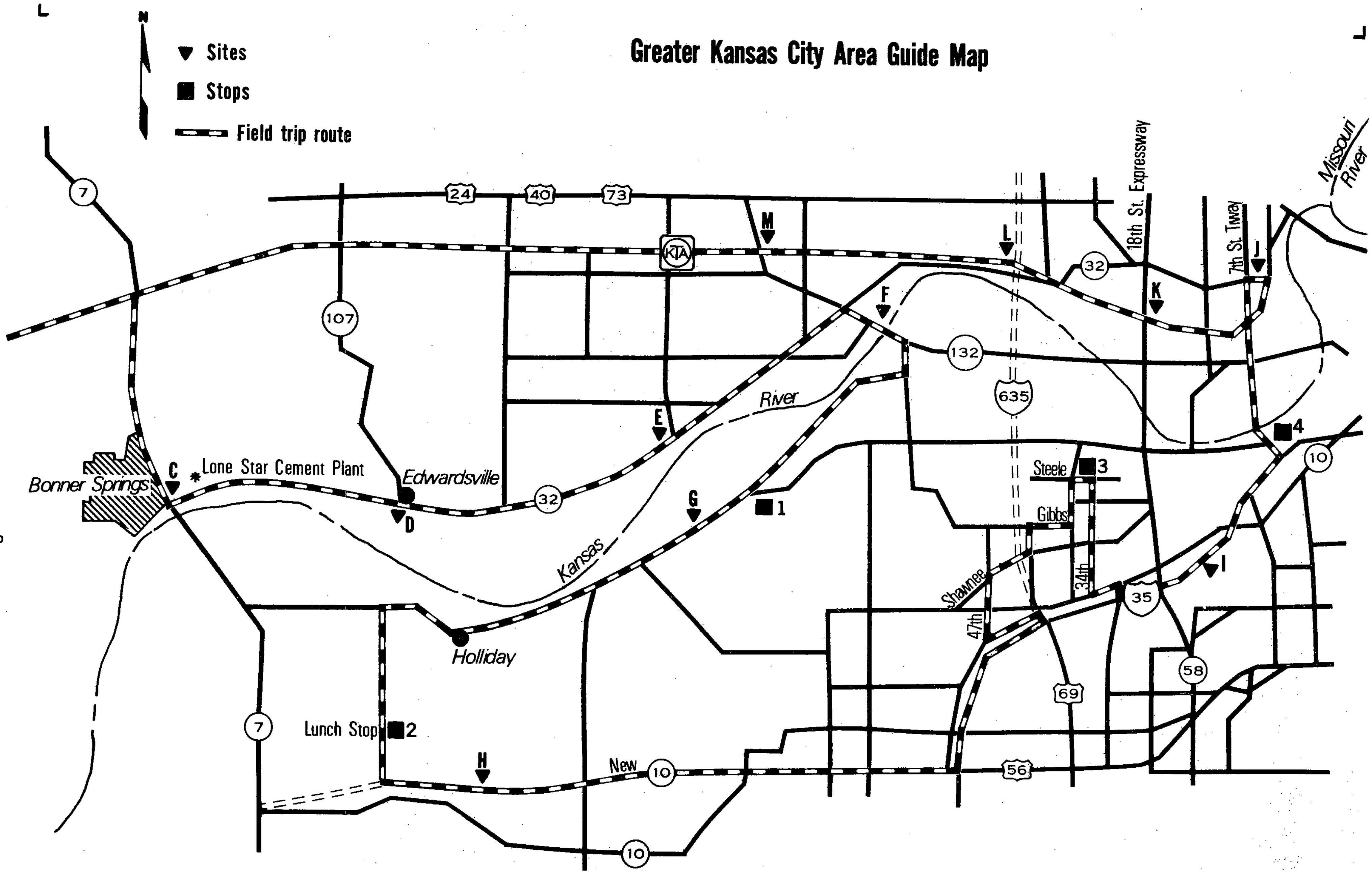


Figure 1



Greater Kansas City Area Guide Map
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Figure 2

INTRODUCTION

HISTORY OF FIELD TRIP AREA

The area through which you will travel consists of Douglas, Wyandotte and Johnson Counties. The story of this area and surrounding counties is unique in the pages of Kansas and United States history. Its early development was greatly influenced by the presence of the Delaware, Shawnee, and Wyandot Indians before 1854 and subsequently by the turmoil that preluded and accompanied the Civil War.

In 1828 Shawnee Indians were removed to eastern Kansas from the vicinity of Cape Girardeau, Missouri. In 1832 the remaining members of the tribe were removed from Ohio. The original treaty gave the tribe 1,600,000 acres of land on the south side of the Kansas River which included what is now Johnson and Douglas Counties.

In 1829 the Delaware Indians were removed from Ohio and settled on the north side of the Kansas River in what is now Wyandotte County.

Both tribes had a knowledge of agriculture and many had habits of industry. They opened farms, built houses and cut roads. Missionaries were very active in both tribes. The first Delaware Baptist Mission was established in 1832 near what is now Edwardsville (near site D on guide map). The Shawnee Methodist Mission was established in 1830 and buildings were constructed in 1839, 1841, and 1845. (These buildings still stand in northeast Johnson County, three miles south of Stop 4 on guide map.)

During the occupancy of these counties by the Indians, few white men became residents. Some of the first were the Choteau Brothers, Frenchmen, who built trading houses among the Shawnee and Delaware. In 1820, four buildings were constructed within the present city limits of the City of Bonner Springs (site C on guide map). Moses Grinter operated the first ferry on the Kansas River. He lived in a cabin near the river until 1857 when he built a house on the north side of the valley. (This building stands today and can be seen at site E on the guide map.)

In 1843 the Wyandot Indians, descendents of the great Iroquois family, migrated from the upper Sandusky in Ohio to what is now Wyandotte County. They purchased 23,040 acres of land near the confluence of the Kansas and Missouri Rivers from the

Delawares. These people were very industrious and intelligent and by treaty in January, 1855, the majority of the Wyandots received rights of citizenship. (The Huron Cemetery in which Wyandot Indians were buried beginning in 1844, is near downtown Kansas City, Kansas less than a mile north of site J on guide map.)

During this era, the lure of trade with settlements of the Spanish Southwest was strong and the Santa Fe Trail, which traverses southern Johnson County, was federally surveyed and used. The Oregon-California Trail, which separated from the Santa Fe Trail near Gardner, Kansas, in Johnson County, was used extensively by western home and gold seekers during the 1840's and 1850's.

On May 15, 1854, the Shawnee Indian Reservation was decreased in size to 200,000 acres. This event along with the signing of the Kansas-Nebraska bill by President Pierce, opened the area to settlers. The bill provided for popular sovereignty, the right of the new residents to choose whether the territory would be free or slave. The unsettled situation created the name "bleeding Kansas" and the struggle over slavery in Kansas was the prelude to the Civil War. Into this hotbed of political dissension came people like John Brown, militant abolitionist, and the notorious Confederate, William Quantrill.

Many townsites were laid out but none were more important than Lecompton and Lawrence. Both were located in what is now Douglas County. The Lecompton townsite was located approximately ten miles northwest of Lawrence and consisted of 600 acres. It was surveyed in 1855 and was the headquarters of the pro-slavery party. It was the design and expectation to make it not only the capital of the territory and future state, but to make it a large city as well. A territorial capital building was started in 1856 but never finished. Another building was constructed on the foundation in the early 1880's for use by Lane University which opened in 1865 and closed in 1903. Dwight D. Eisenhower's parents met as students at Lane University and were married in 1885 in Lecompton.

As early as July, 1854, agents of the new England Emigrant Aid Society visited Kansas to select a site for a settlement. The present site of Lawrence was chosen and during the same month the "pioneer party" of Eastern emigrants left Massachusetts for Kansas. The new town was laid out and on October 6, 1854, christened "Lawrence City" to honor Amos A. Lawrence who supplied funds for the initial settlers. The "hill" which is the present site of the University of Kansas, was called Mount Oread in memory of Mount Oread Seminary at Worcester, Massachusetts.

Lawrence was the "Free-State" headquarters and its early years were turbulent. The question of freedom or slavery was fought with more than moral arguments. Still the city prospered and by 1860 had a population of 2,500. Its prosperity was dealt a devastating blow on August 21, 1863, when Quantrill burned the city and killed 143 people.

An institution of higher learning was opened in Lawrence on April 11, 1859, under the auspices of the Presbyterian Church. It was called "The University of Lawrence." The first building was known as North College. This name is used today to designate the dormitory on the north end of the Kansas University campus overlooking the City of Lawrence. On January 9, 1861, a new charter was granted by the Legislature to the University and the name was changed to "Lawrence University of Kansas."

On January 29, 1861, the Free-Staters gained an impressive majority and Kansas entered the Union as the 34th State. At the same time Congress set apart and reserved 46,080 acres for a State University in Kansas. Lawrence was chosen over Emporia as the site of State University but only by a majority of one. The first meeting of the Board of Regents was held March 21, 1865, at which time the Rev. R. W. Oliver was elected Chancellor of the University.

The present city of Lawrence has a population of about 50,000 which includes approximately 18,000 University of Kansas students and 1,000 Indian students at Haskell Institute. Lawrence has an economic base consisting of agriculture, industry, and educational institutions. Agriculture is chiefly grains, livestock, and dairying. Industrial production includes chemicals, boxes, pipe organs, food canning, printing, greeting cards and gift wrapping ribbon, plastics, movie productions, and agricultural products processing. Education is an important segment of the economy with about 2,000 faculty and staff employed at the University of Kansas and Haskell Institute.

Haskell Institute, located on a 320 acre campus at the southeast edge of Lawrence was founded as the Indian Training School at Lawrence on September 1, 1884, with an enrollment of 14 students. The name Haskell was chosen in 1890 in honor of Dudley C. Haskell, Representative from Kansas and chairman of the House Committee on Indian Affairs. At the present time Haskell is one of the oldest and largest Indian Schools in the United States with an enrollment of 1,010 students. In 1962, Haskell Institute was designated a "Registered National Historic Landmark".

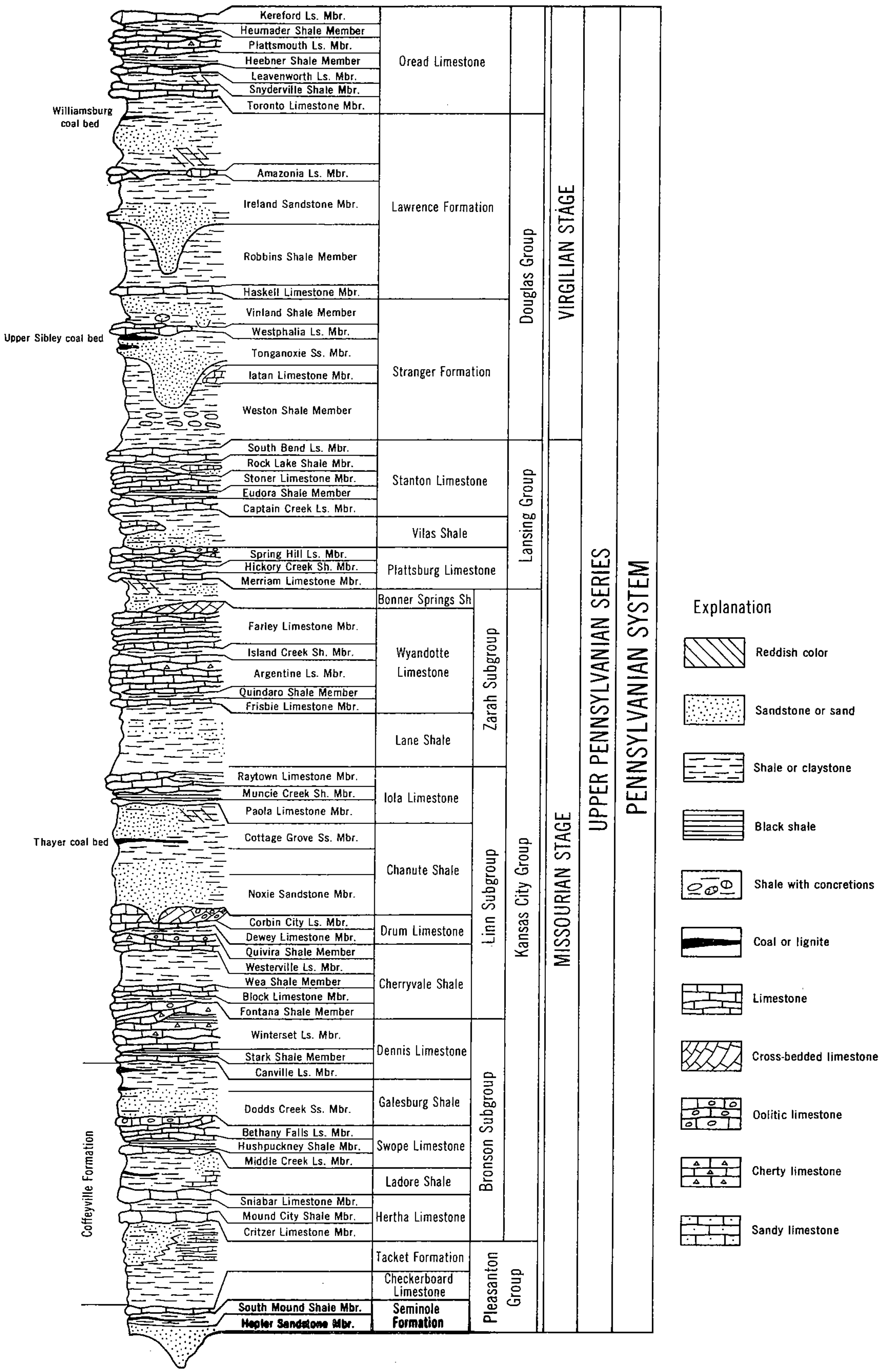


Figure 3. Classification of Pennsylvanian Rocks Outcropping In the Field Trip Area

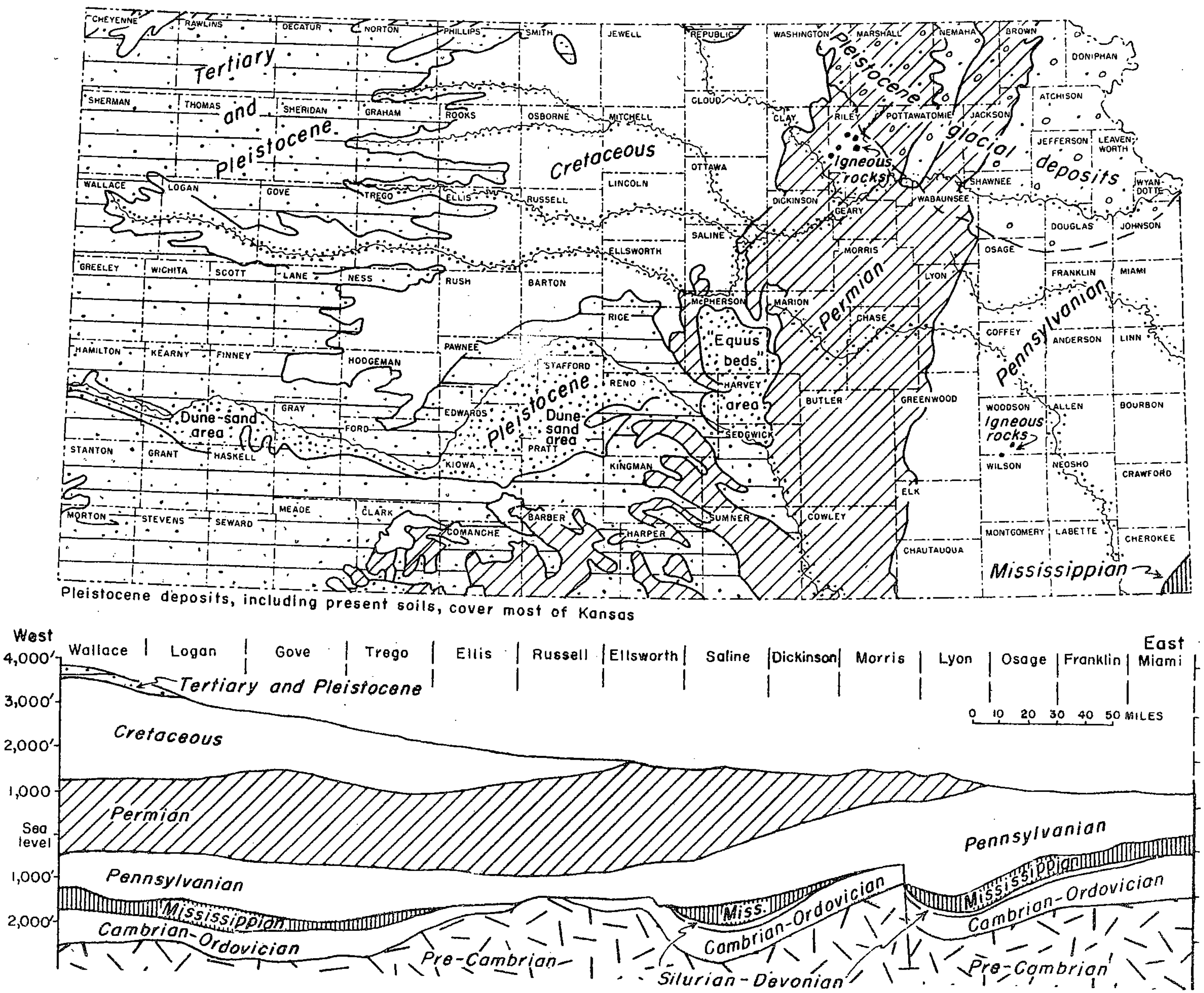


Figure 4
Generalized Geologic Map of Kansas And
Cross Section Showing Major Rock Units

State Geological Survey of Kansas

THE GENERAL GEOLOGIC SETTING OF THE FIELD TRIP

The field trip area lies within the Kansas River Valley. The underlying bedrock in this area, like in all of eastern Kansas is marked by eastward facing escarpments. This physiography expresses gently-westwardly dipping limestone formations interbedded with generally thicker shales beveled below a more-gently eastwardly inclined land surface. The surface is an exhumed peneplain that was developed probably early in Mesozoic time. The general westward dip of these Pennsylvanian rocks was acquired soon after their deposition. Their peneplanation accompanied, or followed immediately, their uplift and tilting. Much later during the Laramide orogeny when Kansas' surface took on an eastward slope, the dip of Paleozoic rocks in the eastern part of the state was lessened. Figure 3 shows the classification of Pennsylvanian rocks outcropping in the field trip area. Figure 4 shows the generalized geologic map of Kansas and cross sections showing major rock units.

