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CONTENTS

HOMESTAKE'S GRIZZLY GULCH TAILINGS DISPOSAL PROJECT

By

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ABSTRACT

Homestake Mining Company's $12.2 million tailings disposal project is one of the mining industry's most significant environmental projects. This paper describes the role of geological engineering in the location, design, and construction of the dam.

Recent environmental legislation has totally precluded traditional methods of tailings disposal. In order to comply with these federal regulations, Homestake Mining Company is in the process of constructing a complete, environmentally safe disposal system consisting of:

1. A 2-million cubic yard dam
2. A runoff interceptor system
3. A 3-mile pipeline and pumping system
4. A water recycling system
5. An emergency diversion system

In order to validate initial siting, design and construction assumptions, a number of geologic and geotechnical studies were carried out. Areal studies of geomorphology, lithology and structure were used to validate site location. Physical properties of local soil and rock materials were studied to establish quantitative information used for dam design and seepage control.

The results of the geologic and geotechnical studies were incorporated into a finalized project design in 1976. Actual construction was begun in 1976 and is expected to be completed by late 1977.

INTRODUCTION

In the Northern Black Hills of South Dakota, within a distance of fifty miles from Rapid City (Figure 1), one of mining industry's most interesting and significant environmental projects is under construction. This project is Homestake Mining Company's $12.2 million Grizzly Gulch Tailings Disposal Project. As with most construction projects of this size, engineering geology has played a vital role in determining the basic parameters of site selection and design. It is the engineering geology of this project that will be highlighted here.

HISTORICAL BACKGROUND OF THE PROBLEM

The Black Hills of South Dakota has been a significant gold producing district since 1876. Many mines have operated during the subsequent century. But due to the uncertainties and high risks of gold mining only the Homestake Mine remains as an economically viable mining venture.

In the past, disposal of tailings, or waste material, from mining operations, created only minor difficulties. The standard practice throughout the world was to merely deposit tailings in the most convenient place, generally a stream or river.

This practice has been legally brought to a halt through the enactment of the Federal Water Pollution Control Act Amendments of 1972. This legislation set a deadline of 1977 for compliance with standards which totally preclude disposal of industrial wastes into the waters of the United States. Homestake Mining Company was therefore f aced with the problem of disposing of approximately 3,000 tons of tailings daily. These tailings consist of pulverized material which is too fine to be used in the mine as backfill and that, at present, is being discharged into Gold Run Creek. Flow from Gold Run enters Whitewood Creek, which in turn is a tributary of the Belle Fourche River.

It was fortunate that a few years earlier, the federal government had allowed sale of gold on the open market. This action meant a new lease on life for the gold mines of the United States, and for Homestake, the fluctuating, but generally increasing, free market price of gold has made it economically feasible for the company to solve the newer problem of disposing of its tailings in an environmentally acceptable manner, consistent with federal and state regulations.

THE SOLUTION

To comply with these regulations, Homestake Mining Company assigned Dames & Moore the responsibility of developing a plan to dispose of these tailings in an environmentally safe impoundment area.

The plan, approved by South Dakota Department of Environmental Protection, involves the transport of tailings in slurry form to an impoundment area in the Grizzly Gulch drainage basin, located about one mile south of Homestake's Lead, South Dakota, mill site (Figure 2). The impoundment will be created by the construction of a 2-million cubic yard earth dam, 235 feet in height. The dam will be capable of being raised in 50-foot increments, via a modified centerline method, to accommodate the storage of additional tailings in the decades ahead (Figure 3). The ultimate storage capacity is for fifty years of gold production.

The schedule for raising the dam, and for operating the dam prior to raising, is based on a Mine Enforcement Safety Administration, or MESA, requirement that the dam always have a freeboard that will accommodate the runoff from probable maximum precipitation, or PMP, plus a 100-year flood. The freeboard for the first dam increment, now under construction, will be approximately 10 feet.

A 15-foot wide interceptor canal some 4.5 miles long will be constructed around the impoundment area to intercept normal stream flow and runoff from the slopes of the gulch before such waters can enter the impoundment and become contaminated. This water, up to a maximum of 450 cfs, will be discharged through two spillways located just downstream of the dam.

Another key design feature related to the impoundment area is a 20-foot deep, 25-foot wide seepage collection trench, which will be constructed across the gulch downstream of the dam.

Tailings will be transported along a 3-mile route from the mine to the impoundment in one of two rubber-lined 8" steel pipes. These pipes will be installed above the ground on steel or concrete pipe supports for ease of maintenance. One line, handling approximately 800 gpm of slurry at about 600 psi, would be adequate to carry the necessary tailings tonnage. The second line is provided as backup. Eight rubber-lined centrifugal pumps operating in series, each driven by a 75-hp electric motor, will provide the necessary pumping energy. A second bank of identical pumps will be installed as backup.

A barge-mounted pumping station will be located in the free waters of the impoundment area to recycle decant water back to the mine. To prevent freezing of the decant water, this 6" steel pipeline will be buried along the same route traversed by the slurry pipeline.

Should the system experience a loss of power, the pipeline will empty automatically through a borehole to a nonproductive mine area some 1,100 feet below the surface.

GEOLOGIC STUDIES

The geologic data necessary to validate drawing board concepts of dam siting and design were developed by a review of the geology of the area and by a number of field investigations carried out jointly by Dames & Moore and Homestake geologists and engineers.

Geology

The Homestake Mining Company Geology Department of Lead was assigned the task of determining the geologic nature and structure of the rocks underlying the dam site and whether nor not the formation of the gulch was geologically controlled by faulting or other structural discontinuities. This data was critical to the basic question of whether or not the Grizzly Gulch area was a viable location for the proposed impoundment area.

In the early stages of the project rock exposures in the gulch were limited. They were restricted to numerous bulldozer roads recently cut for logging access, a few scattered outcrops along valley walls and more or less continuous outcrops along the crests of the surrounding ridges.

Grizzly Gulch is a youthful drainage (Figure 4). Its course is essentially straight with only minor meanders and steep valley walls. From its junction with Strawberry Creek it trends southwest for approximately four thousand feet, then turns abruptly southeast and follows an essentially straight path to its head. The stream is, at present, actively downcutting through Precambrian, Cambrian and Tertiary rocks.

The major drainage patterns of the Black Hills originated in the early Tertiary as a result of laramide uplift. At this time the area was covered by a nearly complete sedimentary section, thousands of feet thick. composed of rocks ranging in age from Cambrian to Cretaceous. Although uplift has been somewhat sporadic the streams have done downcutting more or less continuously since the Tertiary. Streams now actively downcutting through Precambrian rocks have been superimposed by drainage patterns developed in much younger rocks. Therefore, structural control of the Grizzly Gulch drainage would have to be Paleozoic or younger, probably Tertiary.

Rock Types

Rocks exposed in and adjacent to the site area include Precambrian metasediments and metavolcanics, lower Paleozoic sediments of the Cambrian Deadwood formation and Tertiary age intrusive igneous rocks (Figure 5).

The majority of the gulch is underlain by Precambrian rocks of various types. These rocks occur throughout the bottom of the gulch and up to an elevation of approximately 5,600 feet. Exposures were inadequate and the area covered was insufficient to establish stratigraphy or to determine formational boundaries. For mapping purposes the Precambrian rocks have been divided into three relatively distinct groups. These groups are, however, probably not indicative of formational stratigraphy.

Amphibolites - This group is characterized by green to green gray medium to fine grained rocks ranging from schistose to massive. Mineralogically they consist of hornblende, plagioclase, biotite and chlorite in varying proportions. Quartz is rare and always minor where present. As a group these rocks are considered to be meta-extrusives/intrusives.

Mica Schists, Phyllites and Quartzites - This group is characterized by light gray to gray micaceous rocks ranging in texture from phyllitic, to nearly massive and quartzitic. Mineralogically they consist of biotite and/or muscovite and varying amounts of quartz. Chlorite is rare and always minor where found. These rocks are considered to be metasedimentary in origin and generally reflective of a probably marine environment.

Graphitic Schists and Phyllites - This group is characterized by dark gray to nearly black thinly banded phyllitic rocks. Schistose and nearly massive textures occur but are rare and generally quite thin. Mineralogically they consist of graphite, biotite and/or chlorite, Quartz is rare except in the form of recrystallized chert which commonly occurs as disrupted bands and pods. Pyrite is common and is generally part of the banding. These rocks are probably metasedimentary.

In addition, two distinct but thin, discontinuous and relatively rare Precambrian rock types occur in this area. These are (1) a light gray "streaked quartzite" which appears to be a recrystallized chert, and (2) a ferruginous cherty schist which is generally too well oxidized to determine the mineralogy. They seem to occur in association with the graphitic rocks and are generally limited to the western part of the gulch.

The Precambrian/Cambrian unconformity is not exposed in the area but other data indicates its presence at about the 5,600-foot contour.

Paleozoic rocks exposed in the area are restricted to the Deadwood formation of Cambrian age. Although the Deadwood formation consists of several members only the lower quartzite remains in the Grizzly Gulch area. The quartzite generally consists of a light gray, medium to fine grained massive, relatively pure detrital quartzite with silica cement, and is exposed in quite a few prospect pits and small adits which generally occur 20 to 50 feet above the 5,600-foot contour.

The Deadwood quartzite is overlain by a large sill of quartz feldspar porphyry of Tertiary age. This prophyry has intruded between the quartzite and the "Lower Contact" dolomite member of the Deadwood formation. The porphyry generally forms the present erosion surface and all higher rocks have been stripped away.

Felsic dikes of Tertiary age occur throughout the area and intrude rocks ranging from Precambrian to Tertiary. Many of the dikes appear to approximate the Precambrian foliation, however, crosscutting relationships are not uncommon. Multiple intrusion over a considerable period of time is evidenced by the fact that the dikes exhibit compositional variations ranging from rhyolite to phonolite and that many dikes crosscut other dikes of different composition.

Structure

Lack of an identified marker bed makes detailed study of the Precambrian structure extremely difficult. It is known (Figure 6) from adjacent areas that the Precambrian rocks are complexly folded and that many of the rock types are complexly intercalated and often pinch out over relatively short strike distances. Projection of known structure into the Grizzly Gulch area indicated that the Precambrian rocks in the gulch should be part of the east flank of the Lead Anticline as identified in the Homestake Mine. It is probable that the Precambrian rocks of Grizzly Gulch are folded in a style similar to that of the Homestake Mine. Although structural data is inadequate to allow precise analysis equal area plots of plunge data and foliations indicate that the general structure strikes north or slightly northwest, dips 60° to 70° east and plunges approximately 60° to the southeast.

An equal area plot of fractures measured from the more resistant outcrops fails to reveal a significant trend although several minor maxima do exist.

Structural Control of the Grizzly Gulch Drainage

As previously stated, control of the drainage pattern in the Grizzly Gulch area was probably established during the Tertiary. The stream, which is still actively downcutting in Grizzly Gulch, has, through its history, penetrated a large igneous sill and at least part of the Deadwood formation before entering the Precambrian rocks (Figure 5). The youthful nature of the gulch precludes any adjustment of the drainage pattern to Precambrian structure.

Diamond drilling and field mapping yielded no data indicative of significant faulting. A few very thin, highly-weathered clay zones were found in the road cuts but no correlation between them was apparent and associated structural or stratigraphic discontinuities were not observed.

Several lines of direct observational evidence indicates the lack of faulting in Grizzly Gulch.

1. Two Tertiary rhyolite dikes (Figure 4, area A) located in the center of the area, cross the gulch at shallow angles. They are exposed in several closely-spaced outcrops and no indication of offset has been observed.

2. A large Tertiary phonolite dike (Figure 4, area B) crosses at the upper part of the gulch at nearly right angles, and although not continuously exposed, shows no evidence of offset.

3. In a small area of the gulch (Figure 4, area C) Precambrian rocks are continuously exposed in the streambed and for a few tens of feet up the valley walls. The rock appears to be in place and undisturbed with no evidence of faulting or shearing.

Based on the foregoing discussion, it was determined that, from a structural standpoint, Grizzly Gulch represented a viable impoundment site.

GEOTECHNICAL INVESTIGATIONS

A broad geotechnical investigation program was undertaken at the Grizzly Gulch project site to quantify some of the geologic factors and parameters just covered, and more specifically to obtain the detailed data necessary for the design of the dam, including the actual location of the dam and the materials to be used in its construction.

The principal investigations were performed in the fall of 1975 in three phases:

1. Geologic reconnaissance and geotechnical mapping
2. Subsurface exploration, including diamond core drilling, packer tests, soil borings, test pits, and trenches
3. Laboratory testing.

Supplemental investigations were necessary in the spring of 1976 to evaluate an alternative construction proposal that had been received. This proposal was based on constructing the dam from materials borrowed exclusively from within the reservoir area, instead of using mine waste for the basic material as previously envisioned. The supplemental investigation involved the excavating of additional test pits and the conducting of supplemental laboratory tests.

Geologic Reconnaissance and Geotechnical Mapping

The geologic reconnaissance, the first phase of the investigations performed at the site, involved the mapping of all proposed pipeline routes and road cuts, and all potential slide areas. Also it included the collection of representative bulk samples of residual/colluvial soils, weathered bedrock and alluvial soils.

Geotechnical mapping consisted of the documentation of all existing road cuts in the vicinity of the project site and the mapping of trenches in the reservoir area and along the proposed dam axis. This mapping was integrated with the geologic and structural mapping previously accomplished by Homestake geologists. The geotechnical mapping was used to estimate the thickness of the residual/colluvial soils and the weathered bedrock in order to determine quantities of materials needed for construction of the dam.

Subsurface Exploration

The subsurface exploration program consisted of:

1. Diamond core drilling and packer testing, with eleven holes being drilled.
2. Soil borings, with four holes being augered.
3. Test pits and trenches, with four pits and two trenches being excavated.

For the diamond core drilling, two drilling rigs were used--a Mobile B-50 and a 1946 Joy rig. In each of the holes drilled, coring was started as soon as competent rock was encountered (*i.e.,* rock which would stand open without being cased) and continued to a depth of approximately 100 feet. Drill log data included: rock lithology, fracturing, core recovery, the rock quality designation, weathering, packer test data, and water levels.

To obtain quantitative permeability data for the bedrock, packer tests were performed on all holes using double pneumatic packers with 10-foot spacings.

In addition to diamond drilling, the Mobile B-50 drill rig was also used for soil borings. Four 15-foot holes were bored using 4-inch continuous flight augers. Disturbed samples were obtained at intervals with a Dames & Moore soil sampler and a standard penetration sampler.

Four test pits and two test trenchers were excavated to a depth of 10 feet with a D-6 dozer.

Laboratory Testing Program

The above field exploration program was complemented by an extensive laboratory testing program. The laboratory program was designed to test the performance of "borrow" materials being considered for use in the compacted embankment. Details of the laboratory program are beyond the scope of this paper. However, it included the following tests or determinations: soils classification, gradation analysis, Atterberg limits, permeability, compaction, and triaxial compression testing.

RESULTS OF INVESTIGATIONS

Two major elements of the Grizzly Gulch project were directly affected by the results of the foregoing investigations. First, soil and rock data, including the distribution and physical characteristics of "borrow" materials, became basic control factors in design of the dam cross section or embankment. Second, soil and bedrock permeability data provided control factors for foundation design and seepage control measures.

Soil and Rock Data

Materials within the reservoir area consist generally of a very thin layer of overburden soil overlaying a 20- to 30-foot deep layer of highly-weathered, broken bedrock.

The weathered bedrock grades from highly-weathered, thinly-bedded schist with about 30 to 40 percent soil-sized material to a harder less-weathered schist with less than 20 percent soil sizes. Lab tests indicate that the weathered bedrock will break down during compaction to produce a dense, strong, semi-pervious material which is well suited for use as a shell for a zoned embankment. It is present in sufficient quantites, approximately 1.7 million cubic yards, to construct a dam 235 feet high.