Northeastern Kansas, roughly the area north of the Kansas River and east of the Big Blue River, was severely glaciated in Kansan time (Figure 4). Figure 5 gives the geologic classification of these Pleistocene deposits in Northeastern Kansas.

Glaciation strongly affected the development of the Kansas River Valley. Figure 6 shows the generalized cross section of the Kansas River Valley near Lawrence. Here the Kansas River valley is about 2.25 miles wide and is a trench nearly 300 feet deep cut into Pennsylvanian shales, sandstones, and limestones. Pleistocene alluvium underlies the flood plain and low terraces to a depth of 45 to 90 feet. In Kansan time the valley was filled with outwash approximately to the present level of the Menoken Terrace Surface. Since the Kansan filling, most of the Kansan and much Illinoisian and Wisconsinian alluvium has been removed leaving the valley terraced as we now see it (Figure 6). At Kansas City 150 feet of till has been reported below about 90 feet of Kansas River Valley alluvium (O'Connor and Fowler, 1963).

Rock-Stratigraphic		Time-Stratigraphic		
Eolian and fluvial deposits		Recent Stage	PLEISTOCENE SERIES	QUATERNARY SYSTEM
Bignell Formation	Fluvial deposits	Wisconsinan Stage		
Brady Soil				
Peoria Formation Gilman Canyon Formation	Fluvial deposits			
Sangamon Soil		Sangamonian Stage		
Loveland Formation	Fluvial deposits	Illinoisan Stage		
Yarmouth Soil		Yarmouthian Stage		
Loess	Fluvial deposits	Kansan Stage		
Cedar Bluffs Till				
Fluvial deposits				
Nickerson Till				
Atchison Formation				
Afton Soil		Aftonian Stage		
Loess	Fluvial deposits	Nebraskan Stage		
Iowa Point Till				
David City Formation				

Figure 5
Classification of Pleistocene Deposits
In Northeastern Kansas

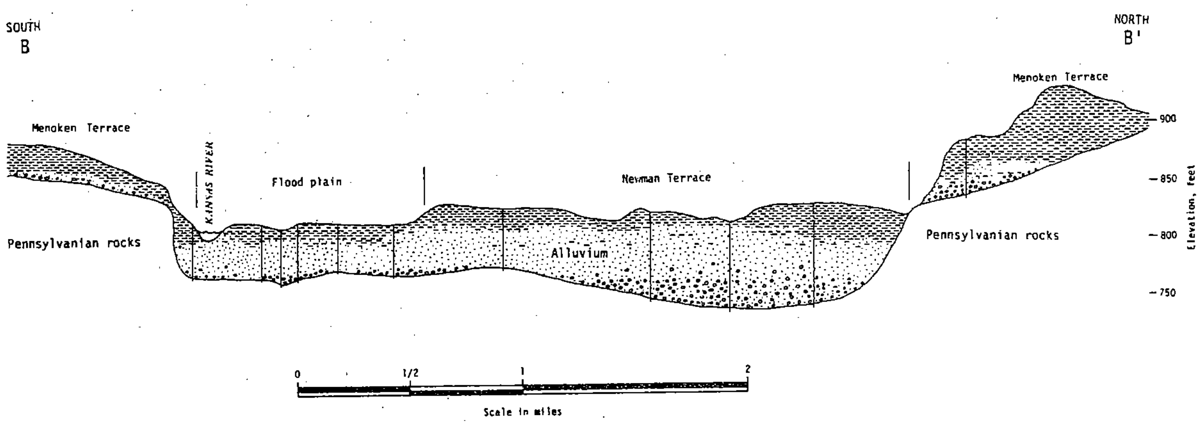


Figure 6
Generalized Cross Section of Kansas River Valley
(From Davis and Carlson, 1952)

THE FIELD TRIP ROUTE

GENERAL

The field trip route is shown by the broken line on the Guide Maps (Figure 1 and Figure 2). There are areas of special interest along the route, which are designated as Sites. A Site area may have several locations of interest. All observations of Sites will be made from the bus. There are four Stops along the route. Passengers will leave the bus at all Stops. Stop 2 will be the lunch stop. A road log is available in appendix 1 of this guide book.

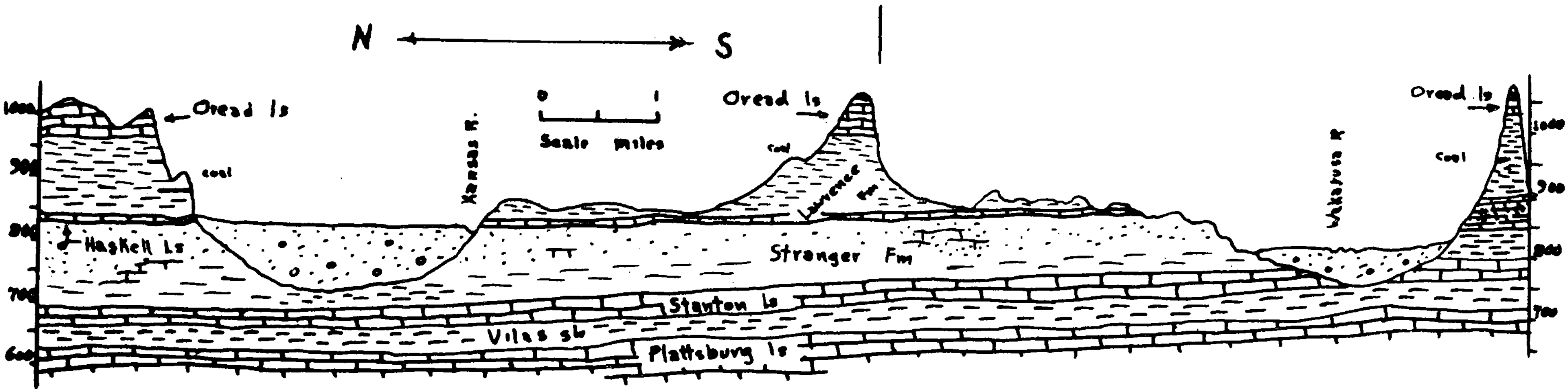
BEGINNING POINT OF FIELD TRIP - Mount Oread

The field trip originates from the campus of the University of Kansas, which occupies a high ridge between the Kansas and Wakarusa River Valleys. This high ridge, called Mount Oread, is underlain by the Oread Limestone Formation and the Lawrence Formation. Mount Oread is an eastward extending peninsula-like prong of an eastward facing escarpment or cuesta (Figure 7).

Site A1 - Exposure of Oread Limestone N.W. Lawrence
A2 - Kansas Turnpike Bridge over the Kansas River at Lawrence
A3 - Kansas River Valley at Lawrence

A1 - As we travel north on U.S. Highway 59 through the west edge of the K.U. campus past the junction with U.S. Highway 40 we can see the open pit quarry in the north wall of the Kansas River Valley which is in the Oread Limestone (Figure 3).

A2 - Traveling eastward on the Kansas Turnpike we cross the Kansas River bridge (Figure 8). This is a continuous deck truss bridge setting on mushroom type pier shafts and pile bent abutments. The river piers are founded on spread footings in the shaly sandstone of the Stranger Formation 40 feet below river level. The other piers and abutments are founded on pile footings driven to bedrock.



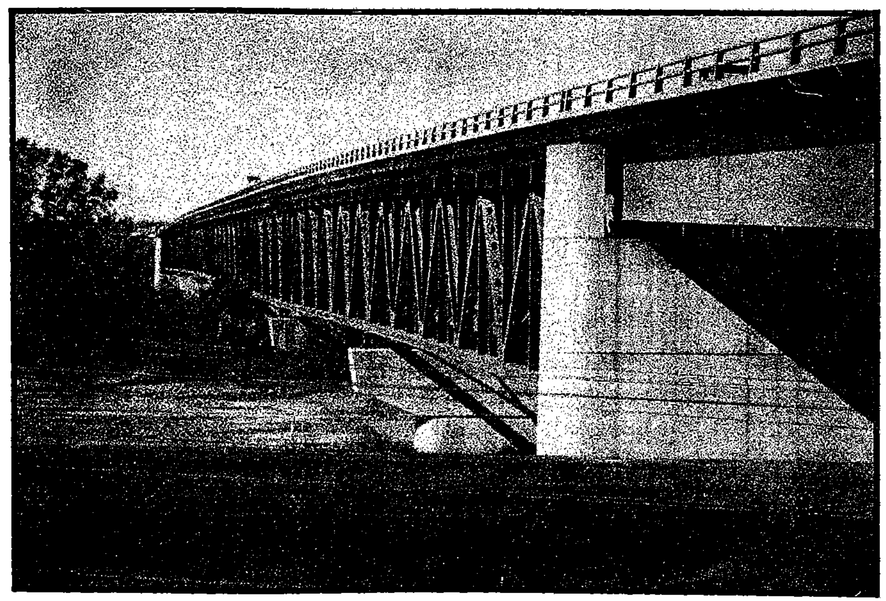


Figure 7 Above

Geologic cross section through Mount Oread, upon which the University of Kansas Campus is located, and extending to the river valleys on either side.

Figure 8 Right

Kansas Turnpike Bridge crossing the Kansas River at Lawrence. A continuous deck truss bridge setting on mushroom type pier shafts and pile bent abutments.

A3 - A short distance farther east, one of two local sand and gravel pits can be seen along the south side of the turnpike. The alluvium in the valley is an important groundwater reservoir. Wells having capacities ranging up to about 2000 gpm furnish municipal, industrial, and agricultural irrigation supplies.

SITE B1 - Highway Cuts in Douglas and Lansing Group Rocks along Kansas Turnpike.
B2 - Agricultural Hall of Fame and National Center near Bonner Springs

B1 - At intervals along the route from about 3 to 11 miles east of Lawrence are exposures, largely sandstone, of rocks in the lower part of the Douglas Group (upper Pennsylvanian). There are several outcrops of Tonganoxie Sandstone, an important aquifer. Shallow wells have capacities up to 100 gpm. Traveling eastward one sees exposures of rocks in the Lansing Group, strata next lower and older than the Douglas (Figure 3). Lansing rocks are exposed both east and west of the Bonner Springs exit about 21 miles east of Lawrence. In the buried Lansing-Kansas City Rocks, oil production is found within structural basins ranging in depth from less than 2,000 feet in the central portion of Kansas to slightly more than 4,000 feet in the Western Portion.

B2 - The Agricultural Hall of Fame and National Center is located about three-quarters of a mile north of the Bonner Springs exit. This center is dedicated to the continuing contribution agriculture, and related industry, has made to the American economy. The Hall of Fame wherein past leaders will be recognized, is only one of about 18 proposed features at the site.

Formal opening of the first Hall, one of a complex of ten buildings being planned, was observed June 12th, 1965. This unit consists of an agricultural museum, historical agricultural library, administrative offices and outdoor exhibits of heavy machinery, such as steam engine tractors, farm implements, and other equipment.

Site C1 - Older Highway Cut Slopes on K-7 at the East Edge of Bonner Springs
C2 - Newer Highway Cut Slopes on K-32 at the East Edge of Bonner Springs
C3 - Lone Star Cement Plant at Bonner Springs

C1 - This highway cut, in alternating strata of limestone, shale and sandstone, has been exposed to weathering for about 10 years (Figure 9 and Figure 10). The slope design recommendations were: Soil mantle on 3:1 or flatter slope and seeded; the Tonganoxie Sandstone and all the shales on 1:1 slopes except that the Eudora and Vilas Shales could be steepened to a $\frac{1}{2}$:1 slope provided an 8' minimum bench was placed at the base of the Captain Creek Limestone.

The slopes were constructed nearly as recommended, except, there was no bench placed at the base of the Captain Creek Limestone.

The slope design recommendations being made at the time this highway was constructed probably intended to limit the width of right of way as much as building a stable slope.

Obviously, the shale slopes are too steep to retain slaked material, therefore, spalling of shale intervals cause certain limestone strata to be undercut. Benches as recommended in design would have been desirable at the base of the thicker limestone units.

Since this highway was constructed before presplitting of the limestone strata was in general practice, the slope faces were finished in a roughened condition due to over-blasting. Loose limestone in the joint pockets still remaining on the slope still shows the evidence of blast fracture.

A cracked and distorted road surface has developed over the Spring Hill Limestone Section in the area where the limestone was quarried from beneath the roadway by the contractor to be used in the making of shoulder material. Mud jacking of the pavement was required only a few months after completion due to settlement in the backfill material.

Interceptor type underdrains are used to drain water from permeable zones in the subgrade (Figure 9). An underdrain outlet is located at the guide post in the east ditch near the lower end of the cut.

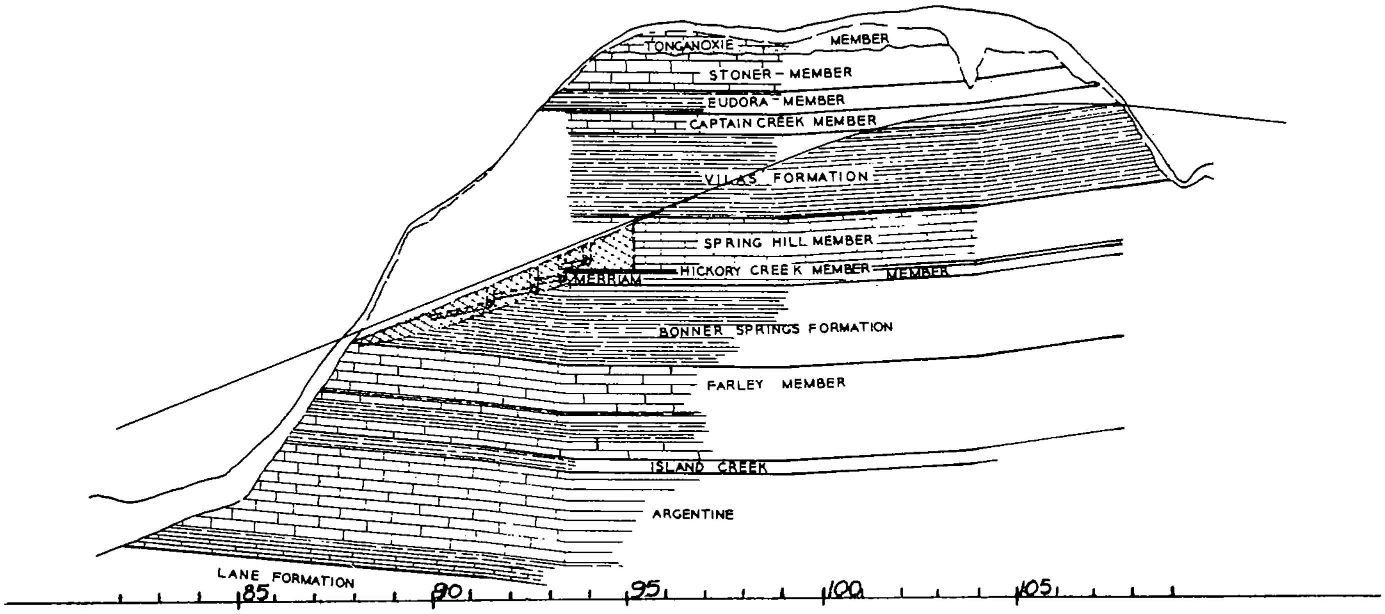


Figure 9
Geo-Engineering Profile of a Highway Cut Section in
Sedimentary Rocks at Bonner Springs

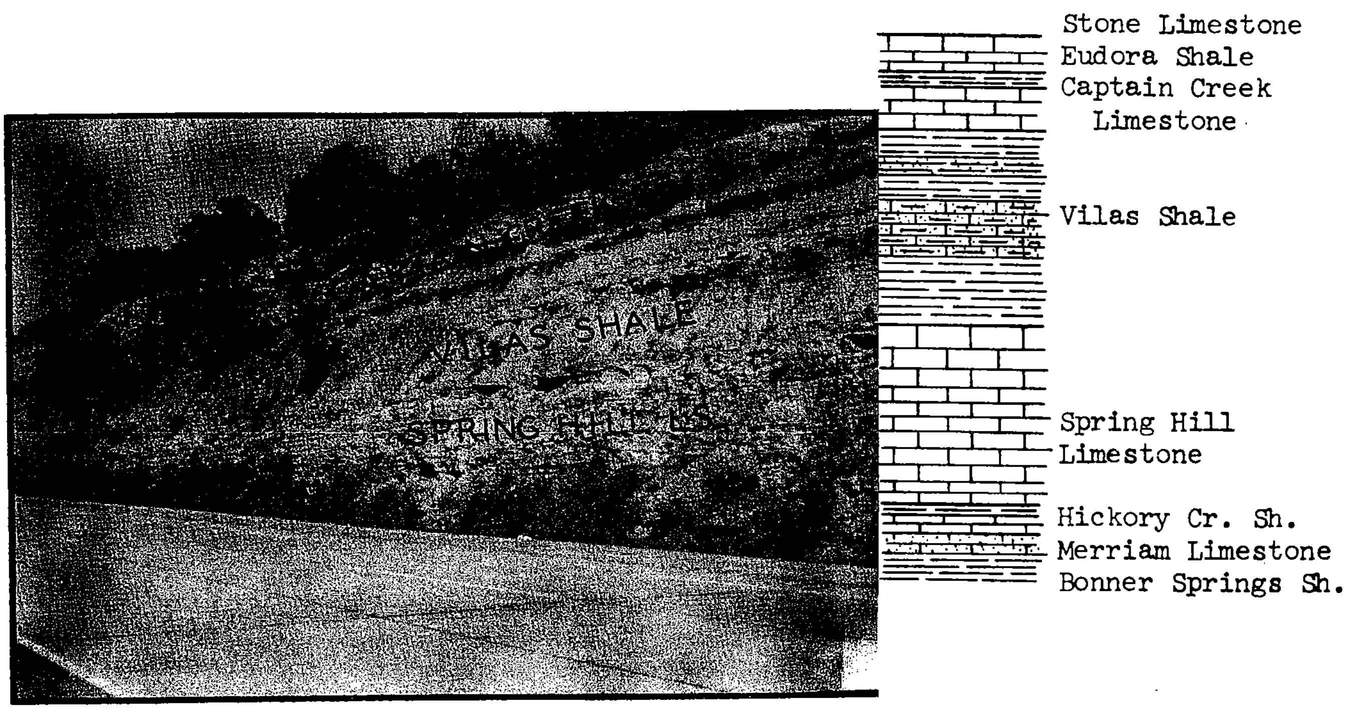


Figure 10
Cut Slope Shown Above, After Exposure
To Weathering for Several Years.

C2 - Here we see limestone and shale exposed along the north backslope face of K-32 Highway. The limestone faces have been presplit and all loose blocks, developing along the joint planes, were removed from the slope face. A bench has been constructed at the base of the Argentine-Frisbie Limestone (Figure 11). Cut slopes constructed in this manner have an improved stability and a more pleasing appearance. The bench constructed on the top of Lane Shale will catch any spalling blocks from the above Argentine-Frisbie Limestone.

Water is induced into the roadway in the west half of this side hill cut by the Paola Limestone, Muncie Creek Shale, and Raytown Limestone Members. This water is being carried away from the west portion of the cut by an interceptor type underdrain (Figure 12). This underdrain, with its 8" outlet running about half full of water, empties into a ditch near the west end of the cut.

C3 - The Lone Star Cement Plant observed at this site produces about 6,500 barrels of Portland Cement per 24 hour day having a yearly productive capacity of 2,400,000 barrels. The plant uses 2,000 tons of cement rock per day. The source of rock is quarried from the Argentine-Frisbie, Farley, and Spring Hill-Merriam Limestone units, with a composite thickness of approximately 75 feet (Figure 13). The Bonner Springs Shale is wasted in the quarrying operation. Portions of this geologic section are shown in Figure 9 and Figure 10. Gypsum is required in the manufacturing process and this is mined from strata of Permian Age in South Central Kansas.

This is one of six Portland Cement plants presently producing in Kansas. The other plants are located at Iola, Humboldt, Chanute, Independence and Fredonia.

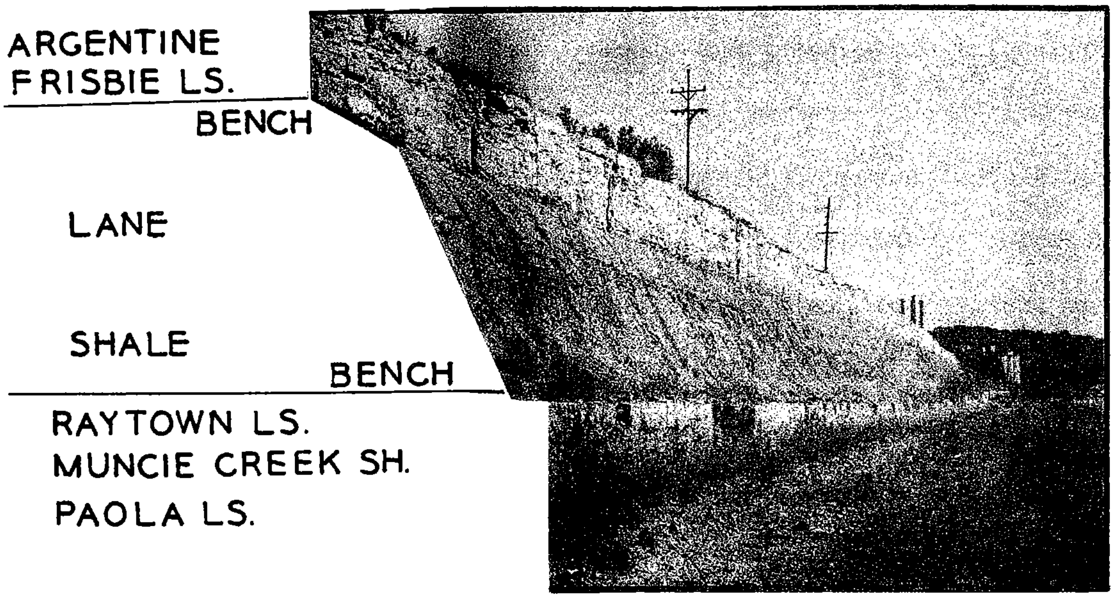


Figure 11 Above
Exposed Highway Cut Section
In Sedimentary Rocks On K-32
At Bonner Springs

Figure 12 Right
The Installation Of An Underdrain
At The Bottom Of The Cut Section
Shown in Figure 11

PORTLAND CEMENT MANUFACTURING PROCESS
AT
BONNER SPRINGS, KANSAS

(Courtesy Portland Cement Corporation)

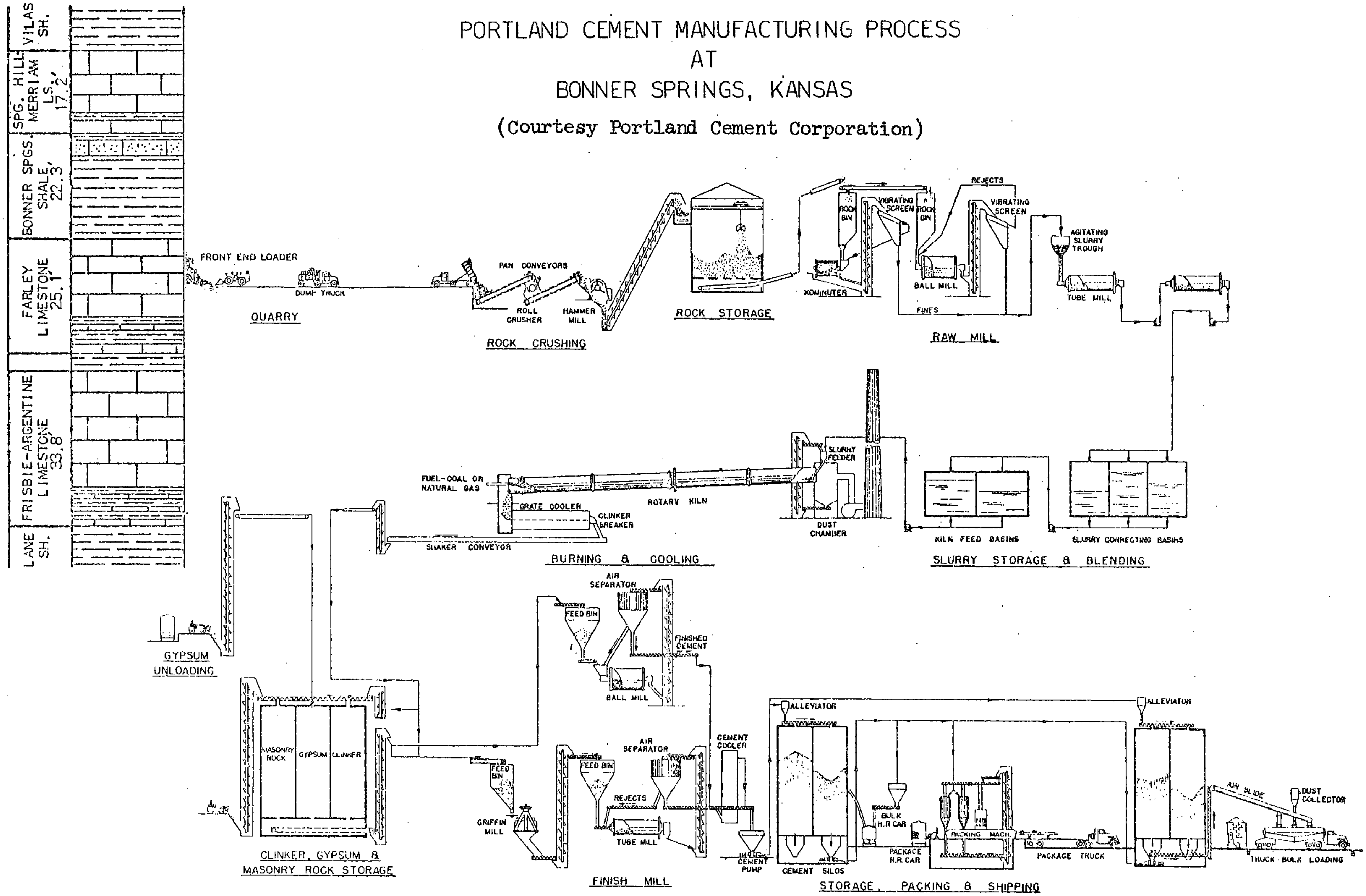


Figure 13

Site D - Sidehill Backslope Construction on Highway K-32 Running East of Bonner Springs

K-32 Highway running eastward from Bonner Springs was constructed in 1928. In order to keep the highway as high in elevation as possible and not cut through the valuable farm land and at the same time take advantage of the flat valley floor for construction, the highway was aligned along the bottom of the steep, 180 foot high valley wall. An electric railroad line was constructed on the toe of the valley wall just above the highway.

During construction of the new highway in the past year the old highway and the abandoned railroad right of way was utilized with very little additional purchase of right of way.

Construction along a steep side hill such as this poses several problems. Number one is the fact that if the slope is designed on a stable angle the amount of excavation becomes extremely large and the right of way may extend out several hundred feet. In this case the backslope had to stay inside of the old railroad right of way so it was not possible to design a natural stable slope. Another problem is the large amount of colluvial material which collects along the lower part of the valley wall. When the toe of this material is cut out it starts to slide and as the ditches are cleaned out it continues to slide.

Figure 14 shows the side hill condition a short distance west of Edwardsville. In order to stabilize the colluvial material and protect a house and garage from sliding downhill, a concrete pipe was placed in the ditch to carry the ditch drainage. The backslope was then carried down and out over the pipe in order to buttress against the side slope.

Figure 15 shows an equally troublesome problem where the side hill is predominantly shale. In this situation there are two potential slide planes, one at the soil mantle-shale contact, and the other at the sloping contact between the weathered and unweathered shale. Both of these zones carry water resulting in the loss of friction and thereby sliding is easily produced.

In order to stabilize the backslope a rock embankment was constructed on top of a limestone ledge about 8 feet above ditch bottom.

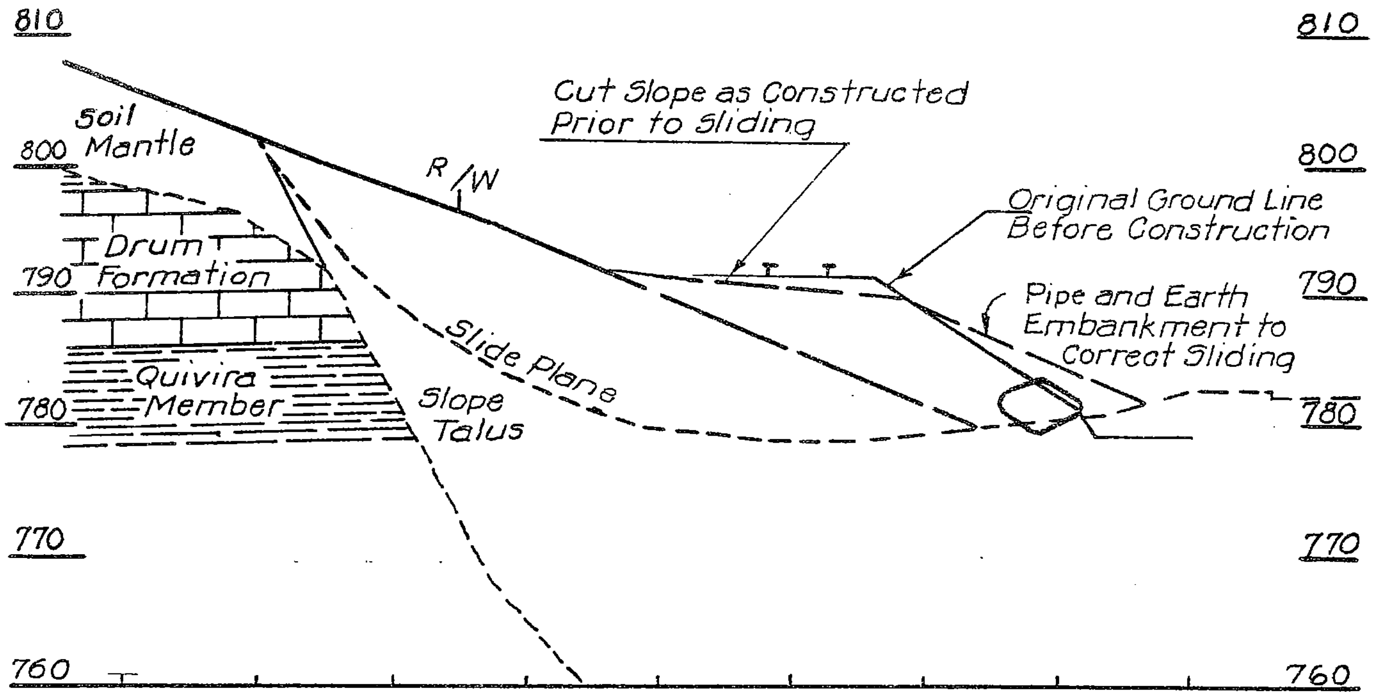


Figure 14
Side Hill Backslope Construction Just West of Edwardsville
Illustrating One of the Problems Along K-32

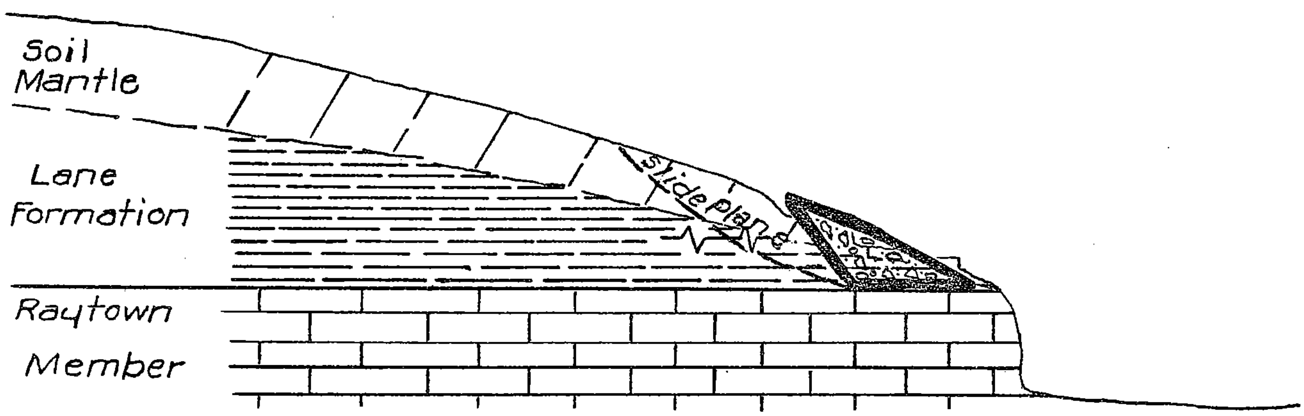


Figure 15
Side Hill Backslope Construction Just East of Edwardsville
Illustrating a Problem Along K-32

Site E1 - The Degradation Pattern of Turkey Creek
E2 - Muncie Creek Plant of the Holiday Sand and Gravel Company
E3 - Grinter House

E1 - About three miles east of Edwardsville, Highway K-32 crosses Turkey Creek. In 1961, when K-32 was a two-lane highway, the bridge over Turkey Creek collapsed as a result of degradation of the stream's channel and undercutting of the bridge's footings. Valuable farm land upstream from the bridge was severely eroded.