Limited areas in the southwest section of the reservoir contain overburden soils of up to 30 feet in depth. Of special interest is a clay which exhibits low permeability and is an ideal material for the dam's impervious fill. While this clay is limited in quantity, there is an amount sufficient to construct a thin, central core within a zoned dam.

Finally, the existence of several intrusive rhyolite dikes in the vicinity of the reservoir area provided an excellent source of rock which can be processed "on site" into large quantites of the drain material necessary to intercept seepage water which will pass through the upstream shell and core of the dam.

Permeability Data

The packer, or water pressure, tests performed in the foundation bore holes indicated a relatively permeable zone of weathered rock about 50 feet deep. Figure 7 is a geotechnical section along the proposed dam axis showing the relation between the depth and the permeability of the bedrock. The dashed line at a depth of approximately 50 feet indicates the bottom of the high permeability zone. It should be noted that the borings were drilled to a depth of 100 feet, so that at least 50 feet of low or zero permeability rock was tested.

Based upon the above data, a study was performed to determine the amount of seepage that could be expected through, and under, the dam. Using conservative values of permeability and conservative assumptions of water level conditions in the reservoir area, a rate of 200 gpm was calculated.

DESIGN OF DAM

Design of Embankment

Based on the foregoing data, Dames & Moore developed the following cross-section (Figure 8).

The upstream shell was designed to provide adequate strength for end-of-construction and earthquake conditions, and to provide a relatively impervious barrier that would minimize the quantity of seepage through the dam. The highly-weathered rock from within the reservoir met these criteria and since this material was available in abundant quantities, the size of this shell was maximized. An upstream embankment slope of 2 to 1 was utilized.

The downstream shell was also designed to resist end-of-construction and seismic loads. However, rather than providing a relatively impervious barrier, it is desirable that the downstream shell material be comparatively pervious and erosion resistant. The less-weathered bedrock from within the reservoir area satisfied these criteria. A downstream embankment slope of 2:25 to 1 was utilized.

The function of the central core is to minimize seepage through the dam. It will be constructed of the low permeability clays discovered in the southwest sector of the reservoir area. Since the quantities of clay are limited, a relatively thin core, tapering from a maximum of 60 feet at the base to a minimum of 31 feet at the dam's crest, was designed.

Design of Foundation and Seepage Control

In order to intercept and handle seepage through the upstream shell and core, a vertical chimney drain 8-feet wide was designed using rock quarried and crushed on the Grizzly Gulch site. A horizontal drainage blanket was incorporated in the design to safely discharge the seepage beyond the downstream toe of the dam. The ability to produce approximately 100,000 cubic yards of suitable drain material from an on-site quarry eliminated the necessity of hauling gravel from sources as distant as 25 miles over tourist-packed highways, and represented a substantial savings on construction costs.

In order to avoid excessive seepage in the highly-weathered zone, a partial cut-off trench was incorporated in the foundation design. Since a zero-seepage condition would be impossible to achieve, this trench will extend to a depth of about 20 feet below the foundation to block only the most pervious zone. It will be filled with compacted clay, which will extend upward to form the impervious core of the dam. Seepage passing under the cut-off will be intercepted by the aforementioned horizontal drainage blanket which will discharge the flow safely out of the downstream toe of the dam.

Since the overall tailings disposal project is based on the concept of a recycle system, it is necessary to provide a seepage collection trench downstream of the drain blanket to collect all seepage through and under the dam which will then be pumped over the dam and back into the reservoir (Figure 9). The trench is excavated in relatively impervious bedrock and is approximately 20-feet deep, 25-feet wide, and 100-feet long. The downstream face will be sealed with a 2" layer of gunite.

STATUS OF PROJECT AND CONCLUSION

The tailings disposal plan was finalized late in 1975 by Dames & Moore, and approved by Homestake Mining Company for funding of over $12 million dollars. Following approval by federal and state environmental officials in early 1976, bids were taken for major phases of the work. Major contract awards were made for construction of the dam and related reservoir facilities to Summit-Delzer, a joint venture of two Rapid City construction firms, and for construction of the pipeline support system to Brablec Construction Company, also of Rapid City.

Work on the project actually got underway in March, 1976, and is scheduled to be completed in the fall of 1977. Upon completion the Grizzly Gulch Tailings Disposal Project will provide an environmentally safe repository for Homestake's tailings for decades to come, and it may well provide a model for other firms with similar waste disposal problems (Figure 10).

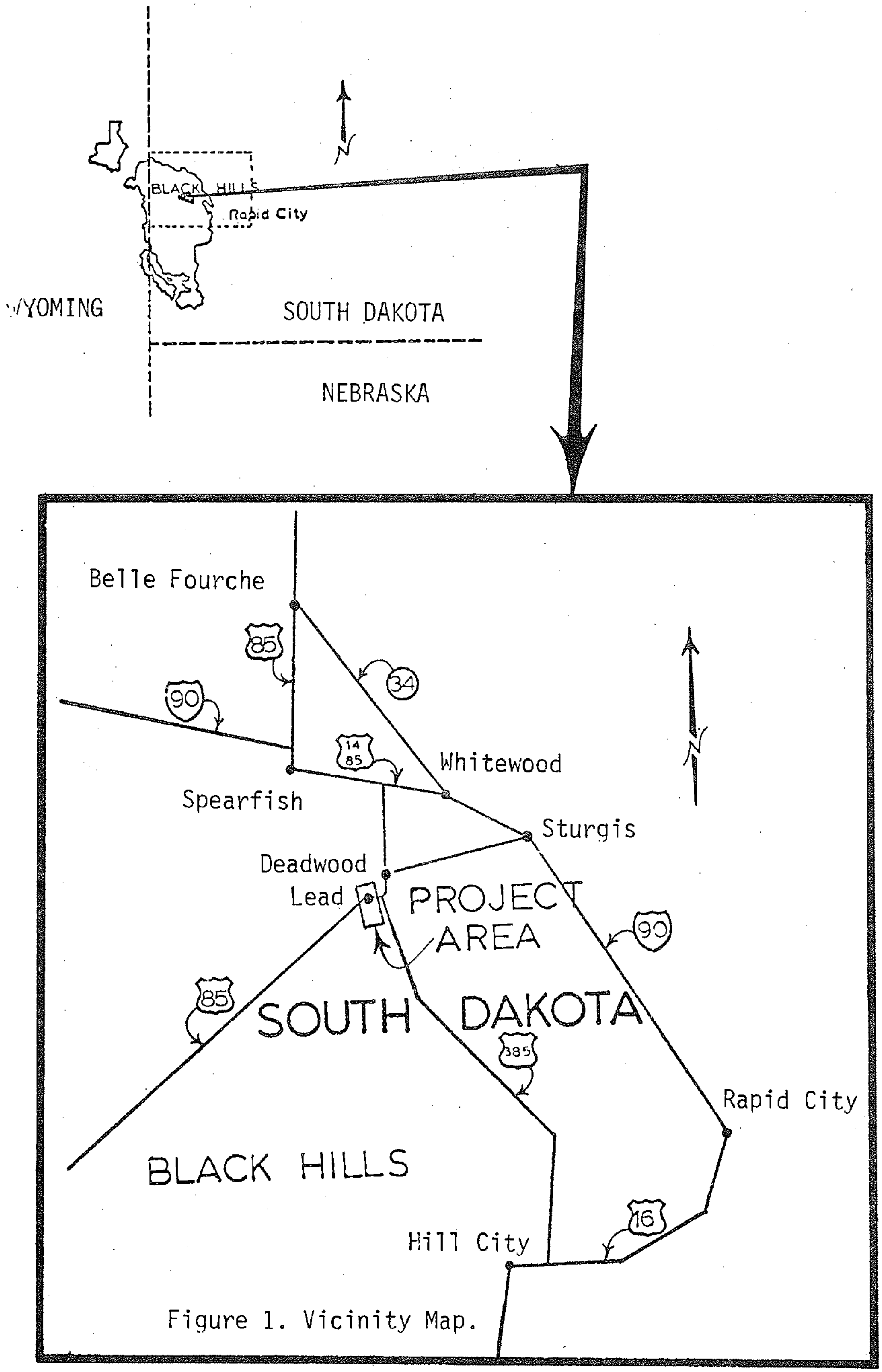


Figure 1. Vicinity Map.

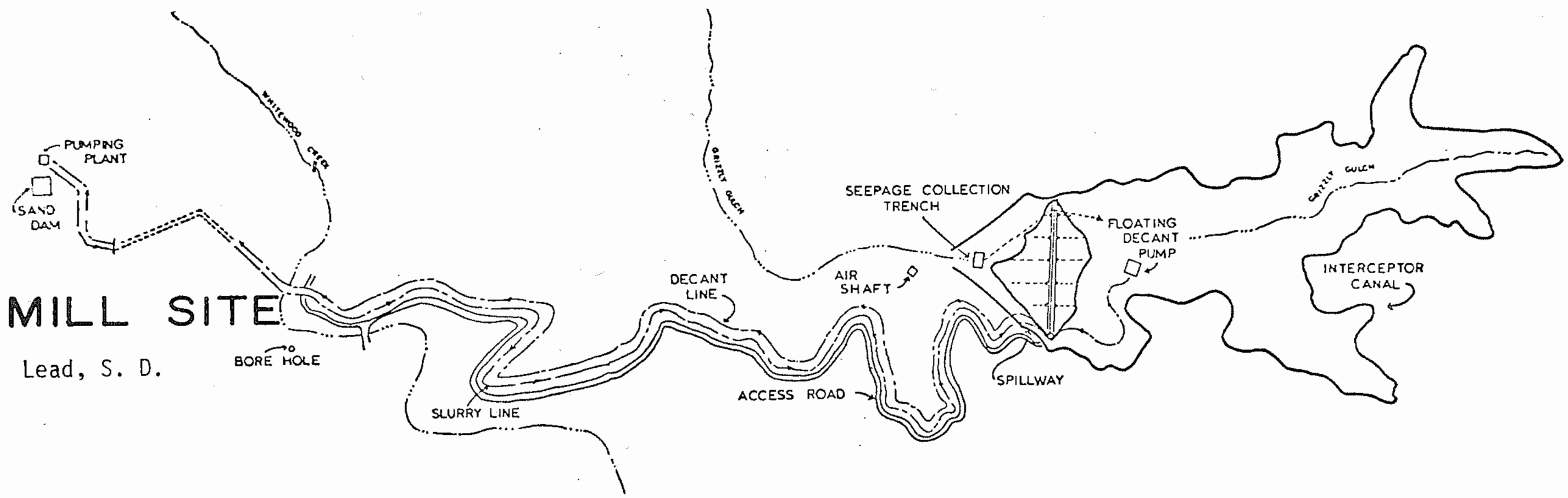


Figure 2. The Plan.

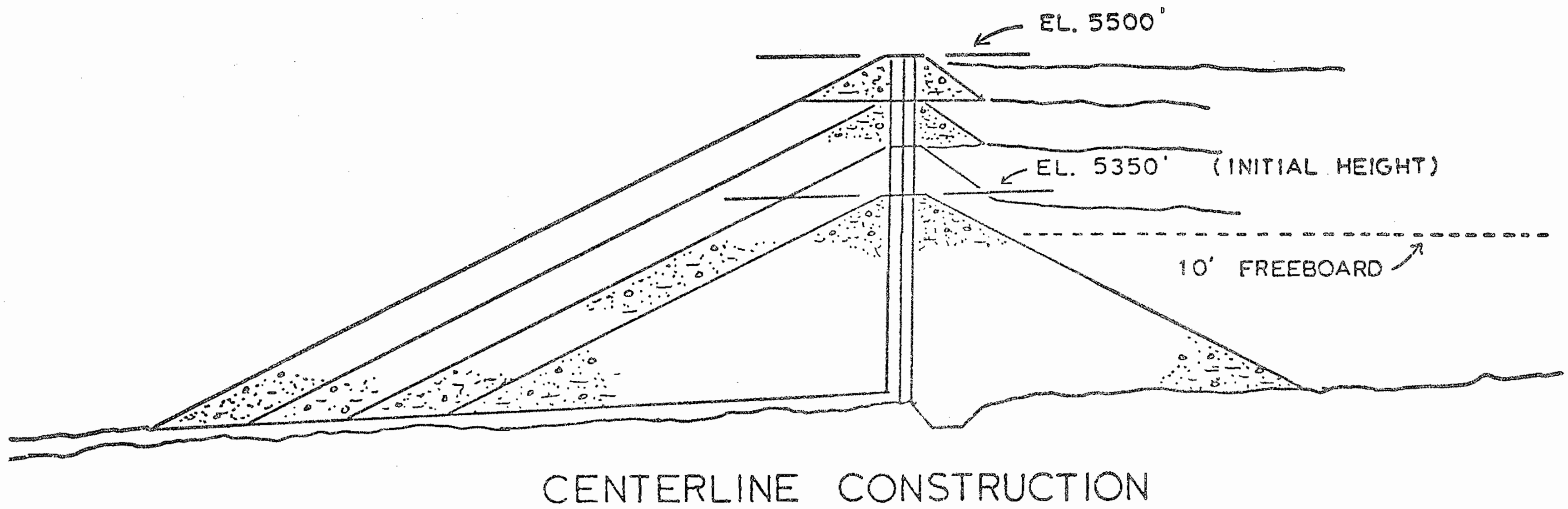


Figure 3. Centerline Construction.