An investigation was conducted to determine the cause of the degrading action which had accelerated sharply five years prior to the collapse of the bridge. Stream bottom elevations at the bridge, taken at different times between 1925 and 1961, were available. Aerial photographs, taken in 1941, 1954, 1959, 1961, and 1962 were used to study lateral movement of the channel and to measure the changes in the channel width at selected locations. Correlation of field and aerial photo data at one point revealed that for each foot of degradation, the width of the channel would increase $4\frac{1}{2}$ feet. Further correlation disclosed that the marked increase in degradation occurred after two channel changes took place downstream from the bridge. The first was a natural meander cutoff which occurred between 1954 and 1959. The second took place between 1959 and 1961 which involved the relocation of the Chicago, Rock Island and Union Pacific Railroad Bridge. Figure 16 shows the changes that occurred between 1941 and 1962 as taken from aerial photographs.

The new bridge, 190 feet long and 82 feet roadway width, has reinforced concrete girder spans on pile bent abutments and pedestal type piers. Steel pile were used in the abutments, driven to 37 tons point bearing on bedrock. The pier columns are set on ribbon footings on the Bethany Falls Limestone about 18 feet below streambed.

To protect the banks from scour, rip rap was placed in the bridge area beginning 50 feet upstream from the bridge and tying into the railroad bridge rip rap downstream.

With this bank protection and with footings on bedrock, any further degradation of the stream channel should have very little effect on the stability of the new bridge.

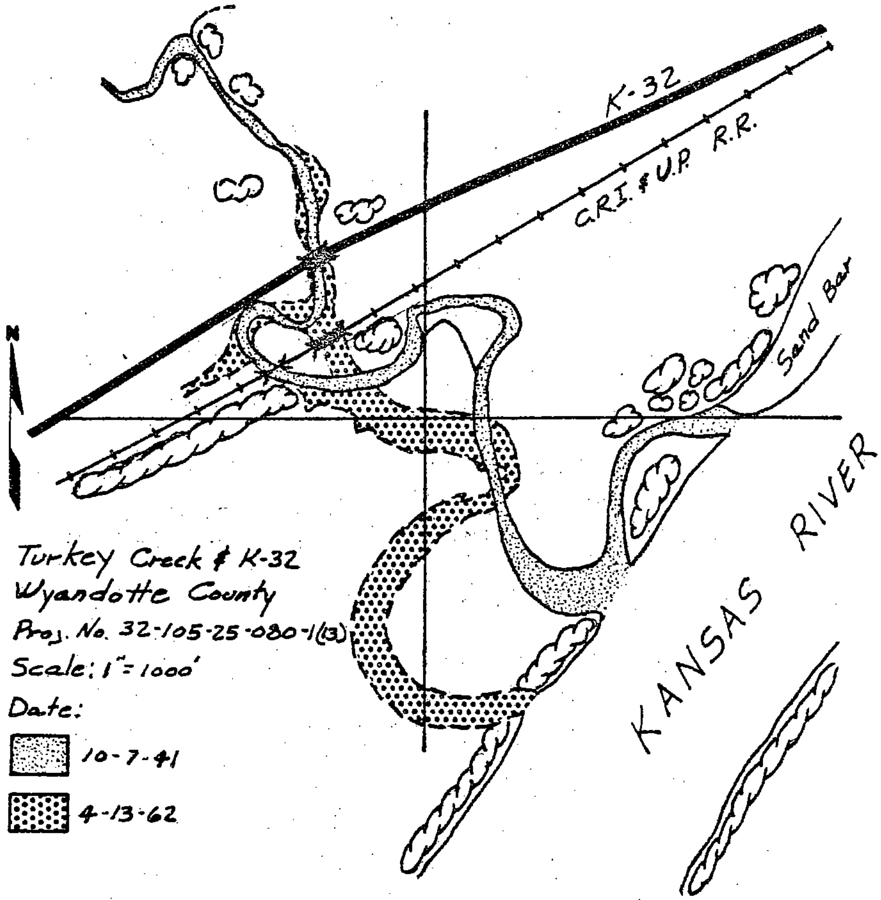


Figure 16
Degradation Pattern of Turkey Creek

E2 - On the right is the Muncie Creek Plant of the Holiday Sand and Gravel Company. Sand and gravel is pumped from the Kansas River for aggregate for construction purposes.

E3 - Approximately one mile east of Turkey Creek, on the left (north) side of the road is Grinter House. This historic house was built in 1857 by Moses Grinter, who operated the first ferry on the Kansas River. The house is now a privately owned museum.

Site F - Kansas River Bridge at Turner

The K-132 Bridge crossing the Kansas River at Turner is a continuous span through truss bridge setting on pedestal type piers and pile abutments (Figure 17). The three river piers are founded on spread footings in the sandy shale of the Pleasanton Formation about 45 feet below water level. The abutments and two end piers are on piling driven to bedrock. The bridge deck is 1323 feet long and is constructed entirely of light weight concrete consisting of expanded shale aggregate and seven sacks of cement per cubic yard. It is estimated that the decrease in dead load of the bridge deck due to the light weight aggregate resulted in a savings of $40,000 in structural steel.

The previous bridge was located a short distance upstream and was washed out in the 1951 flood. The present bridge was open to traffic in 1955.

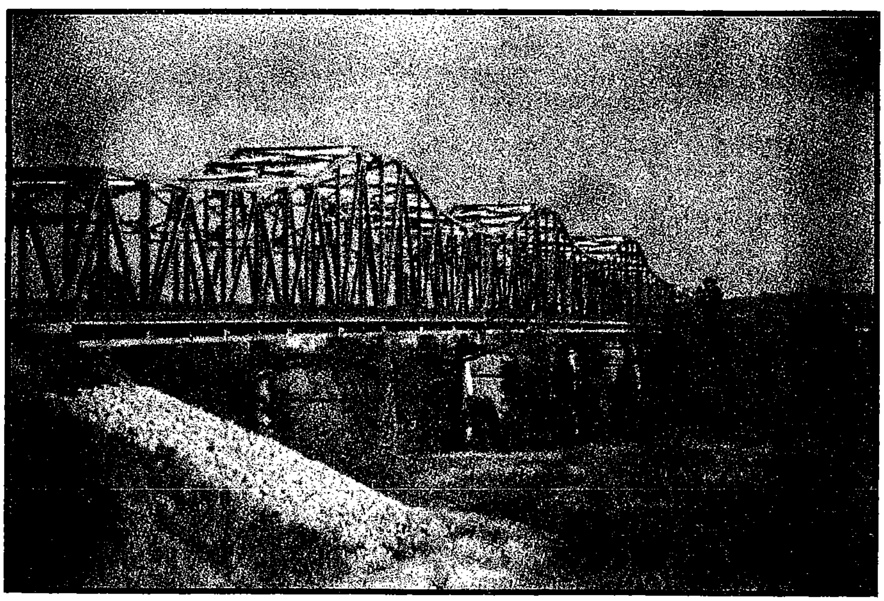


Figure 17
Kansas River Bridge On K-132 At Turner

STOP 1 <u>INSPECTION OF A BETHANY FALLS LIMESTONE MINE</u> (The Inland Underground Facilities)

Here we will inspect a Bethany Falls Limestone Mine (Figure 18). An increasing number of these mines are being developed in the Kansas City area with the growing demands for building aggregates and underground space.

BETHANY FALLS LIMESTONE MINES

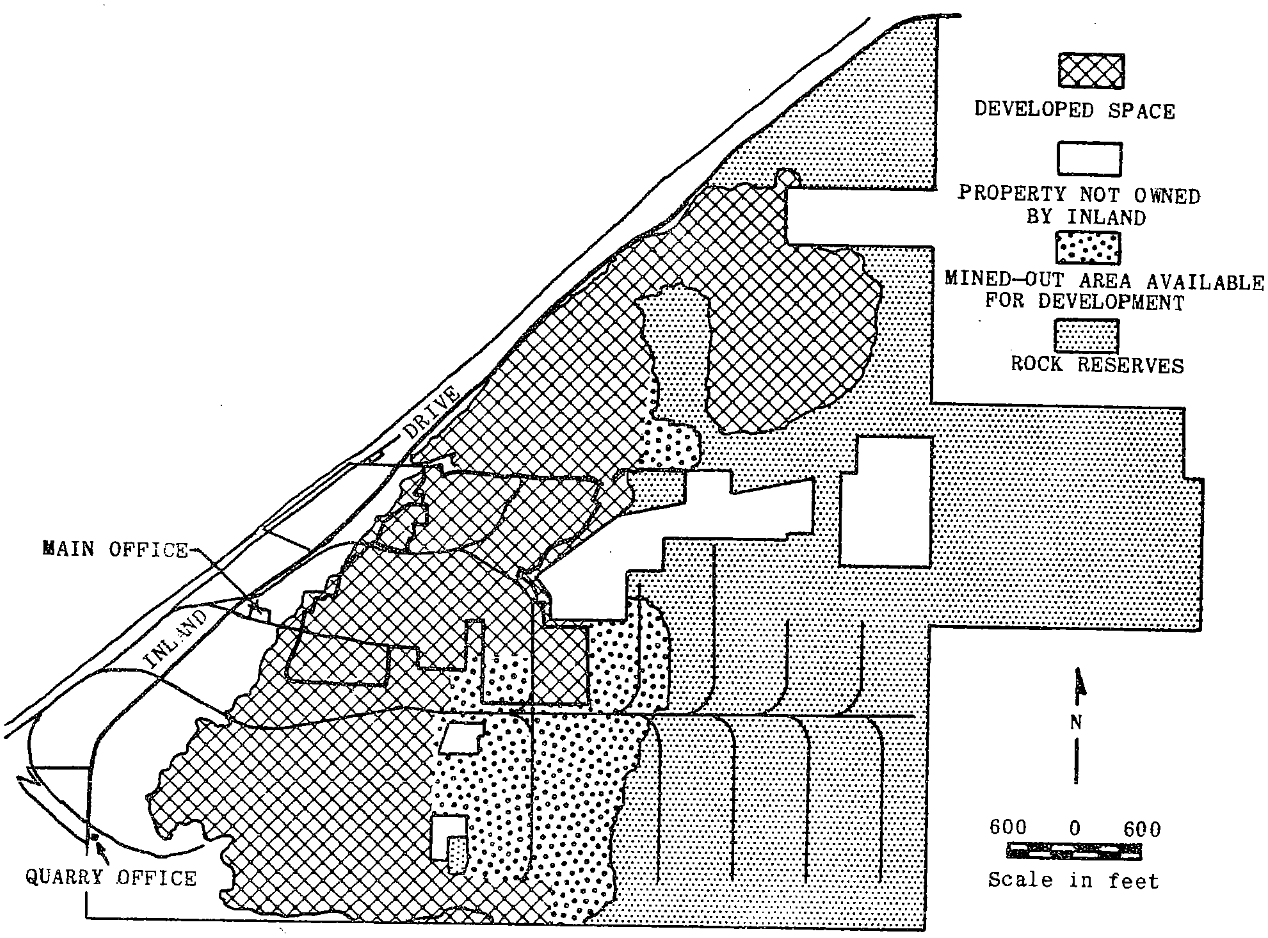
1. Mining Operations for Aggregate
2. Development of Underground Space
3. Underground Transit Ways

1. In the Kansas City area, the limestone most satisfactory for crushed stone is the Bethany Falls Limestone member of the Swope Limestone Formation (Figure 19). The limestone has long been quarried or mined and most of the raw material is crushed and separated into commercial sizes. The finished products are sold as concrete aggregate, road base material, asphalt aggregate, agriculture lime and for manufactured concrete products. The limestone is mined by the room and pillar method. The Bethany Falls is a flat lying, relatively pure limestone ledge with thickness and structure that permits constructing a stable roof and floor. The spacing and size of pillars is an important consideration for permanent stability with usually 65 feet being the maximum distance between centers. By leaving the upper Bethany Falls cap rock in place, rooms have a vertical clearance of approximately fourteen feet. In the operation, about 80% of the rock is removed leaving 20% in pillars to support the overburden which is composed of limestone and some shale. The overburden thickness will usually exceed 150 feet in minable areas (Figure 1). The mining of the limestone, if properly conducted, does not impair or distract from the usefulness of the land surface.

2. Within the past 20 years, the economic development of underground space in the mined-out areas have become increasingly important. Research and experimentation along with feasibility studies have brought about consideration for more and more underground space development. Some of the proven uses for this space are

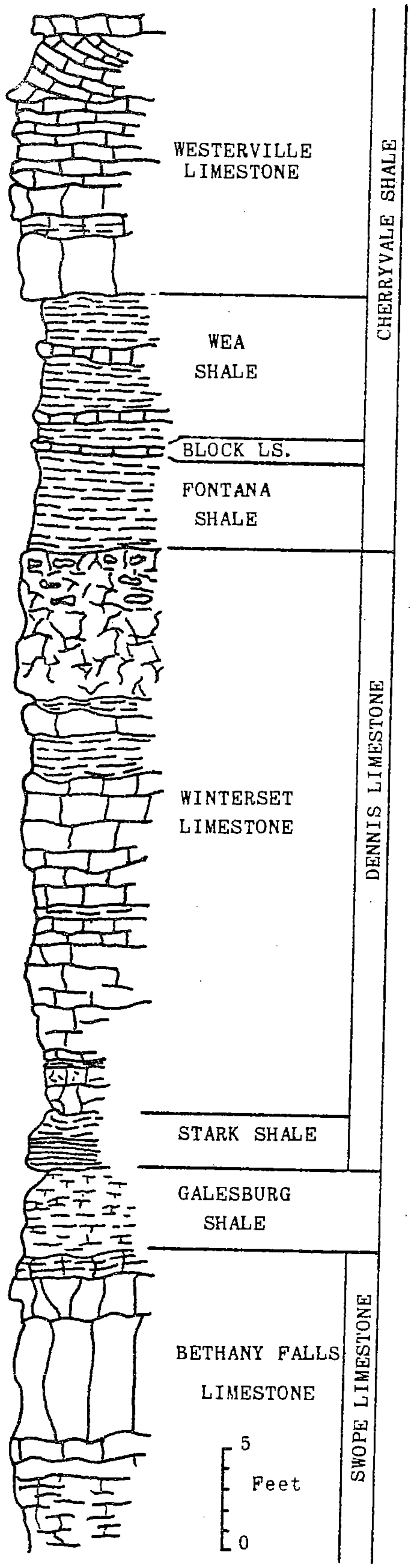
warehousing, manufacturing sites, office space and vital record centers.

3. The building of underground transit ways through the Bethany Falls Limestone, to be carried out in conjunction with limestone mining operations, has been considered possible for many years. In the past several years keen interest has developed in underground transit and underground facilities and public officials, quarry operators and engineers have made serious study as to their feasibility. There are several potential underground transit projects in Metropolitan Kansas City for which feasibility studies will be made at such time as bond issues are approved for them.



(Courtesy of Inland Underground Facilities)

Figure 18
Underground Map, Inland Underground Facilities



Westerville Limestone - light gray on fresh surface, weathers buff, dense, non-oolitic, fossil "hash", lower contact, also may be pinkish-gray on fresh surface. Has a shale parting in lower part, shale light gray on fresh cut, weathers buff non-fissile.

Wea Shale - medium to dark gray to black on fresh surface, weathers light gray to buff. It has two thin limestones in the lower part.

Block Limestone - a thin, medium to dark gray on fres surface, weathers buff, dense, evenly bedded; forms a prominent ledge.

Fontana Shale - medium to dark gray on fresh cut, weathers light gray to buff, upper 6 inches calcareous, fissile.

Winterset Limestone - medium gray on fresh cut, weathers light gray, dense, thin bedded at top, silty, jointed, chert black to bluish gray on fresh surface, weathers light yellowish brown, lenticular or nodular bed, upper surface pitted, unfossiliferous.

Stark Shale - dark gray and black, blocky on fresh cuts weathers light gray, grades upward from black fissile shale to gray blocky shale.

Galesburg Shale - gray-green on fresh surface, weathers light gray, blocky, massive appearance where exposed.

Bethany Falls Limestone - light to dark gray, weather light gray, fine grained to crystalline, algal, massive to thin-bedded, joints poorly defined, small structures light algae, weathers into relief, contains chert locally.

Figure 19
Graphic Representation Of Rocks In The Vicinity Of The Inland Storage Complex (From Jewett And Others 1965)

Inland Underground Facilities

A - History and Development

B - Warehouse

A - Inland Underground Facilities had its inception in 1912 as a quarrying and underground mining operation employing immigrant labor. This business was acquired in 1941 by interests related to the present management. In 1965 it became a division of Beatrice Foods Company.

In 1952, management determined that a major portion of its mined-out underground area was economically usable for other purposes. Certain adjustments were made in the mining procedure to give the mined-out space greater facility for future development. The initial conversion of this space, after some experimentation was for warehousing. Under steadily broadening demand and development of other uses for the mined-out space, major areas have been converted into manufacturing sites, office space, and a vital records center. The extensive mined-out areas are ideal for many other commerical uses such as light manufacturing, food processing or any other industry requiring constant temperature and humidity conditions. This space is highly desirable as a Department of Defense "Hardened Site" for firms or personnel engaged in critical occupations or storage of strategic supplies.

Inland Underground Facilities owns approximately 700 acres of land at this location, about half of which has been mined (Figure 18). At the current rate of extracting, approximately 10 to 12 acres of limestone a year, this property contains a sufficient deposit to meet requirements for the next 35 years. The rate of depletion is geared primarily to the demands from the construction industry in the Kansas City, Kansas area. With the availability of ample storage space for stockpiling, the mining operation continues on a year-round basis.

This property is contiguous to the main line of the Atchison, Topeka, and Santa Fe Railroad. It lies within the Kansas City Railroad Switching Limits, thereby enjoying free reciprocal switching privileges with all twelve main railroad lines that serve Kansas City. The plant is also situated within the Commercial Trucking Zone of Greater Kansas City.

B - Inland Underground Facilities have been successful in turning this mined out space into an excellent refrigerated warehouse by taking advantage of the following factors:

1. A constant 55 degree F. temperature that provides a below zero refrigerated space for less than one half the cost of conventional above ground freezer space.

2. All employees work in ideal controlled temperature and humidity including the dock workers, working along an extensive dock area where sixty rail cars and sixty trucks are handled simultaneously.

3. It is centrally located in the heart of America and near Kansas City, second most active rail switching center in the United States. This makes shipments to the Inland Plant from any direction possible, to remain in storage on an in-transit basis, and to be re-shipped to a maximum number of destinations.

4. With more than one-half of the nations processed and frozen food being produced in the west and a large portion of this food being transported to heavily populated centers East of the Mississippi River, vast tonnage can be stored in-transit in these Inland Underground Facilities. The vast capacity, flexible rail center and geographic position provides for the lowest full carload or truck load rates. There are 60 acres of stacking space, of which 50 percent is below zero degrees F., about 10 percent is in the 32 degree F. range and about 40 percent for storage at 70 degrees F.

The total tonnage of products handled in and out in a normal day is 8 million pounds. The normal inventory stored in the warehouse at any time would provide over a pound of food for each person in the United States at any time.

Site G - Well Field for Johnson County Water District No. 1

Between Inland Underground Facilities and Lake Quivira about two miles to the west, Johnson County Water District No. 1 has a series of 21 wells in Kansas River alluvium. The wells are each 65 to 70 feet deep and yield about 8 million gallons per day. In 1965 a surface water intake in the Kansas River and additional water treatment works were added to the water supply facilities to serve an area of northeastern Johnson County that has a population of about 115,000 people.

Because the bedrock yields only small water supplies in and around Kansas City the large municipal and industrial water supplies are all obtained from wells in the Kansas and Missouri River Valleys or from the Kansas and Missouri Rivers or artificial impoundments.

At Holiday, Kansas (Figure 2), just before we pass under the Santa Fe Railroad tracks is the Holiday Post Office. In the Kansas River flood of 1951 flood waters reached the bottom of the upstairs windows above the Post Office.

Stop 2 - Holiday Gravel Pit and Lunch Stop

At this stop (Figure 2 and Figure 20) Kansas outwash deposits, overlain by a thin deposit of Peoria loess, are exposed in a gravel pit owned by the List and Clark Construction Company. The pit is no longer being worked as a source of sand and gravel. About 30 to 40 feet of outwash deposits are exposed in the walls of the pit. Records of nearby wells indicate there are probably an additional 40 to 50 feet of Kansan deposits, chiefly sand and gravel, below the floor of the pit. A water-supply well at the List and Clark Company office, located a short distance east of the pit reportedly penetrated 40 feet of Kansan outwash. The well is equipped with a 50 gallon per minute pump and provides a water supply for the office and shop.

The gravel pit (Northeast Northwest Section 11, Township 12 South, Range 23 East) is located in an abandoned segment of the Kansas River valley which was excavated and filled during the Kansan glaciation. This valley, called Holiday Valley, is one of several filled and abandoned valleys in the Kansas City area (O'Connor and Fowler, 1964). Holiday Valley has a width of $1\frac{1}{2}$ to 2 miles, and Dufford (1950, page 26) reported a maximum thickness of more than 100 feet of outwash. Although mostly sand and gravel are exposed at this pit, test hole data in other parts of Holiday Valley indicate much of the outwash deposit is sand or sandy and clayey silts with small amounts of coarse gravel.

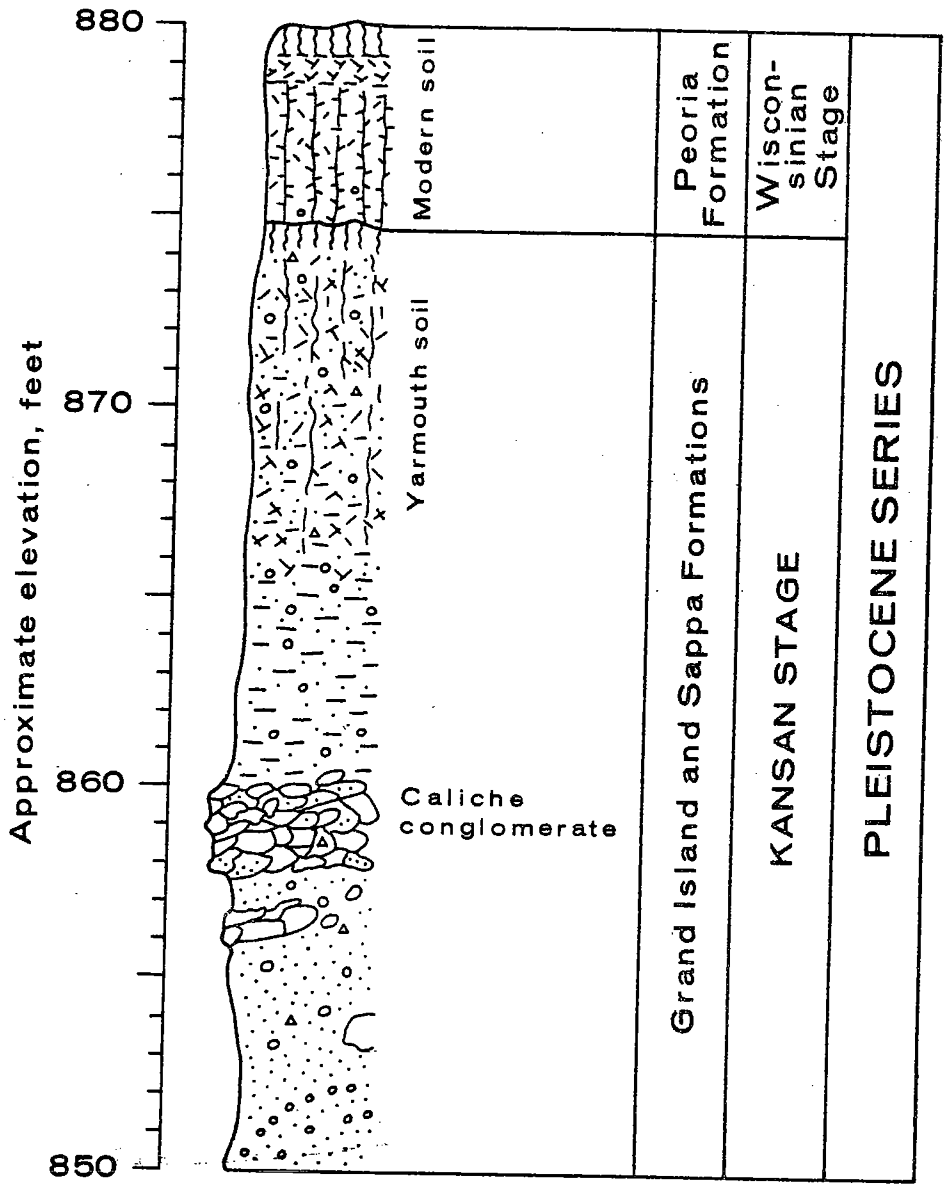


Figure 20
Stratigraphic Section of Rocks Exposed At
The Holiday Gravel Pit

In detail, the sediments exposed in the face of the pit are highly variable from place to place, but in the northeast corner of the pit about 5 feet of Peoria loess is at the surface and overlies a paleosol developed on Kansan outwash (James Thorpe, 1965, personal communication). The paleosol is largely the Yarmouth Soil but may include a few inches of silts representing possible Sangamon Soil just below the base of the Peoria. Except for the horizon of the modern soil and the Yarmouth Soil which are black to gray, the Peoria is chiefly reddish brown, red, and yellowish red. The sediments in the upper 15 to 20 feet of the exposure are leached. Sand and gravel below the leached zone is poorly sorted and irregularly cemented with calcium carbonate to form a hard caliche conglomerate 15 to 25 feet below the land surface. Some of the gravel is heavily stained with manganese oxide. The lowermost 10 to 15 feet of exposed beds are chiefly fine sand to fine gravel, mostly uncemented but with local cemented zones.

Davis (1951) determined the percentage of various lithologies in the 4 to 8 mm size range to be limestone, 53.5 percent; chert, 18.6 percent; shale and sandstone, 14.6 percent; igneous and metamorphic rocks, 13.3 percent. In the cobble and boulder size limestone probably comprises more than 90 percent of the gravel.

Although the sand and gravel deposits were used for road metal and as construction materials for many years, the pit has been used only as a storage area for List and Clark Construction Company recently.

Field trip participants will have time to inspect various parts of the pit during the lunch period scheduled for this stop.

Site H-1 - Underdrainage on Highway K-10
H-2 - Subgrading on Highway K-10

H-1 - Underdrainage - As we travel east on Highway K-10 a red and silver guard post can be seen at the northeast corner of the intersection of K-10 and Mill Creek Road. Here is the outlet for an underdrain, the plan for which is shown in Figure 21. The purpose of the underdrain is to intercept water seepage at the base of the limestone ledge and to carry the water out into the ditch. This will prevent the water from soaking into the roadbed and causing the wearing surface to fail. Construction procedure is as follows: The trench is excavated to the design elevation which is approximately one foot below the base of the limestone. A thin insulating course of aggregate is spread over the bottom of all the trench except in the outlet section. The pipe is then laid with the perforations down (Figure 22). The entire trench is filled with filter aggregate except the outlet section (Figure 23), which is backfilled with soil. The specified kinds of pipe that can be used are 6" I. D. corrugated metal, vitrified clay, asbestos, cement, bituminized fiber, and concrete. A concrete flume is formed around the outlet.

H-2 - Subgrading - Figure 24 shows an area of subgrading on Highway K-10. This location is a short distance east of the underdrain location. When laboratory tests indicate that a shale will be unstable under the roadbed, the shale is removed from the subgrade to a depth of 8" as indicated in Figure 25, and backfilled with more stable material. In this case crushed limestone was used for the backfill.

Treating soil and weathered shale with hydrated lime is also a method used to stabilize poor subgrade material.

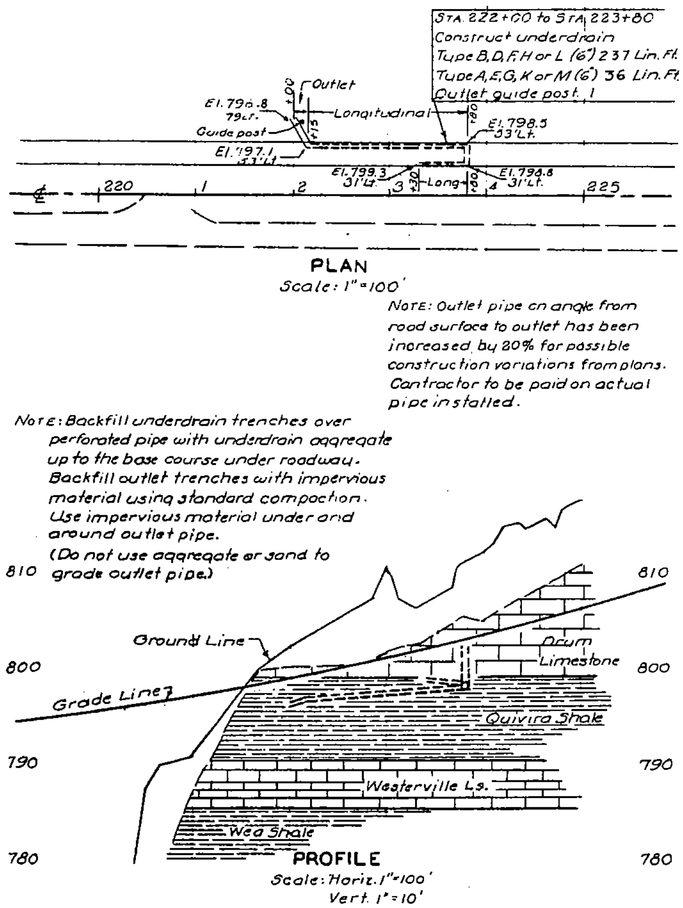


Figure 21
The Underdrain Plan

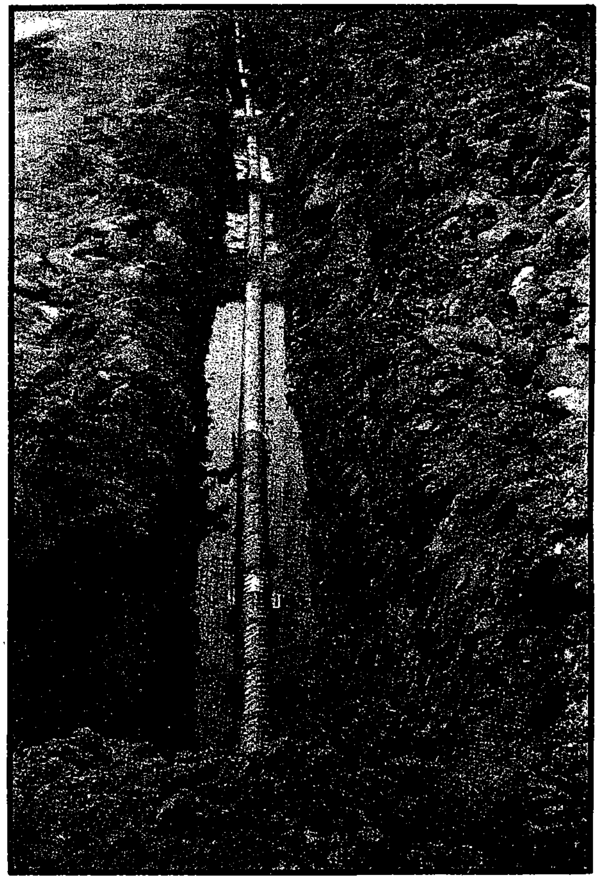
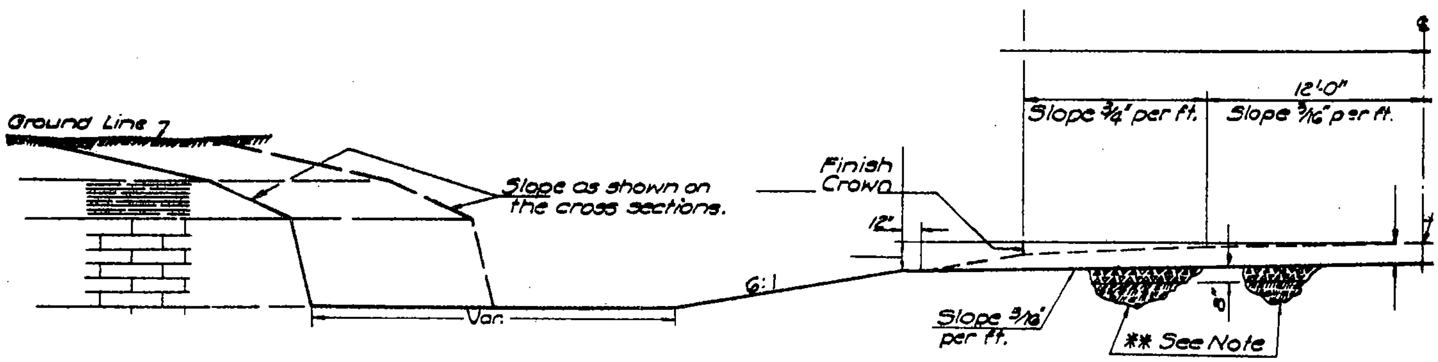


Figure 22
The Underdrain Pipe Laid
Into Trench

Figure 23
The Underdrain Outlet Section

Figure 24
Location Where Shale Was Subgraded



Cut Section

** NOTE: Overbreakage in rock or shale shall be brought to within 8" of subgrade with the best available material, properly compacted, and then brought to subgrade with crushed stone. This shall not be paid for directly but shall be considered a subsidiary item.