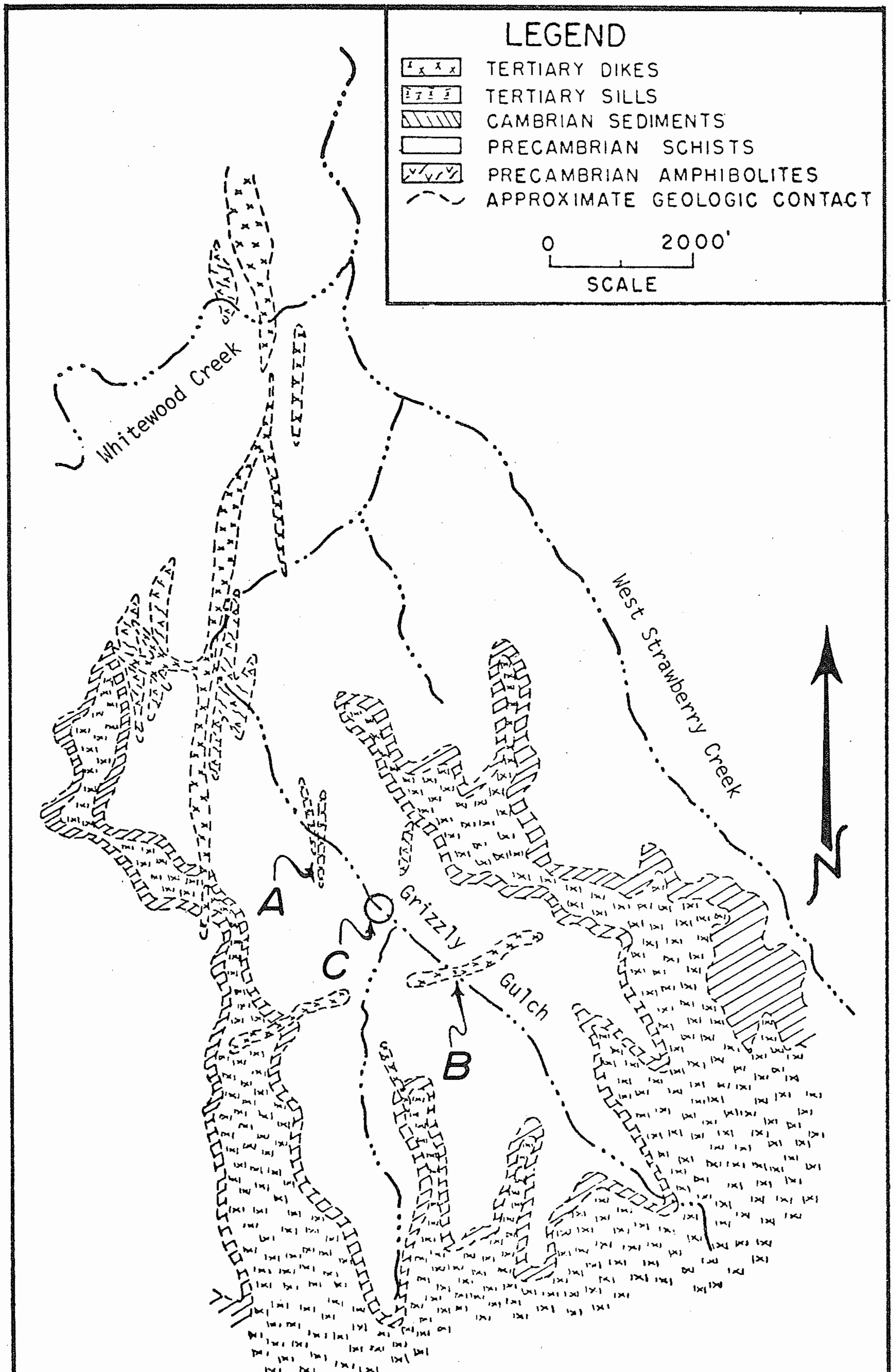
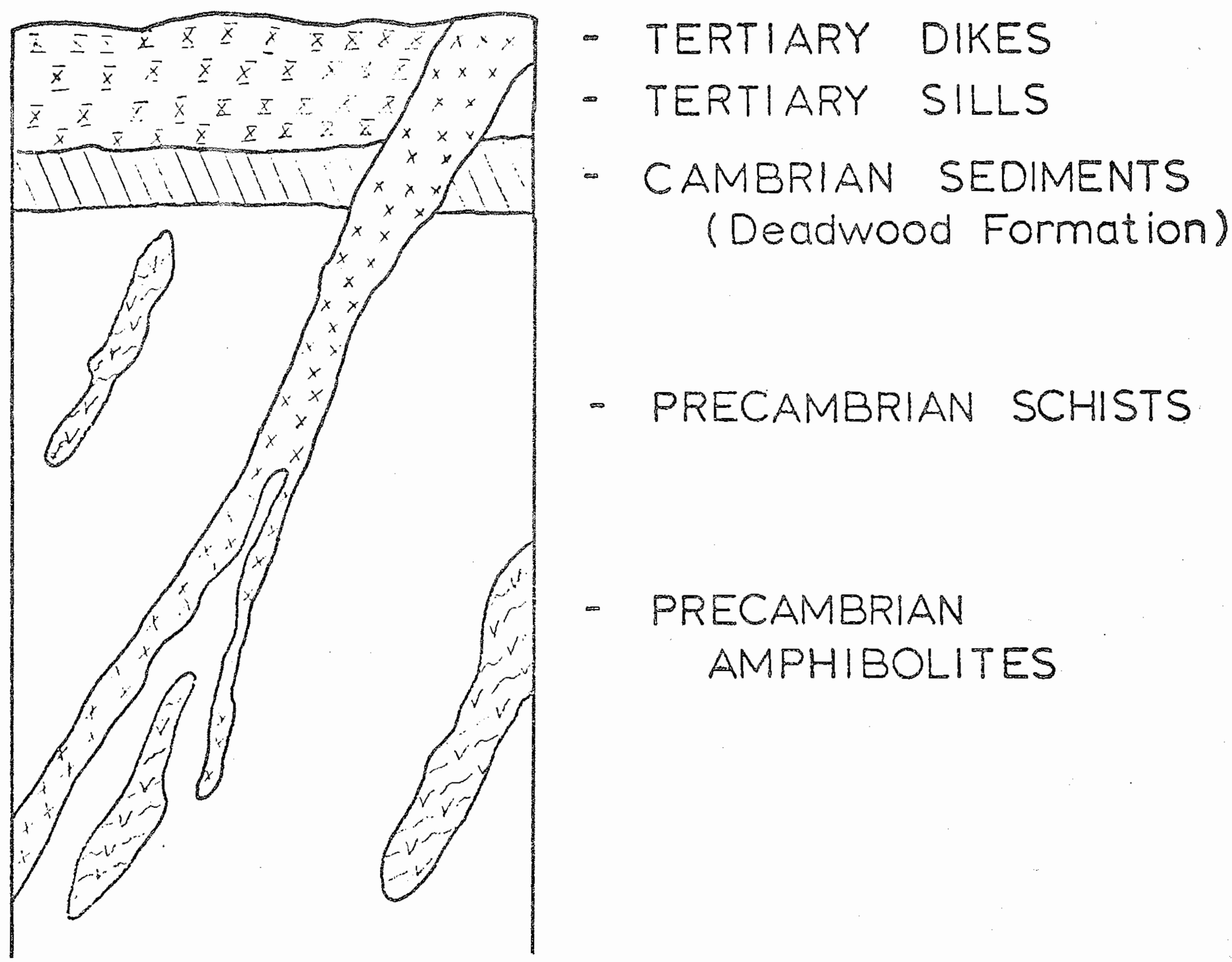


Figure 4. Geologic Sketch Map of Grizzly Gulch.



GEOLOGIC COLUMN IN GRIZZLY GULCH
(SKETCH)

Figure 5. Sketch of Geologic Column in Grizzly Gulch.

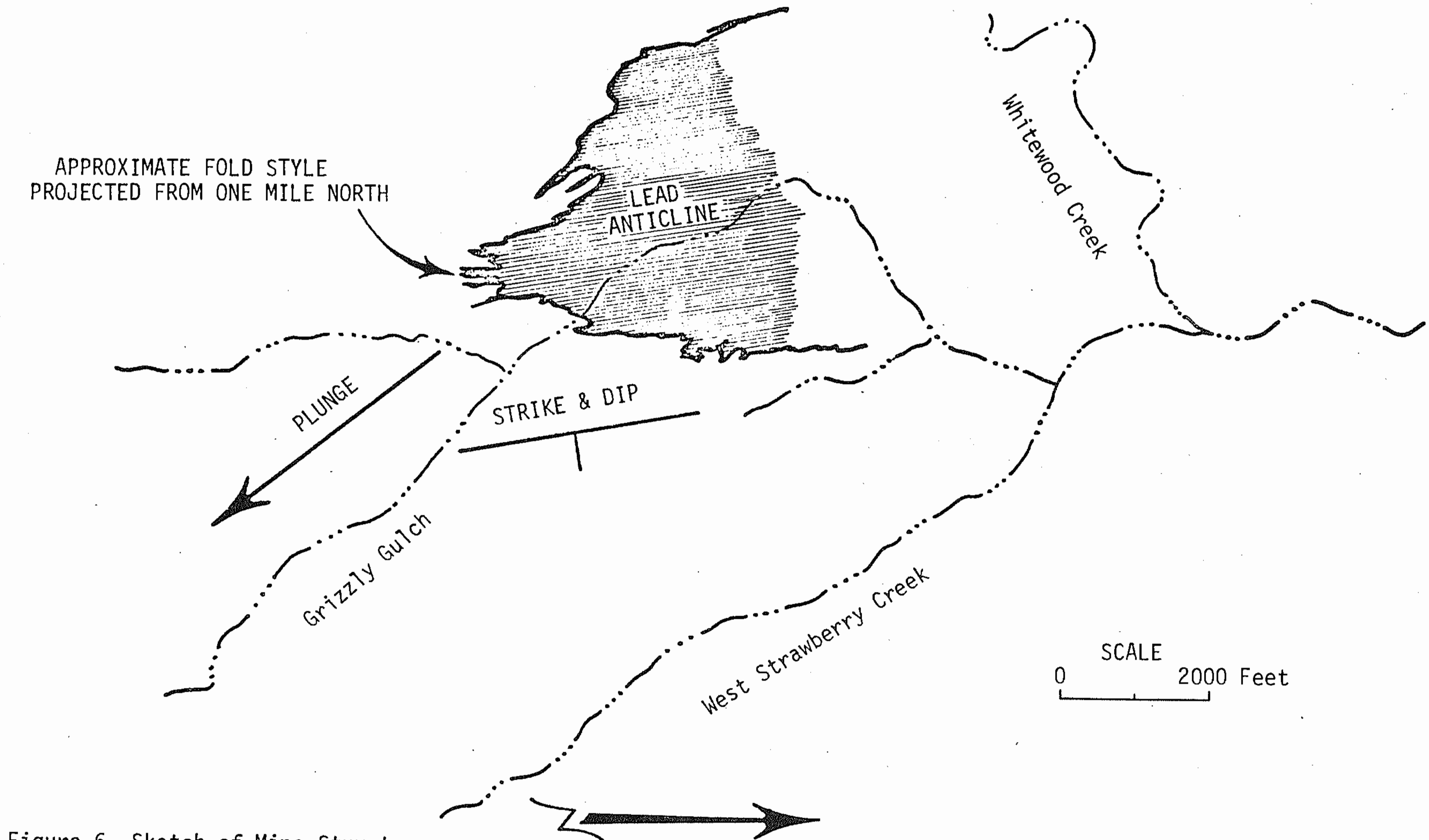


Figure 6. Sketch of Mine Structure.

PROJECTED PRECAMBRIAN STRUCTURE IN GRIZZLY GULCH

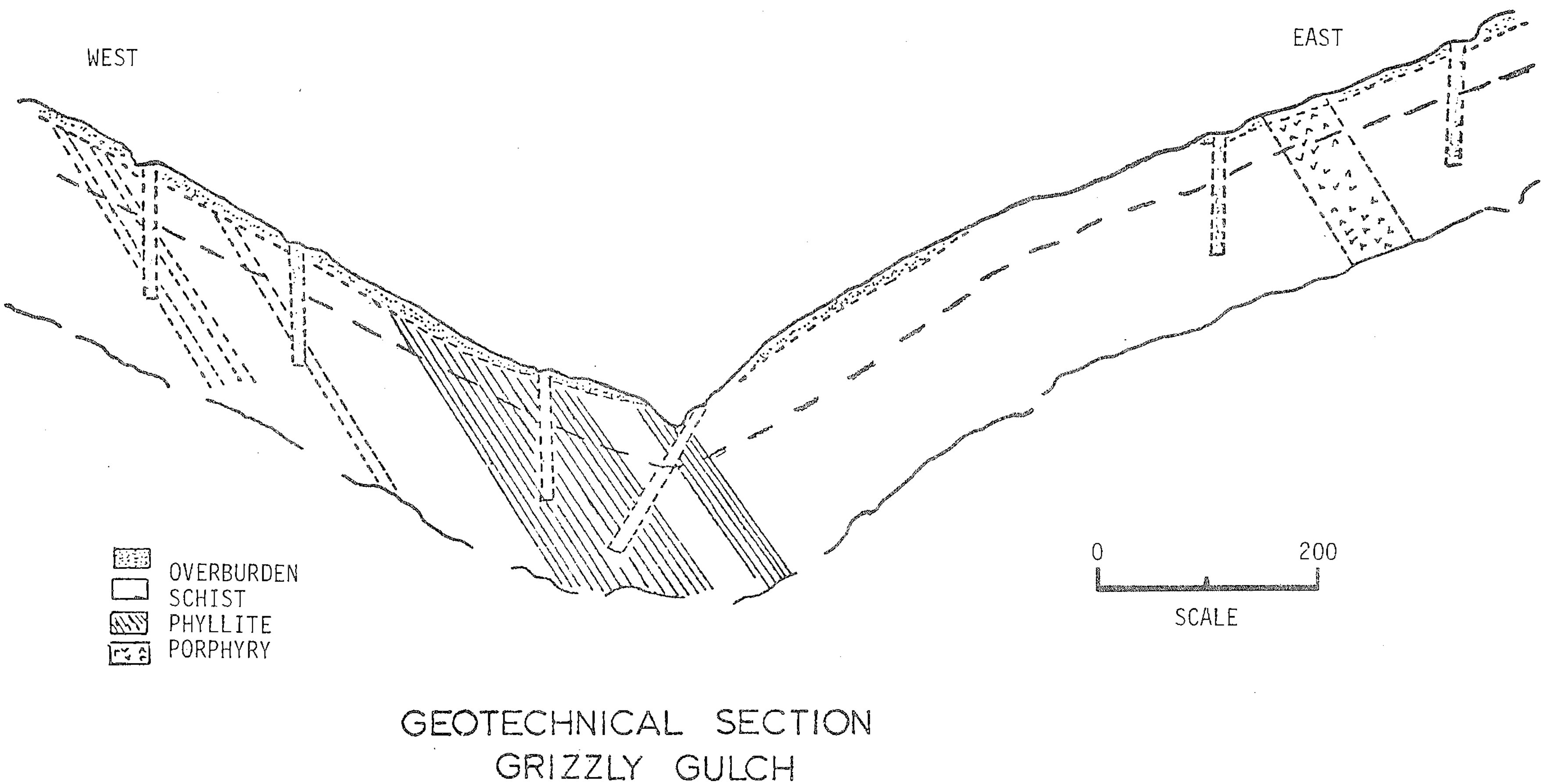
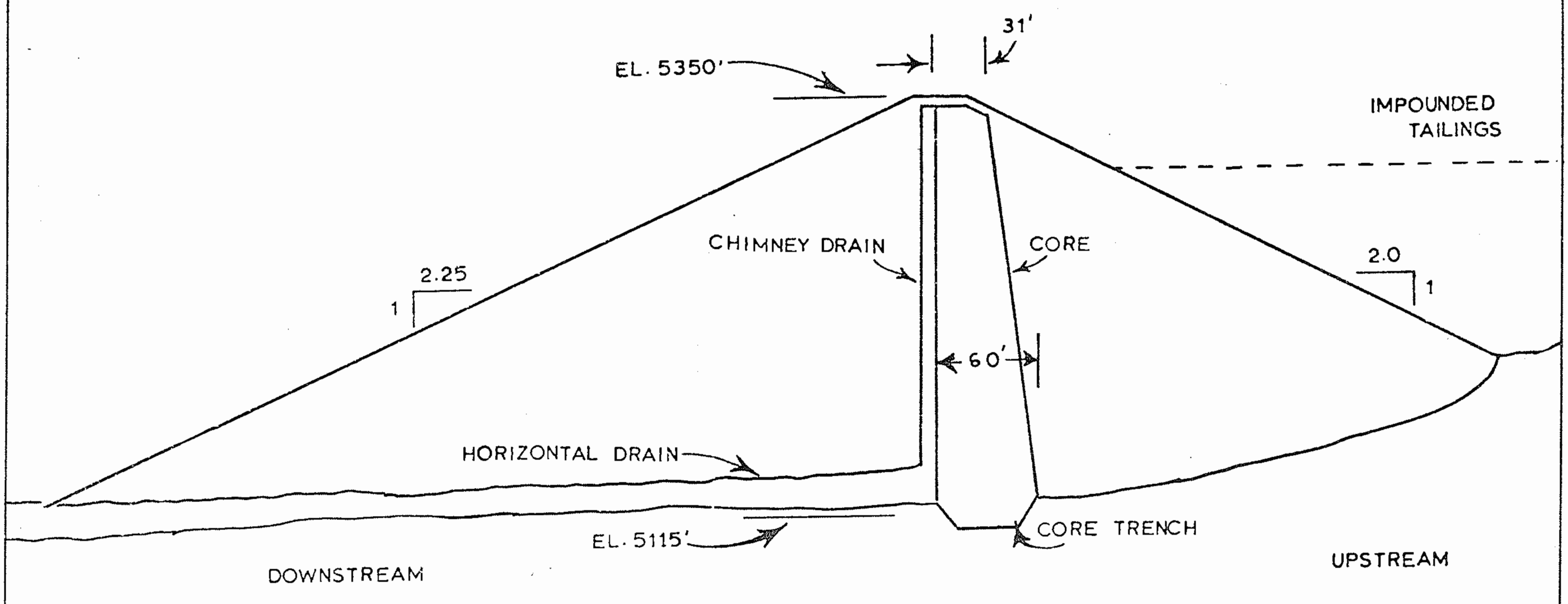


Figure 7. Geotechnical Section Sketch of Dam Axis.