NOTE: Section "Embankments" of the standard specifications shall apply, except shale will be permitted in the top 12" of Embankment unless otherwise noted in the balances.
NOTE: Section "Excavation for Highway" of the standard specifications shall apply, except that subgrading will not be required in rock or shale unless otherwise noted on the plans.

Figure 25
Typical Section Showing Subgrading

Based on extensive studies and observations in the mine and including numerous physical tests on cores taken from various structural components of the mine the following conclusions were drawn. It is obvious that many of the pillars are settling since the loading pressure that is being transmitted to the shale foundation material is much greater than the underlying shale is capable of carrying. In addition the mine roof is tightly clamped along the walls so that when the pillars start settling out in the mine, the roof has to shear up next to the mine wall. From this it might be predicted that most of the caveins will occur along the mine wall. Also, it seems that as a group of pillars start settling the weakest one fails and throws an extra load on the next concentric circle of pillars. Those in turn fail and the load is transferred to the next circle until the roof reaches its maximum spanning ability and shears off in front of either the second or third circle of pillars.

3B - After studying several methods of treating the mine, the decision was made to take limestone out of the cut above the mine, along with some additional borrow, crush it to a maximum 6" size and place this material in the mine as fill. Also, the roof fall material in the mine was used as bulkheads (Figure 28). The material was placed up to within 12" of the roof except under Gibbs Road where a bridge is located half on and half off the mine and here the fill was placed tightly against the roof.

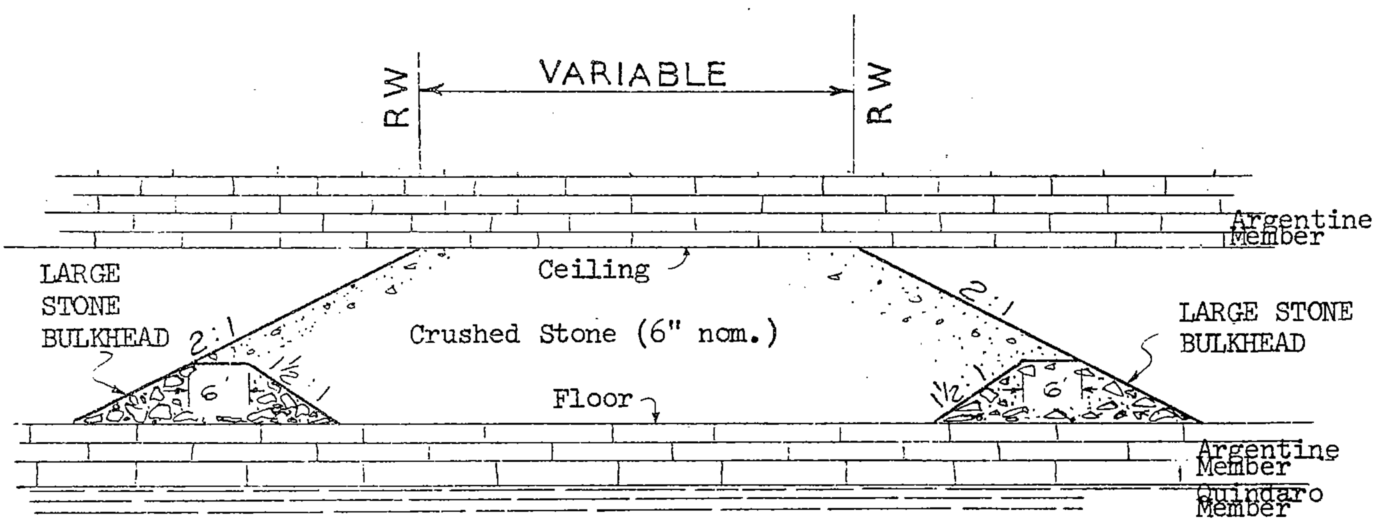


Figure 28
Typical Section Of Mine-Filling Under I-635

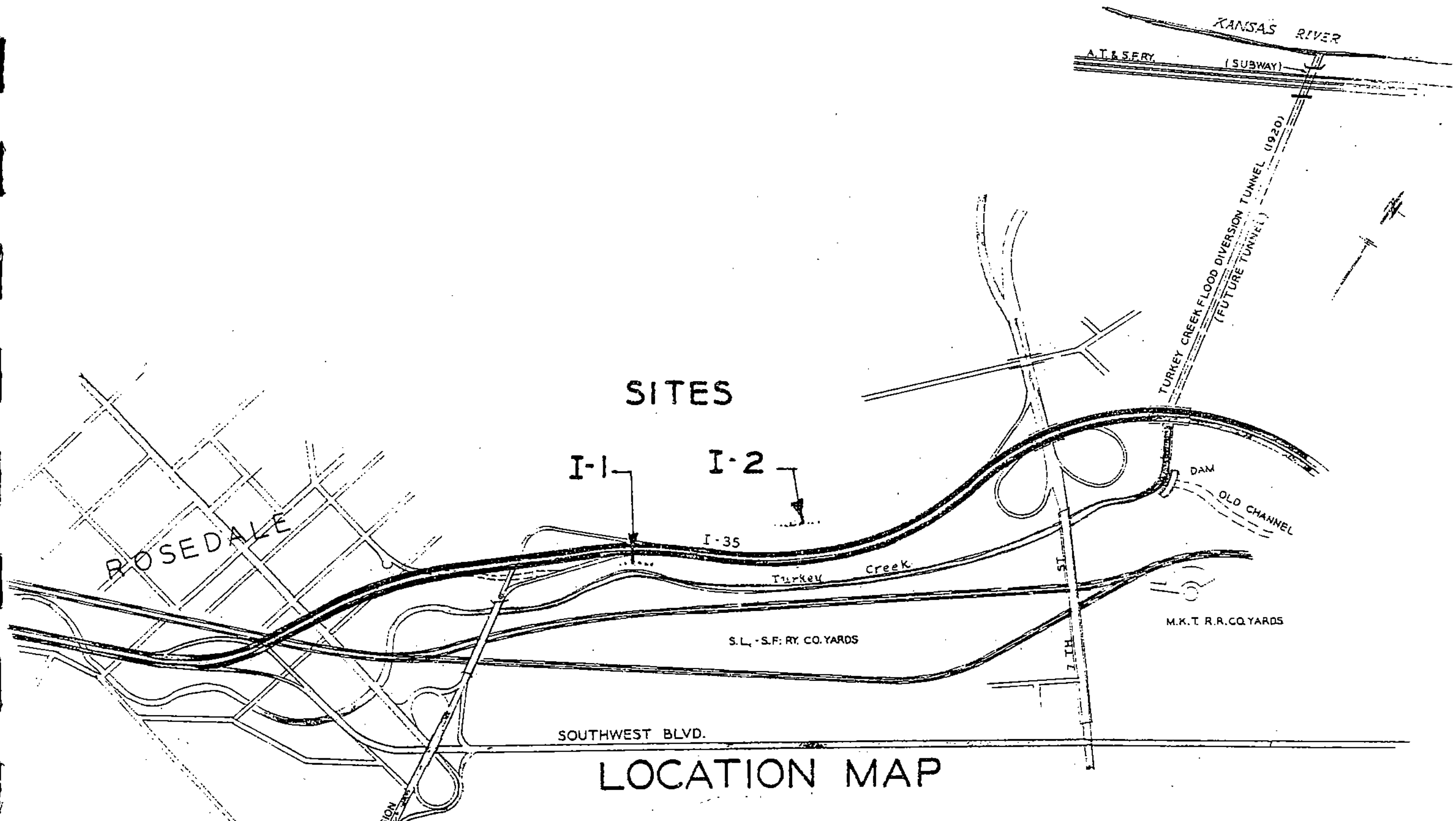


Figure 29
Interstate 35 Along Lower Turkey Creek
And Over Tunnel

Site I1 - Special Construction of Embankment
I2 - Unstable shale in Cut Slope

Several problems were encountered in constructing this section of I-35. The highway is built along the narrow flood plain of Turkey Creek and these problems, in part, were the results of flood protection work already completed or to be completed in this lower reach of Turkey Creek where it now empties into the Kansas River through a tunnel (Figure 29).

I1 - The construction of highway embankments for I-635 close to Turkey Creek channel required special considerations for both excavation and backfill as shown by the example in Figure 30. This method of embankment construction allowed building the embankment over an existing 30" sewer line, which was embedded in soft, wet alluvial material, without damage to the sewer.

Also, this rock backfill against the stream allowed for steepening the embankment slope so as not to restrict the width of the channel now and allow for the future widening of Turkey Creek without endangering the highway embankment.

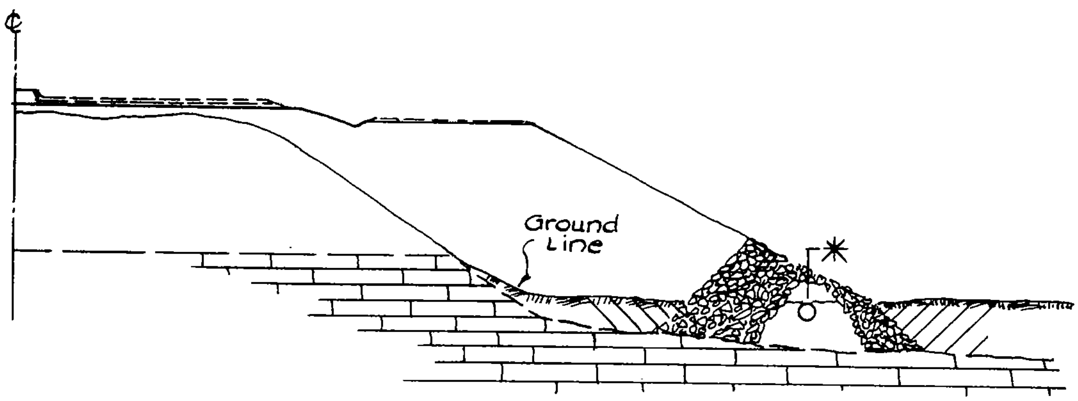


Figure 30

Special Excavation And Limestone Rock Backfill

12 - In this same area a scar can be seen in the lower north backslope where a large block of shale slid out during construction. This formation, the Wea Shale, is rather unstable in the backslopes due partly to its high clay content and probably more important is the irregular slope of the joints within the formation. The only treatment made here was to clean out the slide material. Figure 31 shows the sliding in the cut slope and Figure 32 shows the slope after the slide material has been cleaned away. Higher up the hill a large soil mantle slide can be seen. The slide material is just starting to flow over the top of the slope about 100 feet above the roadbed.

Figure 31
Backslope Slide in Wea Shale

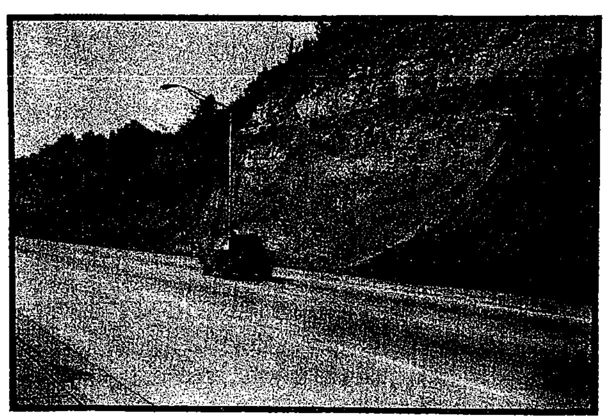


Figure 32
Slide Scar in Backslope

Stop No. 4 - Highway I-35 over The Turkey Creek Diversion Tunnel
4A - History
4B - Future Plan of the Corps of Engineers
4C - Bridge Foundation Plan
4D - Repair of Undermining Inside Tunnel

4A - Turkey Creek drainage is from an area about 13 miles long and 2½ miles wide covering some 15,000 acres. The drainage outside of the flatter flood plain consists of a hills topography with precipitous slopes causing a rapid concentration of flood waters in times of severe rainstorms. In about 1910 the District of Rosedale considered doing something about this serious flooding. Owing to the several interests involved, sufficient cooperation and funds were not available to start work for nearly eight years after the project was launched although damage in the meanwhile had been about half its cost (Engineering News Record). In November 1920, following more than a decade of planning and over 2½ years to construct, the Turkey Creek waters could be diverted into the Kansas River at maximum flow estimated at 20,000 Sec. Ft. through 500 feet of open channel change 1300 feet of 28 x 28 foot tunnel and 200 feet of concrete box subway (Figure 33). This work required a dam at the diversion, a sewer commencing below the dam to carry away the sanitary and storm sewage, the straightening, enlarging and diking of about 5,600 feet of channel above the diversion, and retaining walls beyond the subway to protect Kansas River levees from Turkey Creek Flood water.

The concrete lined tunnel is of horseshoe shape, 28 feet high and 28 feet wide. It passes through several ledges of limestone and shale, the roof and invert both being in limestone; the amount of rock cover near the upper end was small but deemed sufficient for the purpose as the rock appears to be of excellent quality. The rock, however, proved to be very unreliable after exposure, and ultimately the upper 750 feet of tunnel had to be timbered to support the roof. To avoid this difficulty on the lower end of the tunnel the grade was lowered some 3 feet and 20 foot stretches of concrete lining immediately placed at intervals of about 100 feet proved successful in stopping roof troubles.

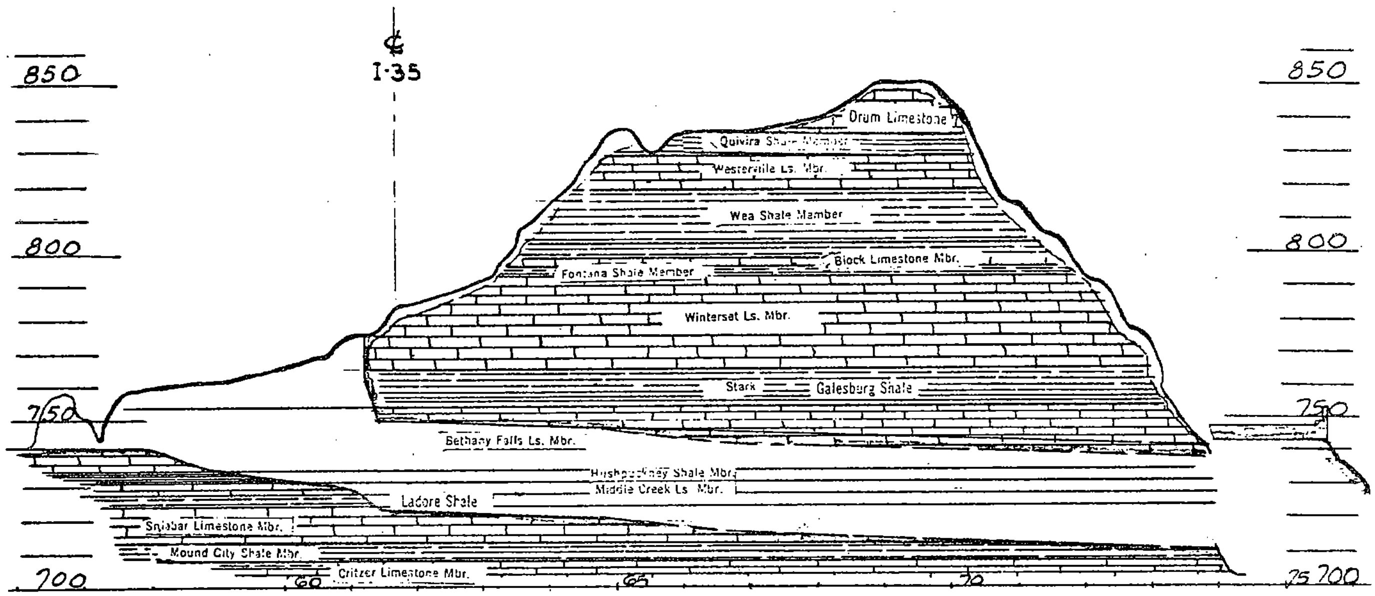


Figure 33
Turkey Creek Flood Protection Tunnel

4B - The future plan of the Corps of Engineers is to increase the capacity of the Turkey Creek Waterway and diversion tunnel. It will require straightening, enlarging and diking of the channel and cutting through a new tunnel along side the existing one. This future work, still in the early planning stage, required that Interstate 35 embankments be constructed on limited right of way in a manner not to restrict the widening of the channel in the future.

4C - The construction of the highway bridge over the portal section of the diversion tunnel required a special foundation design. The piers are on spread footings founded on bedrock. One abutment footing is on pile while the other is spread on bedrock (Figure 34). Figure 35 is the completed bridge over the portal of Turkey Creek Tunnel.

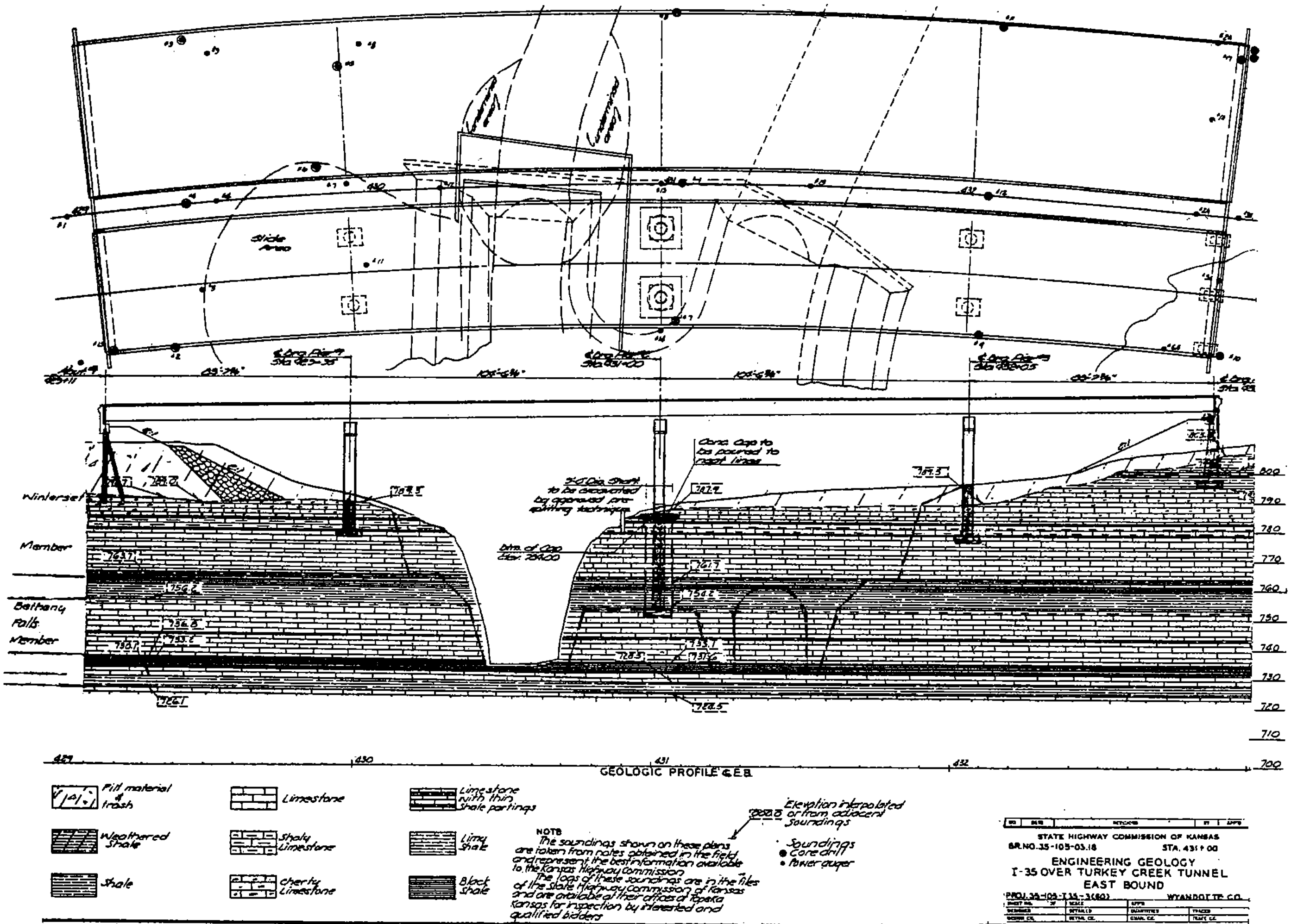


Figure 34
Bridge Foundation Plan Showing Future Tunnel Location

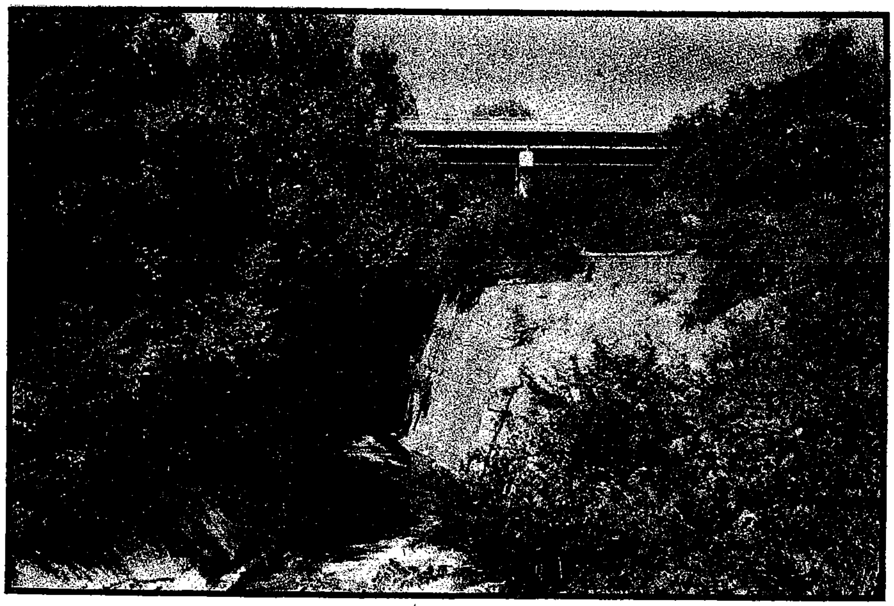


Figure 35
Completed I-35 Bridges Over Turkey Creek Tunnel

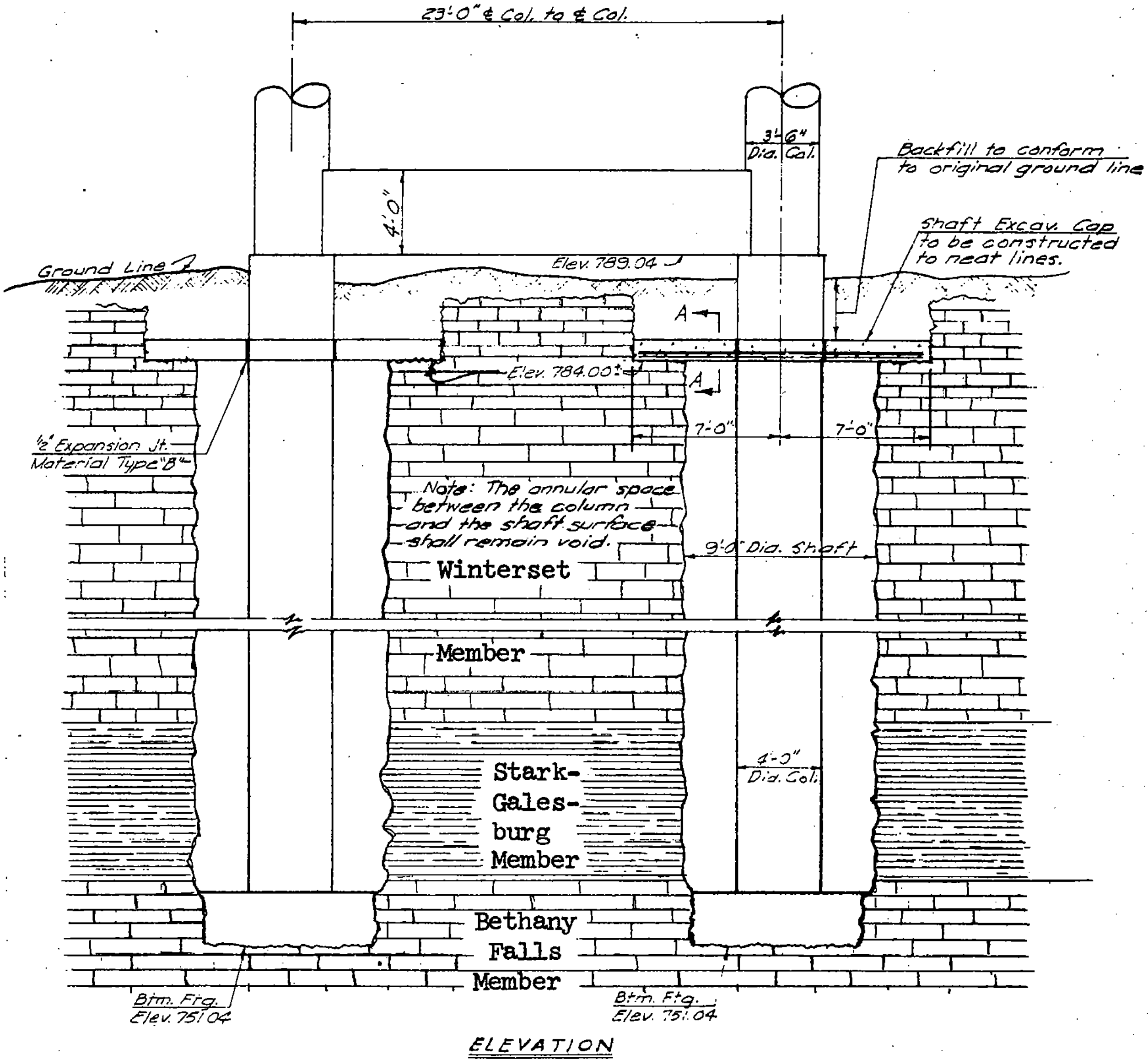


Figure 36
Footings And Columns For Middle Pier

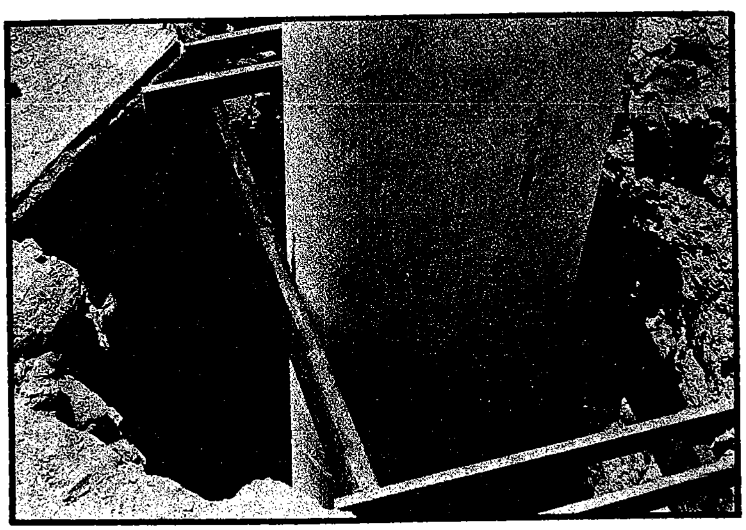


Figure 37
Finished Excavation And Free Standing Pier Column

The middle pier has a specialized column footing. The footings are set on bedrock that in the future will become the partition between the old and new tunnels. Therefore, these footings have been carried down to the top of the Bethany Falls Limestone. The pier columns have been constructed to be free standing through the interval of rock and shale to be removed later when the future diversion tunnel is built. In order to construct these free pier columns it was necessary to excavate 9' diameter shafts down to the footings. This open shaft prevents any overburden pressure against the pier columns and prevents the risk of blast damage during the excavation for the future tunnel (Figure 36 and Figure 37).

4D - To insure the support of the bridge foundation, it was necessary to repair serious undercutting in the mouth of the tunnel where a substantial section of limestone and shale had been undermined by erosion. This repair work required the temporary damming and the diversion of flow in the channel through large diameter pipe while repairing the undermined section, the floor, and the Ogee section of the tunnel. This repair work required forming, and the pouring of 795 cu.yds. of concrete (Figure 38, Figure 39 and Figure 40).

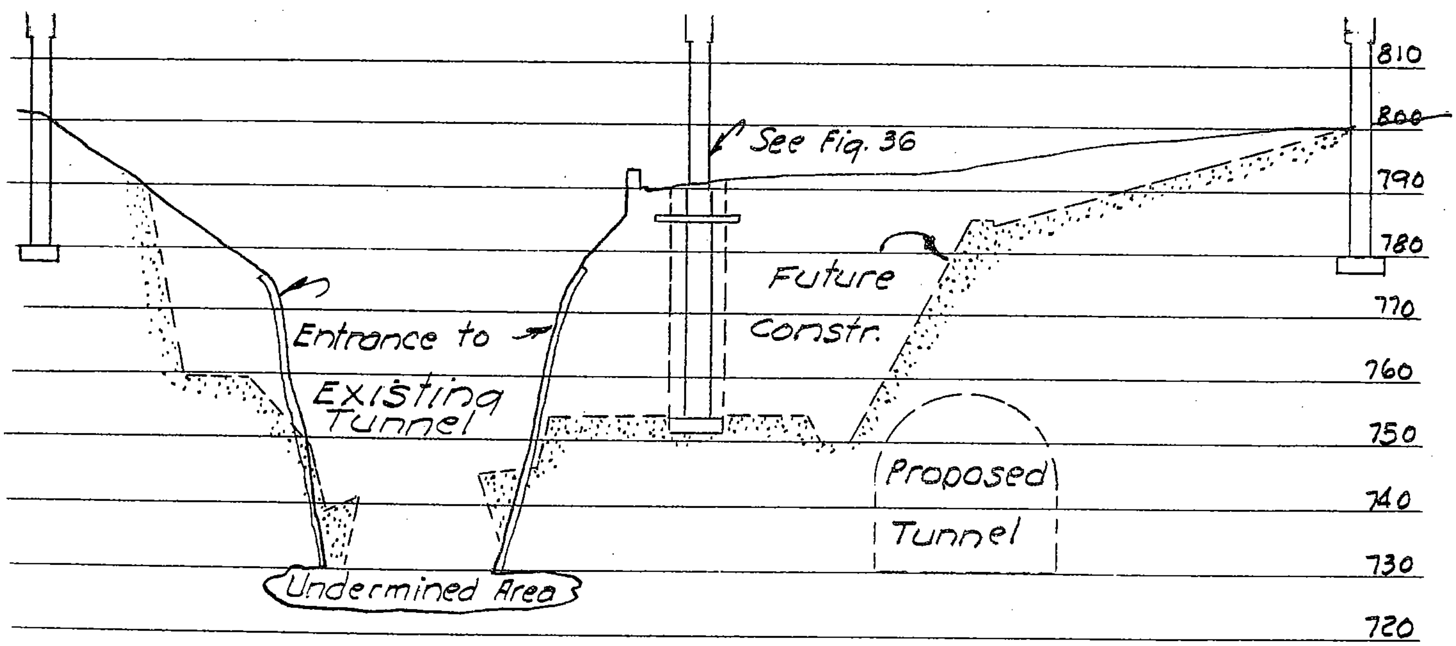


Figure 38
Under Mining In Alternating Shale And Limestone Section
In Diversion Tunnel

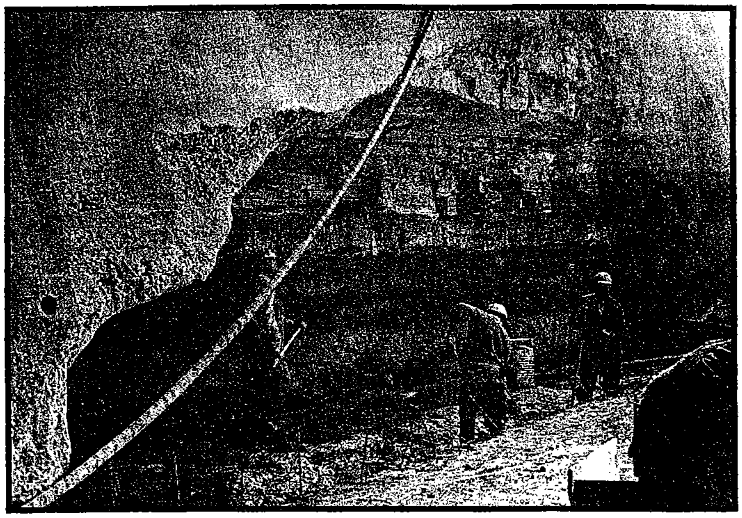


Figure 39
Preparing The Undermined Section of
The Tunnel For A Concrete Filling

Figure 40
The Completed Concrete Filling
Of The Undermined Tunnel Section

Site J - Cut Slope Problem in Pleistocene Deposits Along Kansas River Bluff
J1 - Slide Investigation
J2 - Recommendations

J1 - While under construction this cut section along I-70 near 4th Street slid out (Figure 41). Soundings were made along the toe of the slope, at the top of the slope, and at various other locations in the cut section to determine the locations of each type of material involved in the slide. The various soil layers encountered were tentatively classified into their textural and plasticity groups, and samples taken for laboratory testing.