Figure 7. Geotechnical Section Sketch of Dam Axis



SCHEMATIC OF DAM

Figure 8. Schematic of Dam.

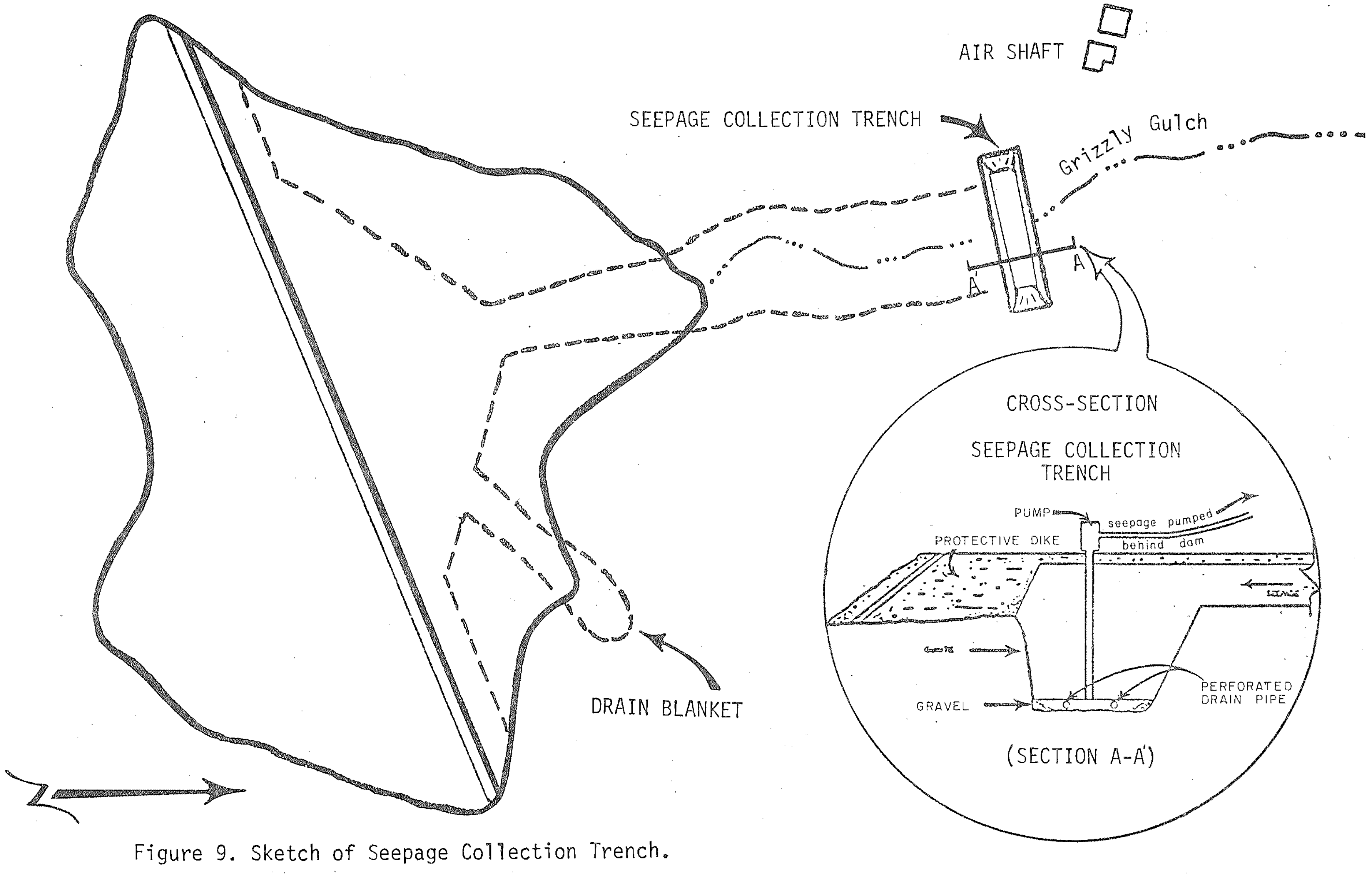


Figure 9. Sketch of Seepage Collection Trench.

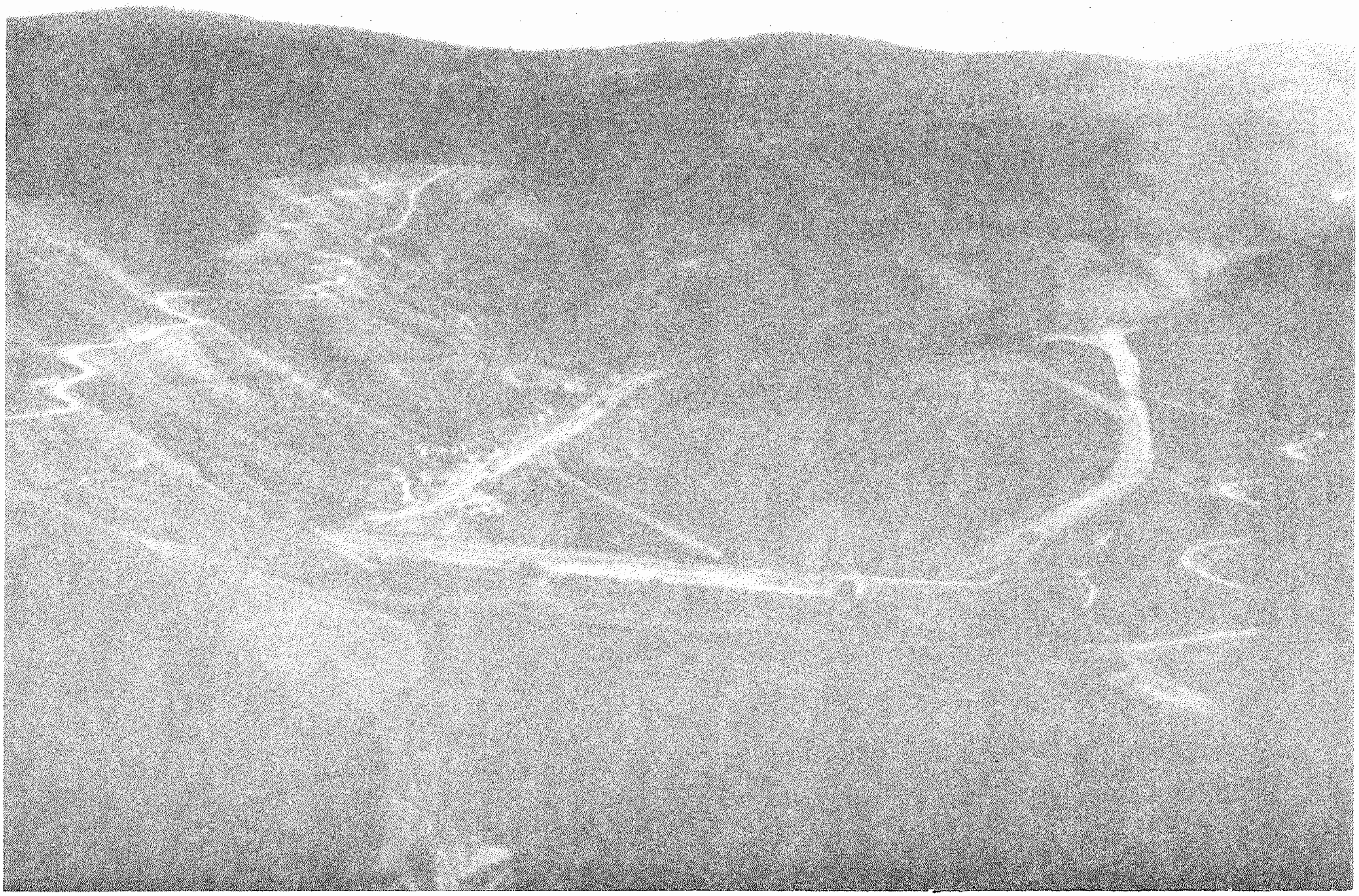


Figure 10. Aerial of Dam. (under construction)

POINT LOAD STRENGTH AND HARDNESS MEASURES OF SHALE

By

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ABSTRACT

The relative abundance of shales through the mid-continent requires their frequent use in compacted highway fills. Where these shales are nondurable, they must be thoroughly degraded and compacted to produce a tight fill. If this is not done, the slaking of the shale pieces will produce large volume changes in the fill, i.e., settlements, and even slope failures.

When the shales combine low durability with relative hardness, special attention must be given to degrading them in the construction process. Among those tests which might be employed to rate the relative degradability of shales are the point load strength and the Scleroscope hardness. These relatively simple degradation measures, particularly the point load strength, combined with simple durability measures, can afford a considerable help in writing appropriate construction standards and specifications.

In this paper we describe the tests and statistical analyses of data gathered from them. While trends and groupings of these data are shown, further data generation will be required to develop a quantitative index of degradability from such tests.

INTRODUCTION

Investigation, design and construction of embankments are generally day-to-day tasks which most of the highway engineering profession are able to accomplish without undue consternation. Only in special cases where exceptional site conditions, design requirements or materials problems exist, do we concern ourselves with unusual measures not currently covered by the state-of-the-art of accepted procedures.

The discussion which follows will deal with such a special problem, the use of nondurable shales as embankment material. Because of the relative abundance of shales through the mid-continent region, the problem of how to treat them is a subject of increasing concern to highway engineers in many states.

As highway embankment material, shales exhibit an array of engineering properties, including some special limitations which may cause considerable difficulty in design and construction. Like most earth materials, shales are typically nonhomogeneous. This lack of homogeneity is of special concern, however, when considering the variable strength and durability of many shales.

The nonhomogeneous nature of shales, their poor strength and weathering characteristics, and unfortunate past experiences have led many engineers to consider shales as "problem material" in embankment construction. This is because the current state-of-the-art does not include adequate means of predicting and evaluating the strength and compressibility of nonhomogeneous and/or non-durable fill materials.

The variability in shale properties, and limited durability, require design guidelines based on a quantitative classification of behavior to enable safe and economical embankment construction. Specifically, tests and evaluation techniques are needed to determine whether a shale deposit can be economically used as embankment material, and if so, to establish design criteria appropriate to the material's characteristics.

A shale research program was initiated at Purdue University several years ago to investigate various aspects of shale classification and behavior which can be applied to embankment construction problems. Early shale research at Purdue included a broad spectrum of conventional soil and rock mechanics tests on samples of several Indiana shales (9). More recent work has investigated the current state-of-the-art in shale embankment construction (5, 10, 30), and compared various index tests and classification systems which describe the durability or resistance to slaking of shales (4, 6). However, this paper will focus on the early results of tests which measured the strength and hardness of shales.

For our purposes, the strength of the material may be evaluated in two distinct parts: the strength of the intact rock, and the strength of the compacted mass. Both types of evaluation must include the effects of various moisture and load conditions.

An analysis of compacted shale strength (stress-strain behavior) has been the subject of a companion study, and will be reported in the future. The strength and hardness of intact pieces will be described here because of its potential value in predicting shale embankment construction and post-construction behavior. Strength and hardness measurements of shale pieces can be directly used to evaluate the following:

a. resistance to excavation,
b. resistance to mechanical degradation during compaction, and,
c. resistance to mechanical degradation due to load, in the compacted condition.

In each of these cases a range in values for various environmental conditions may be used to correlate the behavior of a given type of shale with material encountered during previous work. If sufficient data are available, strength or hardness measurements can indicate the gross homogeneity or variability of the shale within any given formation being considered as fill material. Furthermore, empirical evidence appears to indicate that harder and stronger shales are often more resistant to slaking degradation than soft or weak shales. If this evidence is accepted, then some form of strength test may help in durability classification also.

Current engineering practice does not include any direct relation between the strength or hardness of intact pieces of rock and embankment design parameters. Strength measurements are therefore "classification" tests rather than "design" tests. The different natures of these two designations were defined by Franklin in 1970 (12), as follows:

> "Classification tests are carried out at the stage of site exploration for the purpose of compiling the basic maps showing the nature of the in situ rock, while design tests are carried out at a later stage to obtain information for use in design calculations."

Obviously such information must be readily available before construction starts. To be of any value, strength or hardness data must be easily obtained and easily interpreted. Test results must be clearly understood, and methods of testing must be easily duplicated. With these criteria in mind, initial work was performed to evaluate the usefulness of the Scleroscope hardness and the point load strength (PLS) tests as indices of shale properties and behavior. This report discusses the results of these tests on a variety of Indiana shales.

SCLEROSCOPE HARDNESS TESTS

Several kinds of hardness tests are currently used to provide a measure of strength or resistance to yielding for various materials. In geological engineering, abrasion and dynamic hardness measurements, typified by the Mohs and Scleroscope tests, respectively, are the most common (8).

Dynamic hardness tests provide a limited measure of several material parameters, primarily the elastic and plastic resistance to deformation. Although theoretical correlation is missing, empirical evidence relates the Scleroscope hardness number to several rock properties such as toughness, resilence, strength and elasticity (8). Various investigators have used Scleroscope hardness tests as an indication of workability, toughness, unconfined strength, fabric variability, drillability, and abrasion resistance, for various rock materials (8, 13, 19, 22, 27). Statistical correlations have been developed to indicate those properties and their variability with respect to Scleroscope hardness, and are reported in the literature.

As far as we know, prior to this investigation there was no attempt to use the Scleroscope hardness test to discriminate among various Indiana shales and their engineering properties. However, hardness tests have been used to indicate the strength of various rock materials in previous unrelated studies.

On this basis a testing program using a Shore Scleroscope, model D, was initiated on shale samples provided by the Indiana State Highway Commission (ISHC).

Figure 1 shows the Scleroscope apparatus needed for this dynamic rebound test. Performance consists of dropping a diamond tipped steel rod onto the surface of the test specimen from a fixed height, and measuring the maximum height of rebound of the rod. The operation of elevating and releasing the rod is an automatic sequence which accompanies the manul rotation of a control knob on the Scleroscope apparatus. This same sequence automatically fixes the maximum rebound height of the rod after it strikes the specimen. Counter-rotation of the control knob causes this rebound height to be recorded on a dial gauge. The dial gauge readings are in Scleroscope hardness units, which range from 0 to 120.

Following a suggested standard test procedure, several hardness measurements were made on each shale specimen and an average hardness was reported for each test series (28). According to the manufacturer, the Shore Scleroscope is accurate for specimens with a minimum thickness of 0.005 inch to 0.015 inch, depending on the material properties. This indicates that the hardness of individual lamina in a single shale specimen could be measured and contrasted. The averaging process tends to give a gross hardness measurement however.

Initial Scleroscope hardness tests on shale pieces indicated the following test constraints.

1. The measured hardness was very sensitive to small discontinuities, or "roughness", on the surface of the specimen being tested.
2. The measured hardness for single samples tended to increase as the samples dried out, thus ruling out a single characteristic hardness number for any given type of shale.

Specimens were selected for Scleroscope testing on a random basis from material on hand for tests on eight types of shale. The selected specimens were all about two inches square by one inch thick, with thickness being measured perpendicularly to the bedding planes. The samples were roughly squared by sawing, and then the test surfaces were smoothed with a wood rasp and/or coarse quartz sandpaper. The hardness was only measured perpendicularly to the bedding planes. Final sample preparation consisted of continued sanding with successively finer grades of quartz and emery paper until the shale surface to be tested was polished smooth with no flaws or discontinuities visible or discernible by touch. The sample was then weighed for future water content determinations, and tested.

Each shale surface was tested from nine to twenty-four times, and an average hardness was calculated from the results of these tests.

Following testing, the shale specimens were placed in an oven set at 40°C. Tests were repeated at intervals of 24 and 48 hours, after which the shale specimens were placed in another oven set at 105°C, and tested after a further 24 hours. Prior to each test series, the shale specimens were removed from the oven and allowed to cool in a desiccator jar to prevent moisture gain. When cool, the specimens were weighed and then tested. In this way, each specimen was tested a number of times over a range in water contents. Final tests were made after the specimens had reached a constant weight in an oven set at 105°C. This drying process took as long as 45 days in some cases.