In the laboratory, gradation and plasticity tests were run, also triaxial tests were run under 10 pounds per square inch and under 30 pounds per square inch lateral pressures. These tests were used to compute values of cohesion and the angle of internal friction for each sample.

In the area of the slide a very plastic clay was detected at approximately the same elevation as the bottom of the standard ditch section (Figure 42). Further investigation was made to accurately locate the clay layer and prepare a scale drawing showing its limits. Shear planes were observed in this clay in the pits dug through the layer, giving strong evidence that this clay was the cause of the failure. Further evidence was the fact that this type of clay was not found to underlie the toe of the slope to the north outside the slide area.

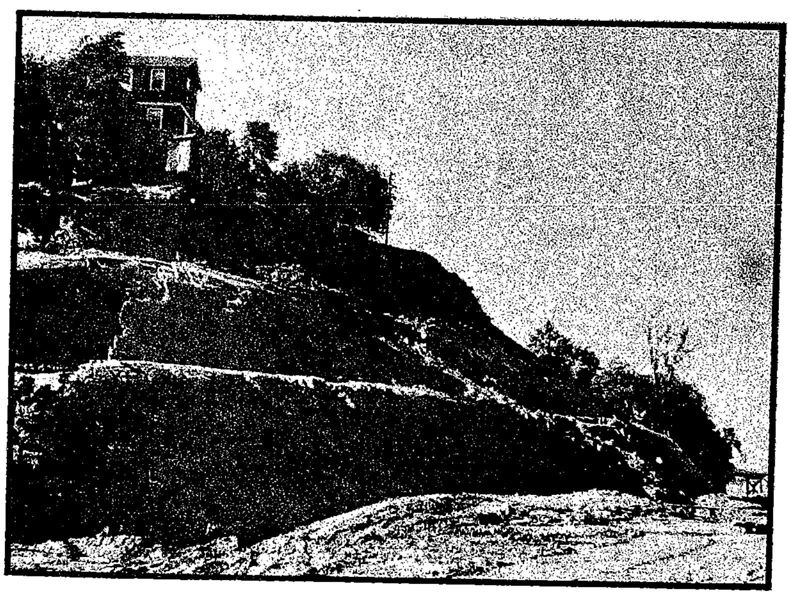


Figure 41
Backslope Problem In Pleistocene Materials On I-70 Along Kansas River Bluff Near 4th Street

J2 - The results of the investigation showed that a 2.7:1 minimum slope would be required to stabilize the material in the slide area. Since this minimum slope allowed no factor of safety, the toe of the slope was loaded with rock.

An allowable slope in the undisturbed portion of the cut north of the slide area was determined as follows:

Minimum Slope	Height
1.7:1	72 feet
1.5:1	60 feet
1.3:1	50 feet
1.1:1	40 feet

It is apparent that success was obtained by the use of this slope treatment, since no further sliding has occurred.

Figure 42
The Clay Layer Distorted By Shear
Located At Base of Slide

Site K - Complex Bridge Foundation at Junction of I-70 and 18th Street

At this site, located near the junction of I-70 and the 18th Street Expressway, the north pier and abutment of this railroad grade separation are located on the eroded and quarried valley wall. Figure 43 shows the pier with one footing setting on piling and one setting half on piling and half spread on limestone.

The north abutment (Figure 44) required varying lengths of pile, battered to fit the geologic conditions caused by an irregular bedrock surface.

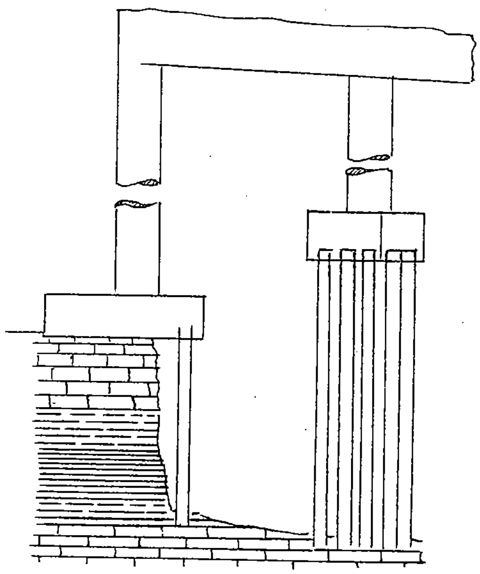


Figure 43
Pier Footing On Combination
Of Spread And Pile

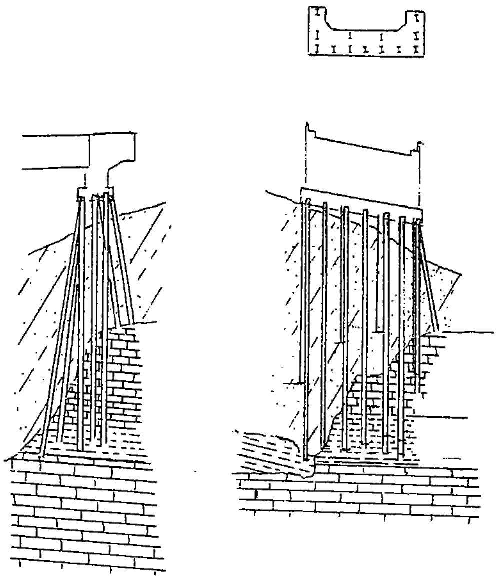


Figure 44
Abutment Footing On Pile Battered
To Fit The Irregular Bedrock Surface

Site L - Proposed Intersection of the Kansas Turnpike (I-70) and I-635

What started as an ordinary State Highway survey in 1950 turned into the location for Interstate 635 in 1960. The route passes through an area of trash which was an active trash dump before 1950. During and after the great flood in the Kansas River Valley in 1951 trash of all sorts was wasted in the area. This trash, which included everything from dead animals to the family sofa, was then covered over with dirt. The area remained an active city dump until 1965.

Figure 45
Proposed Interchange I-635 And Kansas Turnpike

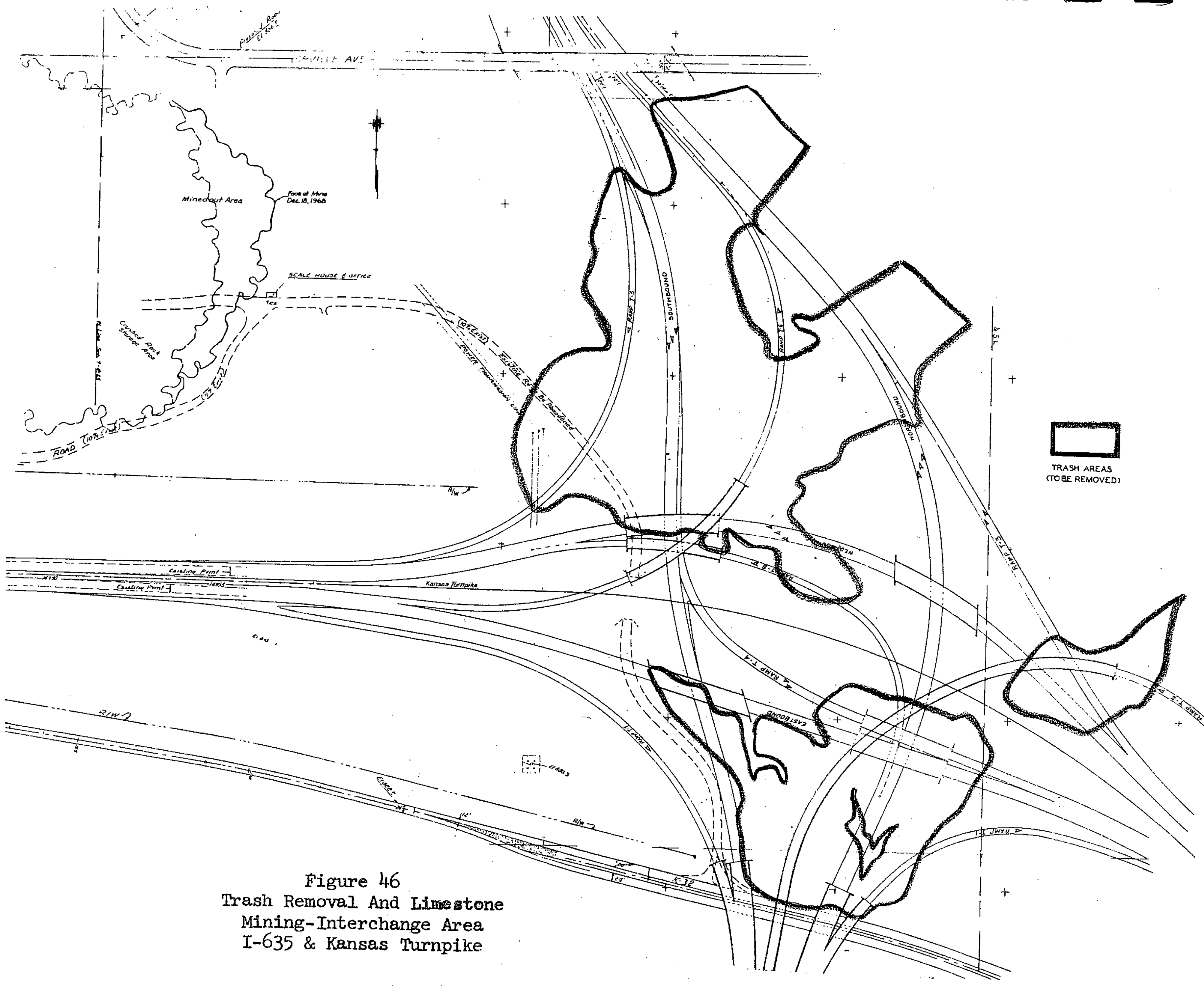


Figure 46
Trash Removal And Limestone
Mining-Interchange Area
I-635 & Kansas Turnpike

When I-635 came into being, a simple cloverleaf was designed for its connection with the Kansas Turnpike. It was known that a small amount of trash would have to be removed. As time passed the design changed several times (Figure 45). It is now known that over 600,000 cubic yards of trash and other waste material, averaging 30 feet in depth, will have to be removed and buried, and this replaced by selected material. Figure 46 shows the area of waste material to be removed. This presents a difficult problem since the location is in a highly populated area. Further complicating the problem is limestone mining operations with a lease to mine the Bethany Falls Limestone from beneath the Interstate Route (Figure 46). This proposes both legal and right of way problems. The legal problem is to determine what extent the Highway Commission will allow quarrying beneath its right of way. Whereas, a right of way problem is the amount of rock to be paid for in the right of way settlement for the areas where mining cannot be allowed. Even though the Highway Commission had made its recommendations on these matters, the final determination will in all probability be determined by the court.

Site M - A Geological Gamble That Nearly Paid Off

The route of the Turnpike is cut through the center of this hill (Figure 47). A drill hole on one side of the hill found loess-like silt with uniform characteristic from top to bottom thought to be Pleistocene in age. A second test hole was drilled on the opposite side of the hill found the same uniform loess-like silt. The width of right of way for slopes in loess-like material had to be considered, that is, to determine between the construction of flatter slopes, requiring more right of way, or using near vertical slopes requiring lesser right of way.

Since uniform loess-like material had been encountered in both holes, it was believed the backslope would stand vertical, and this was shown in the design.

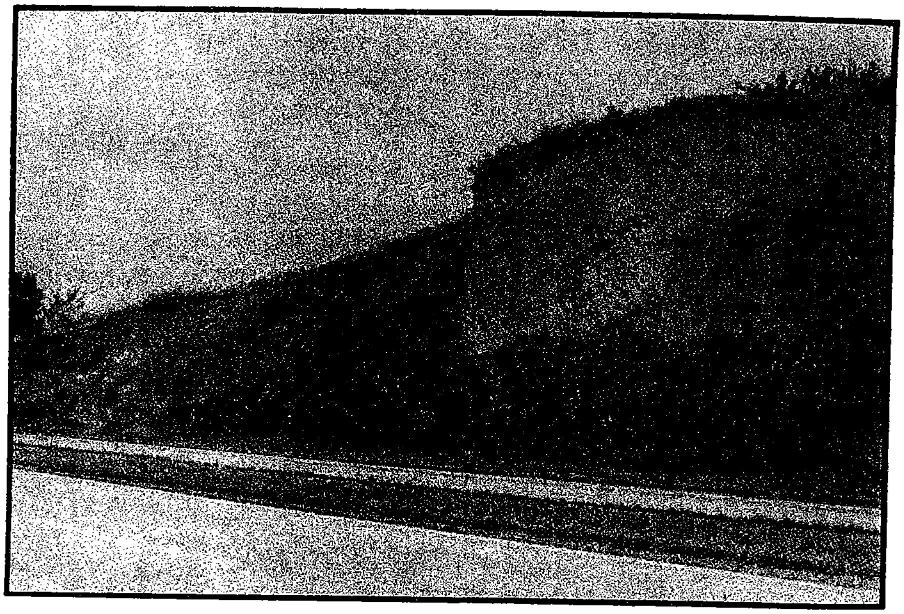


Figure 47
Backslope In Loess-Like Material And Sand On I-70 West Of Kansas City

The Geologist would have won out except that during construction a small pocket of fine sand was encountered in the backslope just above the grade line (Figure 47). In a short time the fine sand flowed out to cause undercutting and thus dropping the material above. But by in large, cutting this material on the vertical was a good gamble, as the slope has now become stable and should stand in its present state for years to come.

Road Log:

Miles

0.0 Jct. US69 & US40
1.1 W. Turnpike Gate
2.0 Site A-2
3.6 Site A-3
8.1 T.P. Service Area
Site B-1 Over Next 20 Mi.
8.7 Haskell Ls. Rt.
11.3 Slide-Lt. Backslope
13.3 Coal-Vinland Mbr.
14.3 Stoner Ls. Mbr.
20.3 Stanton Ls. Fm.
20.7 Plattsburg Ls. Fm.
21.8 Plattsburg Ls. Fm.
22.1 Stanton Ls. Fm.
22.5 Tonganoxie Ss. Mbr.
23.1 Site B-2 Hall Fame
23.2 Bonner Springs Exit
23.8 Bonner Springs Gate
23.9 South on K-7
26.0 Site C-1
26.8 East on K-32
26.9 Site C-2
27.4 Site C-3
29.2 Site D Figure 14
29.6 Edwardsville
30.0 Site D-Figure 15
31.0 Westerville Ls. Lt.
32.5 Site E-1
32.8 Site E-2 Rt.
33.3 Site E-3 Rt.
34.3 Muncie
35.9 Jct. K-32 & K-132
36.5 Site F
36.9 So. on 55th St.
37.4 W. on Inland Rd.
39.0 Cavein Lt.
39.4 STOP 1
39.8 Mining Operation
41.2 Site G North
44.0 Holiday
44.0 W. on Wilder Rd.
44.1 ATSF RR Bridge Clearance 13'-9"
45.2 So. Johnson Rd. 356
46.2 E. to List & Clark Equipment Yard
46.3 STOP 2
47.5 Jct. K-10
48.4 Site H-1
49.2 Site H-2
49.4 Lind Rd.
50.2 Renner Rd.
52.4 Pflumm Rd.
53.9 Nieman Rd.
55.0 North on I-35
57.5 No. on I-635 & Merriam Ln.
57.8 W. on Merriam Ln.
58.6 No. on Antioch
59.4 E. on Shawnee Dr.
60.0 No. on 42nd St.
60.4 E. on Gibbs Rd.
60.6 No. on 38th St.
60.8 E. on Steele
60.8 STOP 3
61.1 So. on 34th St.
62.4 E. on Merriam Ln.
63.0 So. on 24th St.
63.2 E. on I-35
64.2 Ls. Undercutting
65.3 Site I-1
65.5 Site I-2
66.4 STOP 4
67.7 No. on 7th St.
No Trucks Over Kans. R. Bridge
70.0 East on Central
70.3 Site J
70.3 West on I-70
72.5 Site K 18th St.
72.7 Ls. Joints Rt.
74.0 Site L-Proposed I-70 & I-635 Interchange
76.1 Site M

Return to Lawrence

APPENDIX 2

Acknowledgement:

Our thanks to: Inland Underground Facilities, division of Beatrice Food Co. for the visit to the Bethany Falls Limestone Mine; and List and Clark Construction Company for providing the Holiday Gravel Pit for the lunch stop.

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