This procedure was altered somewhat for tests on samples of nine other shales. Only one or two specimens each of these shales were available, and no attempt had been made to store them at their in situ moisture content. A reduced drying range was thus used for these shales.

In all, more than 5,000 single hardness measurements were made on samples from seventeen Indiana shale formations. Specific problems encountered and results achieved, are discussed in detail by Bailey (2).

With some restrictions, the Scleroscope hardness data seem reasonably consistent, except at the zero moisture content condition. Specific test results show that the variability in measured hardness increased as the shale specimens became drier.

Figure 2 shows the change in hardness with drying for all of the shales tested. There is a general increase in shale hardness at lower levels of moisture content. The deterioration of the shale, especially at or near zero moisture content, appears to be responsible for the increased data scatter in this range.

The tendency for lower hardness values at higher levels of moisture content is also clearly evident in the test results for each of the shales considered individually. An example of this is shown in Figure 3, which presents the Scleroscope test results for a single shale type.

Multiple regression analyses were performed to determine the correlation between Scleroscope hardness and moisture content for the various shales. No significant relation between hardness and moisture content was found for samples of seven types of shale, however, significant correlations were found for ten of the shales tested.

For these ten shales, the hardness test results all show the same general pattern of a proportional decrease in hardness of one to four times the increase in moisture content.

Examination of the calculated values of Scleroscope hardness from the hardness vs. moisture relations indicate the shales tested fall into two general hardness groups. The first group, with an average oven-dry hardness of about 32, is typified by four of the shales, and the second group, with an average oven-dry hardness of about 52, consists of two shales. The discrimination between these two groups of shales is clearly seen in Figure 4, which shows a frequency distribution plot of the test results segregated in hardness number increments of 5.0.

Figure 4a shows the distribution of average hardness computed from the regression equations just mentioned. Figure 4b shows the distribution of the average values of hardness actually measured for each of the shales tested. In both cases a segregation of "hard" and "intermediate" shales is evident. Such a segregation may be a useful tool for classifying shales on the basis of hardness. However, more data could tend to "fill out" the frequency distributions leading to a normal or Gaussion distribution for the data.

Figure 5 shows the mean values and ranges of the hardness data measured in all of the Scleroscope tests on oven dried samples. The data show considerable overlap and large scatter for some of the shales, and this suggests that the Scleroscope hardness test may have no value as a classification index unless a large enough quantity of data is available to allow statistical rejection of extreme values.

Our preliminary conclusion is that the validity of a hardness index or of a correlation between Scleroscope hardness and the other properties of Indiana shales cannot be estabilished without further testing and analysis. It is notable, however, that results of Scleroscope hardness tests on seventeen Indiana shales show a significant trend towards greater hardness at reduced moisture contents. While such a tendency may offer positive suggestions towards future construction treatment of shales, it is impossible to isolate specific or quantitative rules-of-practice based on Scleroscope hardness measurements at this time. Further testing and analysis may be of value in this respect. However, difficulties encountered in sample preparation and the large range in data scatter indicate this will be a difficult problem.

POINT LOAD STRENGTH TESTS

Many engineers are certain to be curious about why we selected the point load strength (PLS) test for our shale study, when there are already several more widely used tests of rock strength. Unfortunately, all strength testing procedures have inherent deficiencies and a brief review of these may be helpful in explaining our choice.

Most of the Indiana shales used in this research are not suitable for unconfined or triaxial testing on intact pieces. Diamond drilling and cutting equipment which is generally available uses water to cool the tool face and remove cuttings; this water also causes slaking of the sample. Experience on this project also demonstrated the impossibility of maintaining an original moisture content during the time required for lapping the ends of each sample.

Other tests present disadvantages in terms of the quantity or quality of sample material required. Since initial field investigations frequently consist of diamond drill coring or hand collected bag samples, tests which require large amounts of material may be impractical to perform. Standard uniaxial compression tests require only small samples, but in general each sample must be free from gross defects such as cracks, healed fractures, or intrusive veins. Often core recovery is so poor that it is impossible to obtain a sufficient number of defect-free test specimens.

In order to avoid the problems of special sample preparation, "irregular lump" strength tests have been devised. Hobbs (15) and Franklin (12) have described developmental work by Protodyakonov. This work led the International Bureau of Rock Mechanics to recommend a standard test procedure in 1961. The first indirect tensile strength test was reported by Carniero and Barellos in 1953, the so-called "Brazilian Test." The combined influence of these two test methods led to the development of the point load (indirect tensile) strength test (12, 25). This test has several attributes which will be described in detail.

The point load strength test (PLS test) does not require any particular sample preparation. Specimens as small as 10 cc. (about 30 grams) were tested quite satisfactorily, as were much larger pieces, during this investigation. The upper size limit of sample pieces is governed by the capacity of the loading apparatus. Some of the tests reported herein were conducted on shale pieces whose smallest dimension was in excess of two inches across.

Furthermore, test results do not seem dependent on the sample shape. While various theoretical analyses of indirect tensile failures have been developed for differently shaped loaded areas (11, 12, 14, 18), the differences are small in relation to the effects of nonhomogeneity, anistrophy, and non-elastic behavior in the rocks tested.

Another favorable attribute of the PLS test is the short time required for each test. Since a single test can be completed in about ten minutes, no special attention to preserving initial moisture content is required. This also enables a large number of tests to be completed at low cost in a relatively short period. This is important because the wide variability in shale materials frequently necessitates statistical interpretation of data. Increasing the number of strength tests will improve such interpretations.

Other indirect measures of rock strength have been attempted. Chapman (4) has described the use of the Schmidt rebound hammer with Indiana shales, and D'Andrea et al. (7) have attempted to predict the strength of rocks using a variety of other properties. Both of these studies have reported their own particular problems.

Figure 6 is a sketch of the PLS apparatus. The base of the device rigidly supports the lower platen and the two guide bars. The upper platen is mounted at the end of the loading ram which, with the guide plate, slides freely on the guide bars. A dial gauge, reading to the nearest one thousandth of an inch, measures the height of the guide plate above the base plate. The platens are spherically tipped circular cones of case hardened steel. The platen design conforms to the proposed standards of the International Society for Rock Mechanics (16).

To conduct a PLS test, a piece of shale is placed between the upper and lower platens. The weight of the upper platen and load ram hold the specimen in place. Although samples were approximately centered on the lower platen, no particular care was required to maintain their placement. An initial reading of the gauage indicates the guide plate height, and by calculation, the sample thickness between the platen tips is determined.

The loading force acting through the platens compresses the shale specimen, causing tensile stresses to develop perpendicularly to the loaded diameter. The load was applied by an electrically driven compression testing machine, at a constant rate of strain of 0.01 inches per minute. The load resistance of the sample was monitored through a 5,000 pound capacity, SR4 Type, load cell. Stress and strain were monitored continuously with a dual channel strip chart recorder.

Several methods for calculating the tensile stress at failure have been proposed for PLS (7, 14, 17, 20). These analyses (theoretical and empirical) all include a term equal to the ratio of ultimate compression load and the square of the original sample thickness. Various coefficients, most commonly based on specimen geometry, have been proposed to "correct" this term. Franklin (12) and Jaeger (17) review and compare some of these analytical treatments of the tensile stress. Poulos and Davis (24) describe the stress field resulting from an ideal point load. Roark (26) presents stress equations for the case of a spherical load platen, similar to the spherically tipped conical platen actually used.

Following the conclusions of Franklin (12), PLS results are reported herein as the ratio of maximum compressional load and the square of the initial platen separation (initial sample thickness). No correction factor is included in these data.

Results of PLS tests in this investigation have been considered in two categories because of the effects of specimen moisture. The point load strength of oven-dried specimens (zero moisture content) provides a reference value from which to compare directly the properties of different shales. PLS tests on specimens of the same shale at various moisture contents show a range in strength which would confuse any comparison among different shale types. However this variation may be significant in evaluating the degradation potential of any particular shale.

The first series of PLS tests was performed on Scleroscope test specimens which had been oven dried to a constant weight.

Several of the shales tested in an oven-dried condition show a wide range in strength. This variation may be due to one or more of the following:

a. natural nonhomogeneity of the test specimens,
b. effects of size and/or shape of the test specimens, and/or
c. non-uniform changes in the shale due to the oven-drying process.

Based on the results of previous studies, it seems that the effects of size and shape are the least likely cause of the data scatter.

A gradual oven-drying procedure was used to remove the moisture from each specimen before testing. Visual examination of smooth sanded surfaces on the test specimens showed gradual deterioration of the shale as moisture was removed. This deterioration appeared as flaking, chipping, and the formation of hairline cracks. The extent of deterioration was not constant for all of the specimens observed. Observation of the formation of interior cracks or other deterioration as the drying process extended into the interior of a specimen was not possible. Since the PLS tensile failure is initiated in the interior of the test specimen, the presence of random internal flaws, similar to the drying induced surface flaws, may explain the data scatter.

Nine shales were subjected to PLS tests at a variety of moisture contents. Experience gained from long-term soaking tests indicated the impracticality of artifically inducing high moisture contents in the shales. The prospect of inducing internal flaws through oven drying also prohibited artifically reducing the moisture contents. Thus only shales which showed some variation in water content in the storage containers are included in this category. The shales tested had been stored for 12 to 36 months under "equilibrium" conditions, and this was considered suitable for comparing the change in strength with moisture content. Sample storage procedures and the relative moisture variations are described by Bailey (2). To reduce the drying time required in measuring water content, the test specimens were broken up with a hammer following PLS testing. This enabled relatively fast (2 to 5 days, as opposed to 30 to 40 days) oven drying, and insured that the water content of each specimen could be accurately determined.

Of the nine shales tested at various moisture contents, four showed recognizable linear relations between water content and point load strength. With the exception of one very anomalous value, the PLS data for a fifth type of shale also showed a linear relation with moisture content. For these five shales, there is a clear trend of decreasing strength at higher moisture contents. The PLS test data for four other shale types showed too much scatter or too small a moisture content range for any trend to be recognized.

Despite the fact that not all of the shale types individually showed lower strength values at higher moisure contents, this trend is very evident when all of the shale samples are compared together. Figure 7 shows the results of 190 PLS tests on the shale samples previously mentioned. The data points to the left of the vertical dashed line represent oven-dried specimens, the others represent the various storage moisure conditions. It appears that oven drying tends to both, (a) increase the shale strength by reducing the moisture content, and (b) decrease the strength through alteration of the material structure. This alteration is principally evident as formation of hairline cracks and surface deterioration. The relation between strength reduction due to artificial drying, and the rate of drying, was not investigated.

The change in shale strength with change in moisture content has been previously noted by Bieniawski (3) and others (1, 23). For compaction of shale, an increase in moisture content on the surface of pieces was reported to improve breakdown on one project (29).

Simple regression analyses were performed on the test data for the four shales where a linear relation between strength and moisture content seemed evident. Computerized linear regressions enabled the determination of a "best fit" through the plotted data for each shale type. The second power of Pearson's correlation coefficient R^2, was used to evaluate the accuracy of this line in describing the observed data (21). Because of the range in PLS values for the zero moisture condition, a question arose regarding how to treat these data. Regressions were performed using the non-zero moisture content ($w > 0$) test data, all of the test data ($w \geqslant 0$), and the non-zero moisture content test data with the mean value of the zero moisture content test data, for the four shales which showed a linear relation between strength and moisture content.

These results illustrate the extensive variability in the oven-dried PLS samples. Linear regressions performed on the four shales resulted in higher R^2 (i.e., better linear correlation) values for the $w > 0$ condition than for $w \geqslant 0$. Using a mean value of the $w = 0$ results, with the $w > 0$ test data, resulted in still higher R^2 values for three of the shales. The $w > 0$, and mean value for $w = 0$, data analysis for the fourth shale resulted in an R^2 value greater than the $w \geqslant 0$ condition, but less than the $w > 0$ condition.

As expected, the PLS test provided a relatively quick and convenient means of assessing the strength of shale pieces. Although the test is not extensively used, and lacks a uniformly agreed upon theoretical grounding, it appears sensitive to small variations in the strength of shale pieces. This enables strength testing of relatively weak and non-durable shales which are not amenable to more conventional laboratory procedures. The tests described, required no particular sample preparation. In other words, the PLS test is equally suitable for both core samples and grab samples.

The PLS test results obtained in this investigation showed a wide scatter for shales tested after oven drying. Three possible explanations have been referred to, and are described more fully by Bailey (2). About one-half of the data acquired from non-oven dried PLS specimens show fairly good linear correlations with shale moisture content as the independent variable. The combined results of all PLS tests show a definite trend of decreasing strength with increasing moisture content. This trend agrees with reports from other investigations (1, 3, 23, 29), and with the results of the Scleroscope hardness tests previously described.

The point load strength of shale pieces can be quickly and easily measured. PLS tests can be performed on relatively small pieces of shale, hand-collected specimens or pieces of drilled rock core, enabling the results of a number of PLS tests to be conveniently obtained and subjected to statistical analysis. The natural nonhomogeneity of most shales requires such statistical analysis of data, whether they are to be used for classification or design purposes.

In summation, point load strength testing appears to be an adequate method of measuring the strength of shale pieces. This strength measure may prove to be correlatable with such embankment construction parameters as ease of excavation and degradation due to compaction. Further investigation of these aspects should prove beneficial to the state-of-the-art of shale embankment construction, and results will be reported when we have further experimental evidence.

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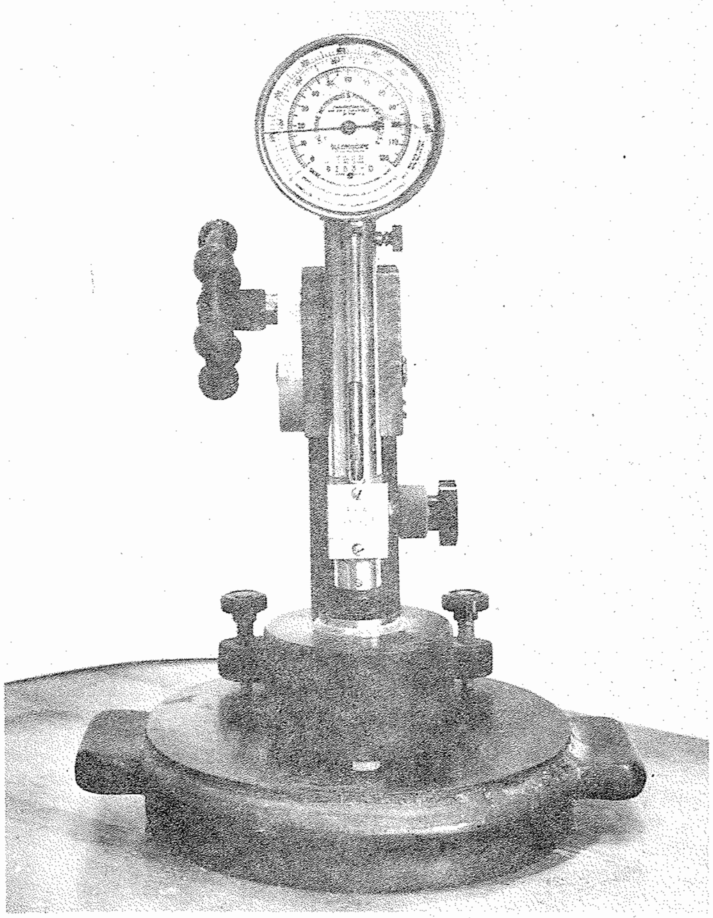


FIGURE I. SCLEROSCOPE (MODEL D) APPARATUS.

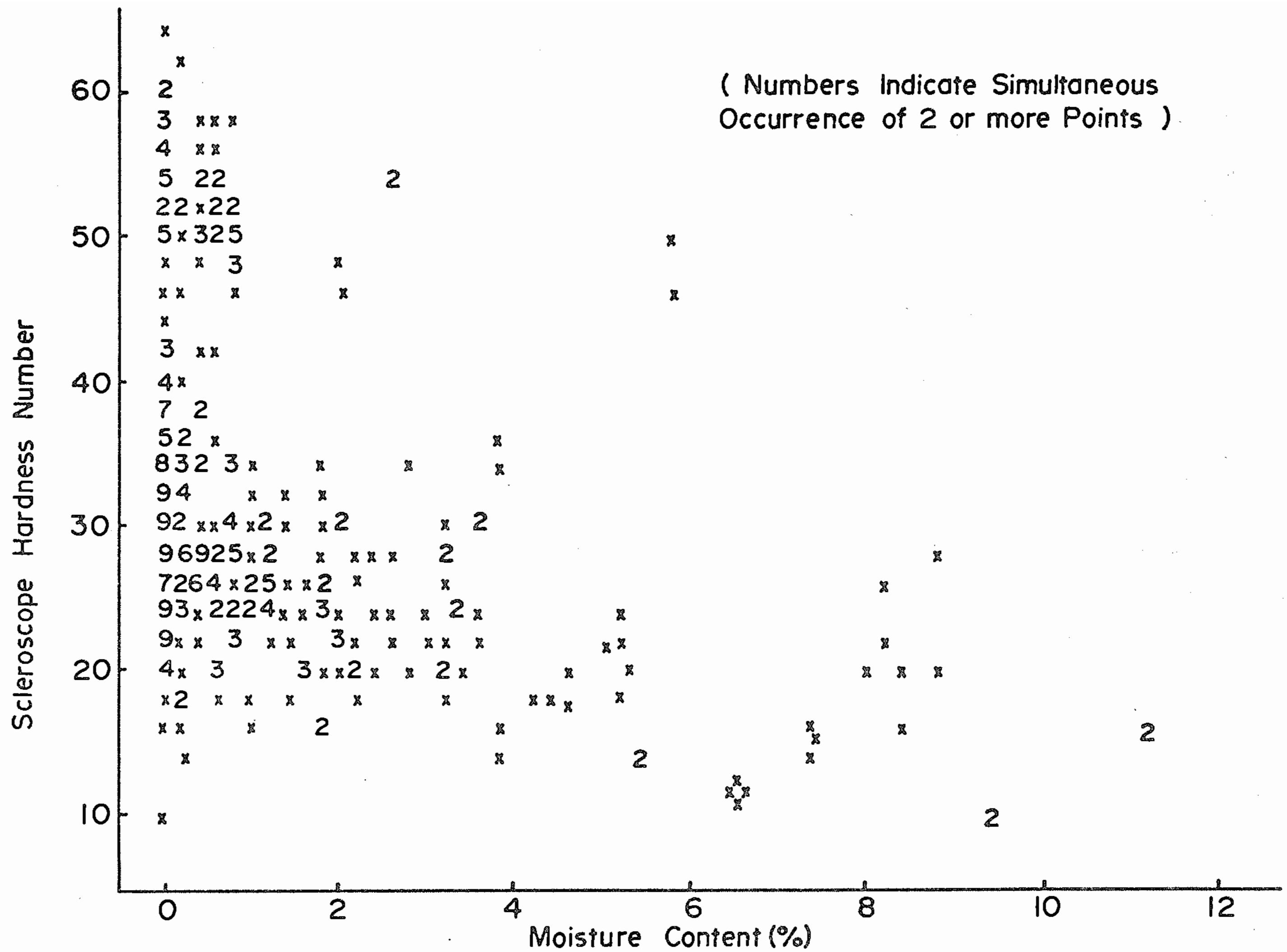


FIGURE 2 RELATION BETWEEN SCLEROSCOPE HARDNESS AND MOISTURE CONTENT FOR ALL SHALES TESTED.

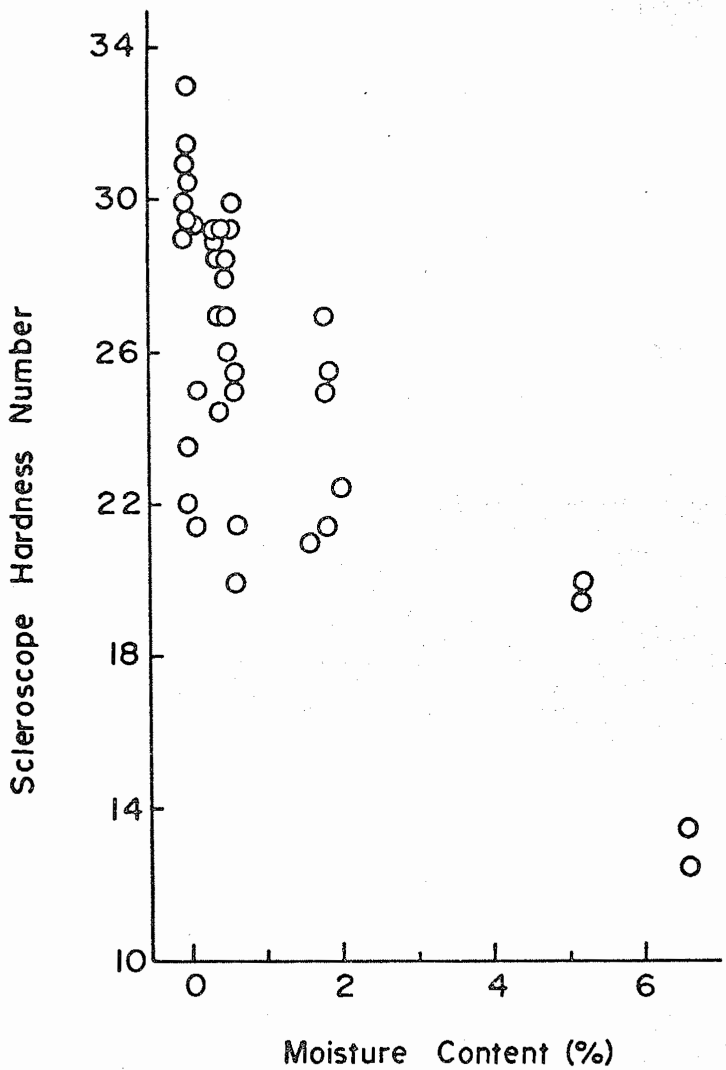
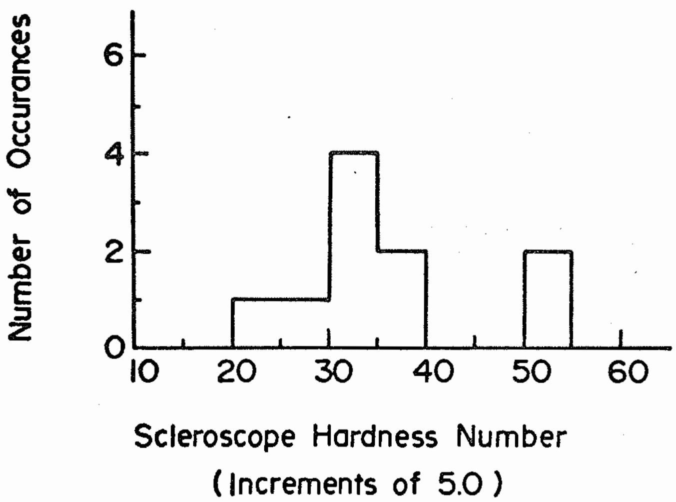
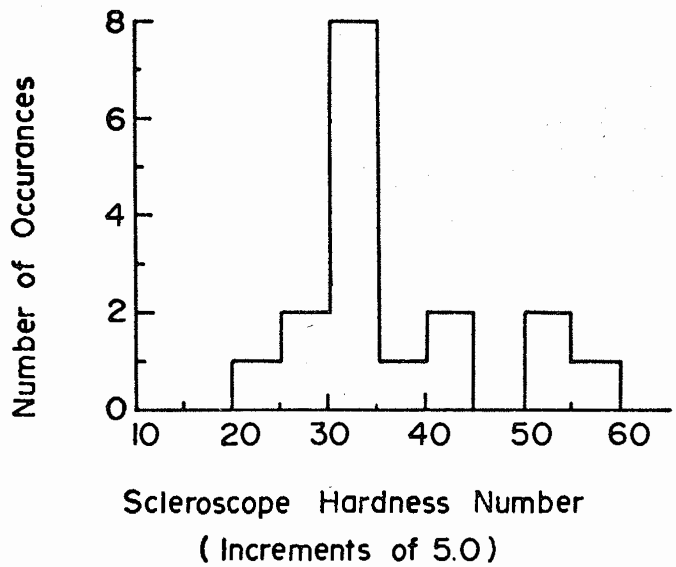


FIGURE 3 RELATION BETWEEN SCLEROSCOPE HARDNESS AND MOISTURE CONTENT FOR SHALE NO. II.



a) Calculated Results of Significant Correlations



b) Actual Test Results

FIGURE 4 FREQUENCY DISTRIBUTION BAR GRAPHS FOR SCLEROSCOPE TEST ON OVEN DRIED SAMPLES OF VARIOUS SHALES.

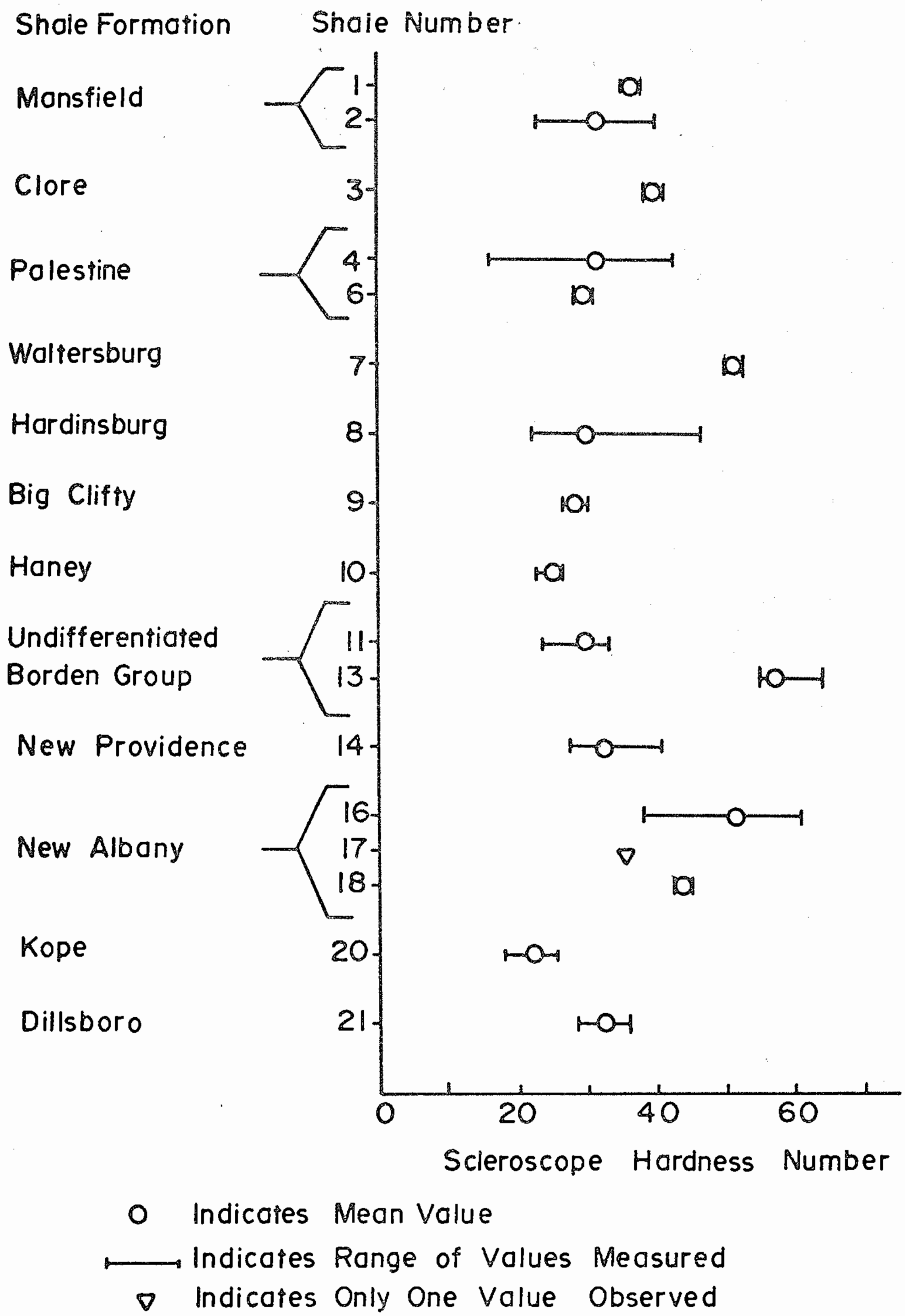


FIGURE 5 AVERAGE VALUES AND RANGE OF SCLEROSCOPE HARDNESS FOR VARIOUS SHALES AT ZERO MOISTURE CONTENT.

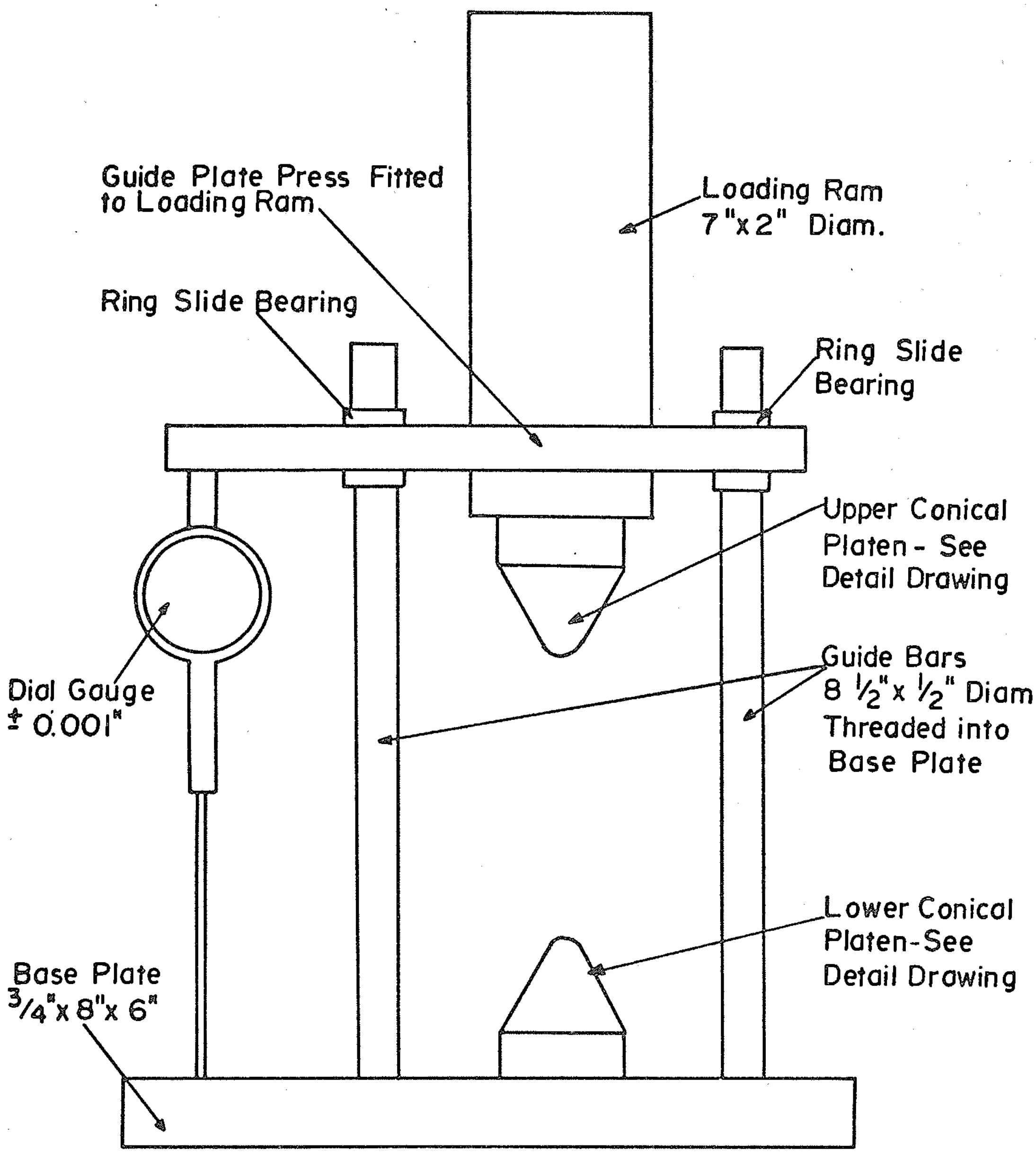


FIGURE 6 SIDE VIEW OF POINT LOAD TEST APPARATUS.

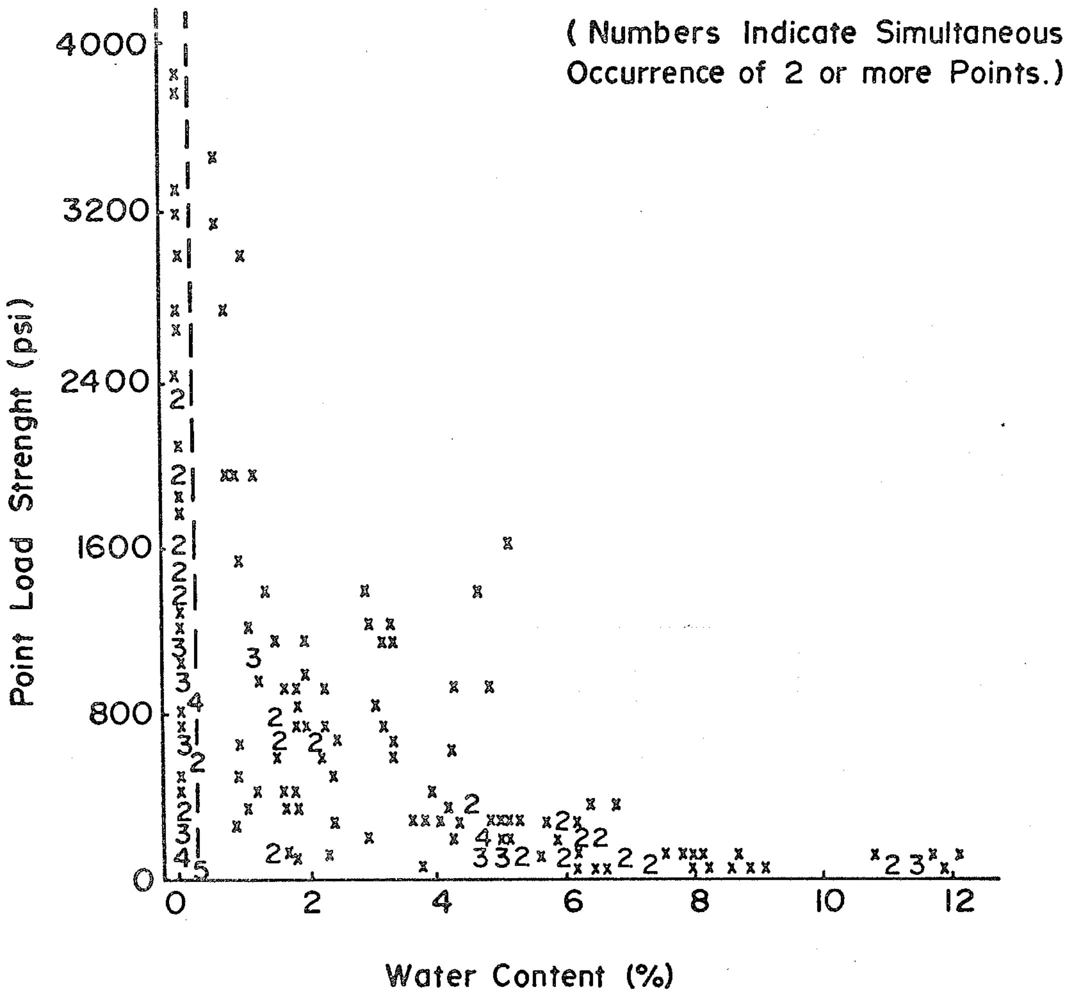


FIGURE 7 VARIATION IN POINT LOAD STRENGTH WITH WATER CONTENT FOR ALL SHALES TESTED.

ANGLE HOLE DRILLING APPROACH FOR HIGHWAY ENGINEERING DATA COLLECTING

By

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ABSTRACT

Selecting an angle hole drill site to obtain data on which sound engineering judgment can be based presents a perplexing problem to highway engineering Geologists.

First of many considerations is the budget limitation. The next is sample information required for laboratory testing of a particular job. Third consideration is the capabilities of the drilling equipment available and the proficiencies of the drilling supervisors and their crews. Last is the proficiency and level of the Engineering Geologist and the Design Engineer to present and utilize the data base gathered.

DRILL SITE SELECTION

Drill site selection is usually lost in a limbo, with prime consideration being given to the fault plane or other structural conditions, such as a spring along a bedding plane. Planning for drill site location should go hand in hand with the data required. When the drilling equipment is restricted to vertical holes, more expense and time will be expended in obtaining the data.

The flexibility of angle hole drilling equipment becomes important when dollar savings can be obtained from drill site locations and site preparation. The ability of the angle hole drill to fit itself into many existing locations or be used where little site preparation is required accounts for its acceptance.

In many instances, major Geologic units are flat or nearly so. The predominantly flat limestone shales and other sedimentary sequences are endowed with vertical joint systems which go undetected if only vertical drilling techniques are used. (Figure No. 1).

Drill site arrangements are a beginning point. To schedule a drill site, the Geologist or Drilling Foreman will know from many years of experience by visualizing his equipment in place; it is a mark of his competence. Those who are not blessed with this experience or visualization facility should have an alternative. It is used by interior decorators and industrial engineers; quite simple in scope. (Figure No. 2). Obtain a quad paper pad, such as ten square, four square or whatever. Scale out the size of each drilling equipment unit to be used, (drill rig, pipe rack, skid-mounted pump, etc.). Be sure to outline preferred work areas such as in back of the drill, beside the drill rack. Then, cut them out with scissors, labeling each for future use.

Now arrange these cutouts into an order, say, on a cut bench ten feet wide, or on a horseshoe bend of a river bar. It will quickly become evident to all concerned where the pieces of equipment will fit. What order will the logistics take? Will extra site preparation be necessary?

This exercise can be done in a cross section where terrain factors are pronounced. In doing a thorough pre-plan exercise on an aerial photo or topographic sheet, the entire picture is evident.

These procedures are useful not only to the designers and budget group, but the drill crews are made a part of the decision process. There must be good communications between all parties for a smooth operating drill project.

Things are brought out in the discussion of the operation such as weak brakes on the water truck requiring blocks to keep it from rolling, or other minor problems. Maybe the air compressor will not get adequate lubricating oil on a particular ground slope. With the model cutouts, a drill site can be moved onto and "rehearsed."

A drilling program at least to this stage, can be "de-bugged" and save the tax payers, contractors and highway departments considerable amounts of time and money.

The reason for drilling the holes is to gather data. A drill site properly placed, using angle hole drilling practices can gather more data by fanning out the drill pattern, e.g., gathering moisture samples in a slide area, or probing the slip plane where water is entering an active slide. The data base is many fold if angle hole drilling is used from drill locations in comparison to vertical drill holes.

It is true that standard penetration tests must be kept within plus or minus six degrees from vertical to have useful meaning. Shelby tube samples are not that particular. Auger spoil is not aware of the attitude it is being penetrated; if the samples are for sieve analysis or moisture, the orientation of the bore hole is not important.

ORIENTED DRILL HOLE

Oriented drill holes are not a new science. In the oil business, Eastman Oil Well Surveying Company have been surveying drill holes and controlling their progress to a definite target for many years. They began this work in earnest at Huntington Beach Field in California, in 1933.

The diamond people in Salt Lake City, Utah, offer a service which magnetically orients diamond core that is scribed with reference line. The core can then be positioned in a (Goniometer) Universal Stage. The dip and strike can be ascertained from this in-place orientation. Geologic conditions often give themselves abundant surface clues which assist in confirming a local preferred lineation and orientation of samples.

The use of three dimensional perspective in thinking of drill holes requires a visualization - what the drill bit is doing. A Geologist, with his background, makes the very best drill operator. Most modern hydraulic drilling machines transmit delicate signals through the drill rod and controls where the perceptive drill operator can discern formational changes, rock types and other local conditions.

DRILLING EQUIPMENT

Most drilling equipment that is capable of drilling angle holes was operating for the mining industry. It was small in physical size and used principally for underground core drilling. These drills were either air powered or diesel when adequate ventilation was available. The angle at which the drill operated was a plane at right angles to the long axis of the drill unit and in any of three hundred sixty degrees in that plane. It was, because of the light weight and small physical dimensions, necessary to securely anchor the drill frame. These anchors were to be drilled in place, rock bolts making a very ridged drill station. On the surface the anchoring procedure was similar. Where soil or extensive overburden was the case, cable buried in concrete filled drill holes were common practice. A special case where anchoring would be redundant is with a heavy fifty to sixty ton angle blast hole drill.

The use in the mining industry of these small angle hole drills has progressed to foundation engineering for dam construction. Progressively larger pieces of equipment have evolved as the needs were met. Today the changes have been that the drill units for underground operation developed into a feed frame with automatic chucking and rod holding, mounted in any position. The units are all hydraulically controlled from a separate console and powered by a remote diesel or electro-hydraulic power source.

The surface drills for angle hole drilling, with the exception of the air track, all drill in an arc parallel with the long axis of the drill carrier. Lighter units, twenty tons or less, still require anchoring because feed pressures have risen along with rotational speeds.

The surface drills have gone to hydraulic rotation, feed and tool handling. They have also been blessed with much higher feed pressures, to sixty thousand pounds and rotational speeds for rotary, 200 to 300 RPM - core drilling up to 1000 - 1500 RPM. To obtain these speeds, very ridged feed frames are required. Unlike the underground unit where three feet to five feet lengths are the rule, surface drill units have evolved with feed frames from ten to thirty feet; ten feet being the best for high speed core drilling, while the longer units are matched with down-hole-hammer rotary air drilling.

The use of automatic hydraulic chucks, hydraulic rod clamps and hydraulic breakout systems has increased the foot per shift of core in the box from thirty to forty feet, to an excess of three hundred feet for underground and surface drill units alike.

DRILL CREWS

The drill crews and supervisory personnel have evolved and adapted to the changing equipment innovations. However, it frequently happens that education and innovation fail to penetrate personnel and personnel management. It is in these bastions of antiquity that the Failing Six-Hundreds, Mayhew One-Thousands, and B-40's still operate.

The 1940's produced opportunities. A great demand was generated for drilling personnel to drill seismograph shot holes. This training period has brought drilling personnel to the level presently enjoyed. But since the mid 1950's little or no large scale apprenticeship program was needed. Which leaves us with a critical shortage of trained drilling supervisory people and few interested in getting into the profession.

Highway drilling divisions while unique, are handled in many fashions and nearly as many variances as there are states. In Arizona where my ties are, it was the early 1970's before the backhoe and jack hammer stepped down to be replaced with drilling equipment in the materials division.

In other states, exploration drilling is a separate organization. Working with many state agencies these drilling organizations had developed significantly in the 1950's to be handling water wells at maintenance camps, construction projects, and foundation investigation at bridge site construction projects.

When the big push came by B.P.R. for the interstate system, the lions share of the foundation investigation went to consulting firms because they were flexible in their private organizations and could acquire the equipment and personnel to provide the testing and sampling services. The various states as a whole are in a commanding position where materials testing, sampling and inventory are concerned as the interstate system nears completion.

The states should not rest at the end - disband the drilling crews, giving each a sack and a sharp stick. It is true the roadside needs constant care. But we have trained people, not rag pickers. They can be utilized in up-dating and re-designing of problem areas on the interstate, rock slides, spring encroachment, subgrade stabilization. The numerous water wells on the interstate road side rest areas should require a water well service crew to pull pumps, stimulate, re-develop these existing water systems.

PROGRESS

The present and future uses of angle hole drilling in highway sampling largely depends upon the innovation of supervisors. The use of angle hole drilling substantially reduces the cost through drill site preparation and the number of drill sites required.

Angle hole drilling procedures causes the report writer extra effort to turn this data into useful information; give a little background for the report reader, noting the cost advantages. Toot this horn softly but show the cost comparison to vertical drill holes. Describe other applicable sampling techniques making it understandable why the method used was selected.

REPORTS

Don't just say . . . "the following samples were taken." This only conveys that samples were taken and in the mind of the design engineer, he remembers the spring day in the afternoon when he was at the University - the class went on a field trip. They grabbed some rock from a stockpile or took a shovel and filled a sack with subgrade. This design engineer has no conception of the difficulties involved in sampling the road construction project; whether you shinnied down a rope off the top of a cut slope in a rain storm, or fought a pack of mosquitos from a drill site and saved the state $400.00 in drilling costs. You, the drilling people, have to tell the story in a fashion that rings of truth with a dash of Smilin' Jack!

SAFETY PARTICULAR TO ANGLE HOLES

These safety precautions are particularly important to angle hole drilling:

1. Pipe handling: Use a rope, not hands. Be particular about mashed fingers between the mast slide and drill pipe.
2. Drill pipe that is left on slide or mast should be secured.
3. Protection for drillers station during windy construction should be provided.

HAZARDS UNIQUE TO ANGLE HOLES

Problem areas particularly prominent in angle hole drilling includes the most prevalent one of hole sluffing from loose or broken ground conditions. This problem increases as the angle approaches horizontal.

The best procedure is to keep good drilling mud in the hole and a wall cake that retards sloughing. Some air track people use drill tubes or casing as a matter of standard practice but I believe this would be an extreme measure for every angle hole.

Tool handling is tougher. It is my experience that pulling ten foot rods is safer and faster than wrestling with twenty foot rods. An automatic chuck and breakout system makes the job on the surface equipment nearly as fast as underground units.

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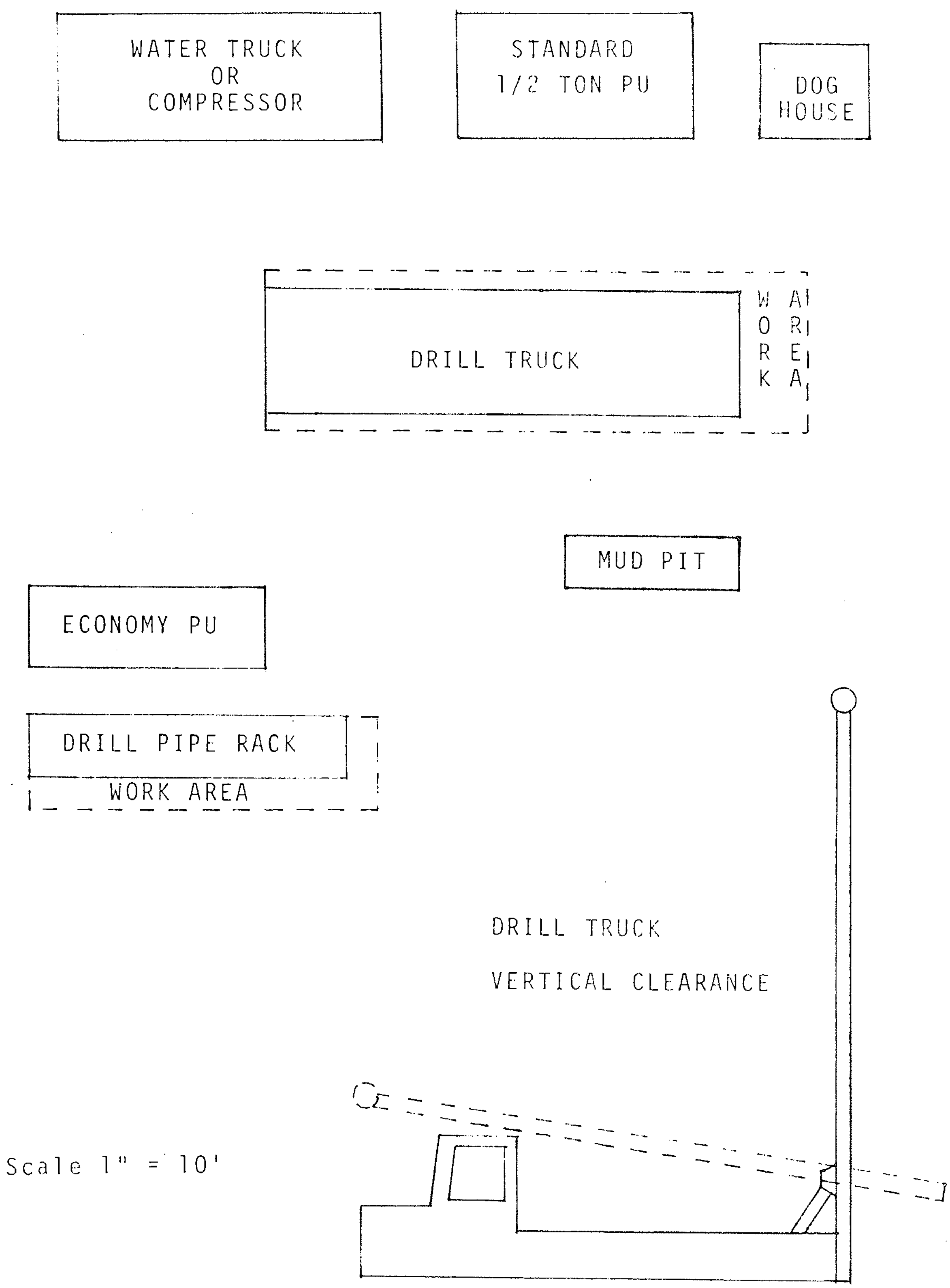

Figure 1. Cutouts for Figure 2.

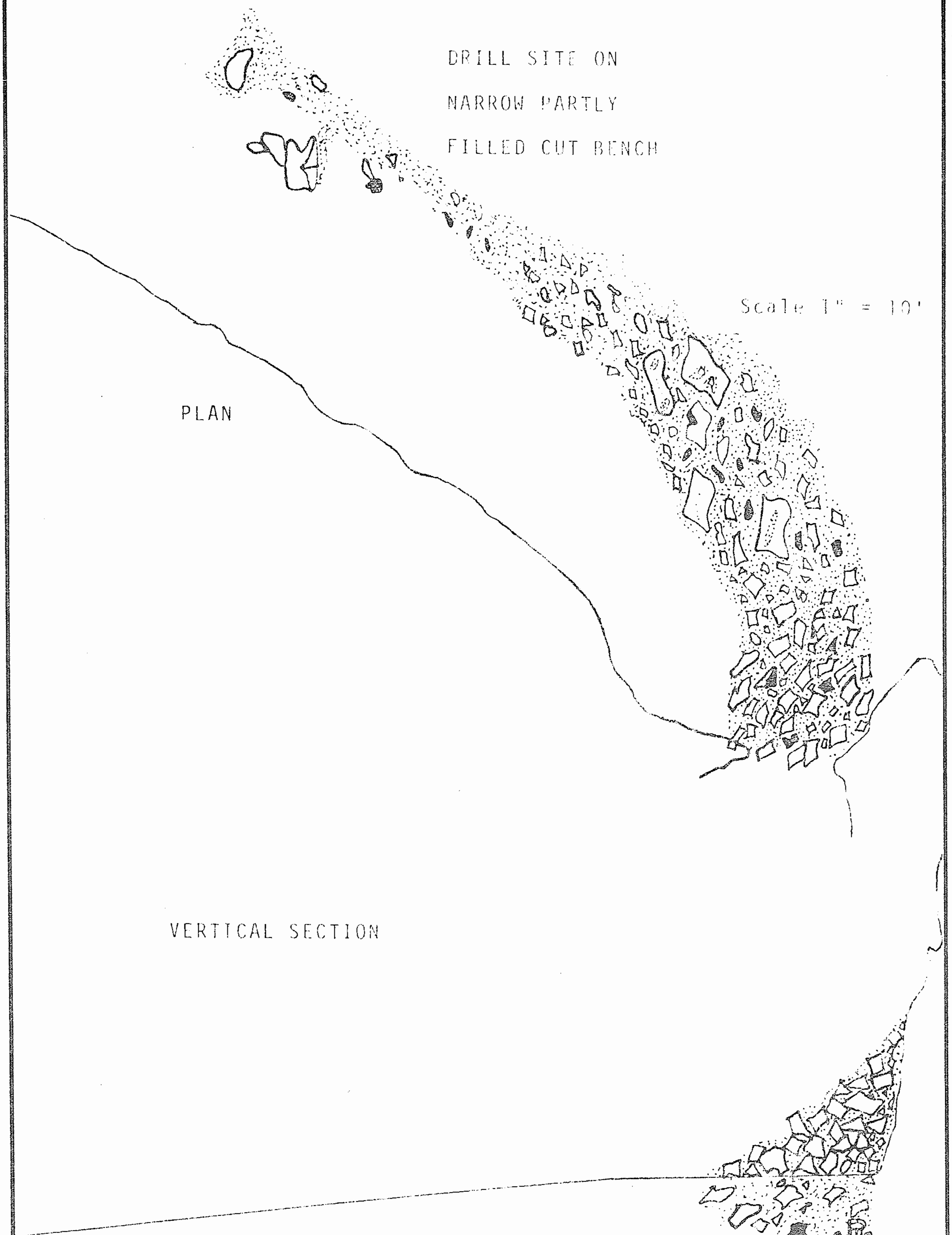
DRILL SITE ON
NARROW PARTLY
FILLED CUT BENCH
Scale 1" = 10'
PLAN
VERTICAL SECTION

Figure 2.

CUT SLOPE DESIGN BASED ON STABILITY CHARACTERISTICS

By

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ABSTRACT

The performance of the materials forming cut slopes can be predicted from their physical and chemical properties. These properties must be known and dealt with in slope design and construction. Materials that comprise cut slopes in the mantle cover and in the geologic formations within the Mississippian, Pennsylvanian, Permian, Cretaceous, Tertiary and Quaternary Systems in Kansas will be discussed in regard to their slope stability characteristics. Problems such as overbreakage and overblasting in construction methods as well as the forces of frost wedging, undercutting, spalling, and slaking through wetting and drying are to be dealt with.

Cut slope design which is determined purely by economy of construction rather than the overall performance may not be economical in the long run. Slopes which are constructed from a design that carefully considers the stability characteristics of the individual geologic units involved are more desirable.

Slopes in nondurable rock should be flat enough or benched to retain the weathering materials.

Some conditions require that benches be constructed in softer layers especially below more durable rock units in order to catch rock fall and to protect the ledge from undercutting.

Presplitting of durable rock tends to reduce spalling experienced under normal blasting.

Dressing slopes with top soil covering for slope seeding to enhance beautification and vegetative stability are desirable at specific locations. Mowing procedures are also of special importance.

PURPOSE

The purpose of this paper is to develop guide lines for a manual usable in the design and construction of cut slopes. It should help to develop a working manual that could be used by the Geologists, and Engineers in both design and construction for an overall view of cut slopes stability and economy.

It is appropriate that an analysis and procedure manual based on an in-depth study of past cut slope performance be developed.

This effort should portray all historic geo-engineering input and the engineering application made thereof in the design and construction of cut slopes. It should support the methods presently used yet be openly critical for advancement of better slope design and construction methods.

Aside from two short special reports by Frank W. Wilson, Kansas Department of Transportation, the Geology files have no records of cut slope guides or manuals.

The broad knowledge of this subject is still disseminated within the various highway project reports. It is desired that this information be brought together for more immediate and better use.

GENERAL SLOPE CONDITIONS

Normally in Kansas we do not experience great difficulty in cutslope behavior throughout the 10,000 or more miles of Primary and Secondary Highway System. This results primarily from several factors, the availability of land, the type of terrain, the types of materials involved, and the rainfall. In most areas across the State there is sufficient land to build upon to avoid, to a large extent, the rough areas that require deep cuts.

There is only a change of approximately 3325 feet of elevation across the State which develops quite gradually. The rainfall is normally not sufficient in any portion of the State to create a broad saturation of slope materials and in most areas stable materials are present.

These benefits alleviate much trouble with cutslope performance other than during unseasonable periods of rainfall. With all these benefits we could easily be lulled into a "misbelief" that we have reached perfection in Kansas Department of Transportation cutslope design.

THE GEO-ENGINEERING ROLE

There is considerably more involved than previously stated. The success the Kansas Department of Transportation is presently enjoying in cut slope performance of its newer highways, as well as for other aspects of geo-engineering, results from the knowledge and experience provided by the highway geologist and his working through the highway engineer for the past 40 years or more. During this period of time many problems have been confronted and solved.

In the years when Highway Geologists arrived upon the highway construction scene to assist the engineer, highway building methods were fast changing. This began the evolution of viewing foundation and earthwork in highway construction from a geologic engineering viewpoint.

The information for foundation and earthwork design was not then being satisfied by information from driving the sounding bar or the hand auger borings to the top of a rock ledge. Unanswered construction problems were arising in the areas of rock excavation, cut slope design and groundwater problems. Previous information, with limited detail such as top of rock and area of surface seepage were too inadequate to analyze the design and construction conditions at depth. These answers were necessary and the geologist could give the most logical answers required.

The beginning geologists, in the Kansas Department of Transportation Section, had the technical knowledge of basic geology. A learning period was required to know how to apply this basic geologic knowledge to engineering problems and to make application of this information and properly present it to the engineer responsible for design and the construction of the highway.

Progress was slow during early years since experience was lacking and there were few people then available having a broad engineering geology experience. Experience had to be gained, only through time, by the observations made of one construction project after another. Learning the game certainly involved some reverses and failures. No doubt many overruns occurred in rock quantities. Many water problems were unresolved and backslopes were poorly constructed from early geological information.

Over the years the acquisition of geo-engineering information for design and construction of Highways has been improved or perfected and very good engineering application of it has developed. A most important factor during this period was that engineers and geologists acquired a good understanding of each other's efforts and began to work more closely to develop the best possible plans for the construction of the highway.

A good working relationship between the Geologists and Soils Engineers is very important in this effort. This relationship can best be accomplished by spelling out the duties and functions of each in a partnership by guide lines for operation and procedure.

Through the years knowledge, ability and judgment in geo-engineering has developed. A mutual confidence has developed between geologists and engineers through the years through the efforts of obtaining thorough geologic data and applying good engineering principles.

DEVELOPMENT OF SLOPE CRITERIA

In the atmosphere of the foregoing philosphy we will enter into the subject discussion of cutslope stability in highway design and construction.

Our present good success in cutslope construction is the result of knowing the slope materials, using good slopes, benches, rock buttresses, presplitting, seeding and erosion control.

The first step in the slope analysis is to have a complete understanding of the present situation. How did we adopt the present procedures? Are we aware of the problems that might exist where changes should be made? We must be fully aware of any problems that exist, and use the best knowledge and judgment required to properly analyze and control them.

The general history of backslopes should be a starting point for this study. We still have some cut slopes that were constructed during the turn of the century, having been in use for the past 70 years or more. There are other slopes to be observed that were built 25 to 50 years ago. These slopes all provide a good laboratory for the study of slope behavior.

Slopes constructed in the later years, for the most part, have given little trouble. However, even through this period there have been extenuating circumstances such as improper design due to improper information or, improper construction from proper design, which have resulted in cutslope trouble.

REPORT ON OBSERVATIONS

The analyses, founded in history, should discover the successes and failures of the past, the evolution that has brought us to the present and predict the improvements needed for the future.

In making such observations, studies, analysis and recommendation a standard check form should be used. This should help in making an accurate description of the conditions which should be the key to solving cut slope design criteria.

It is recommended that the study be made in the following manner:

A study should be made of cut slopes construction of various ages throughout the state to determine the problem areas of maintenance.

This activity should be carried out when time is available in all three Geology Regions with an attempt to study all geologic horizons across the State. Studies would be primarily directed to cut slopes that are pronounced and have good exposures of the geologic layers.

Observations should be made of the function of benches with relation to their width and geologic position and also the behavior of the soil mantle and weathered rock. The geologic history and soil development of the area is particularly important in predicting soil slope performance. Where heavy colluvium or slope creep exists there could well be high permeability for groundwater movement that could cause an unstable soil mantle condition. Quite often a thick colluvial mantle, covering certain types of shales, becomes very weak when water is involved. When there is a high permeability, slippage problems are likely to develop unless there is careful slope design and construction.

Another point in the investigation should be directed to vegetative cover over the various geologic types of formations checking on their ability to support good vegetation. Also the best kinds of supportive vegetation should be reviewed.

The direction the cut slope faces affects its performance due to the natural elements. For example west-facing slopes which catch the afternoon sun might tend to dry more quickly than those facing east thus affecting the vegetative cover. A north facing slope has more tendency for deep frost penetration causing frost heave, breaking away the material on the slope face and forcing it outward and downward.

Observations should be made as to the stability of slopes in regard to ditch cleaning and their relationship to the surface condition of the road.

Interviews should be made with maintenance personnel at various levels in regard to rock fall and ditch cleaning costs. Slope erosion conditions should be studied and recommendations made for its control. Observations should be made of the performance of stepped slopes.

Efforts should be made to develop this information into a cut slope criteria to be used in a Manual of Highway Geology.

This guide should assist the geologist in obtaining better field data and analyzing the data for better design and construction of cut slopes in regard to stability, safety, appearance and economy.

It is desireable that a systematic review be made of newer cut slopes whereby the information obtained would be directed toward the constant improvement of cut slope design and construction.

Fully utilizing this type of information should eliminate any practice of depending upon some standard cut slope design, since proper consideration would be given to all the slope-related conditions involved. Materials from cut to cut may seem basically similar but many stability factors should be taken into consideration before accepting this fact.

It should be considered that many times the stability of a particular formation depends more upon the stability of another formation rather than on the material in the formation itself. For example a limestone bed may be a good, hard stable material but if an underlying shale layer is weak, then the limestone will fail because of the weakness of the underlying bed. Soil mantle is another example of a material that depends to a great extent upon the surface and subsurface conditions of the underlying material for its stability rather than on the soil material itself. These conditions include surface drainage, groundwater elevation and movement, and type and slope of the underlying bedrock. Also, in case of the soil mantle, it has to be decided what slope will support the best vegetative growth, provide the safest and easiest maintenance and still give a reasonable right of way width, since the flatter slopes for the soil mantle many times are the most important determinant of the width of right of way.

Usually cut slopes of considerable height are more closely observed for their evaluation than are the moderate or small ones. Even though there may be emphasis on large cuts this should not prevent full study of the just average slopes since many of these may have similar failings in a smaller way, yet in total, may have maintenance costs that would overshadow higher slopes.

Therefore all facets of stability characteristics should be applied to the design and construction of any cut slope whether large or small because of their importance to the road structure.

This analysis should take into consideration all general types of slope materials that are to be dealt with across the state. Included would be the Soils and unconsolidated materials of which would be unconsolidated loess; the silts, clays and sand mixtures of partially consolidated Pleistocene; the more consolidated silts, clays and sands with mortar beds in the Tertiary (Ogallala); down through the variable series of mantle-covered alternating chalk, shale, limestone and sandstone units in the Cretaceous; the mantle-covered units of the Permian, Pennsylvanian and the Mississippian Formations.

DEVELOPMENT OF HIGHWAY STANDARDS

There are some roads still being used on the Kansas highway network that came into existence before the larger earthwork and grading equipment came into use. Many of these roads were constructed on narrow right of ways with steep backslopes and narrow ditches that have to be cleaned every few years in order to maintain ditch drainage. Careful ditch cleaning practices should be developed for these types of roads to prevent creating slide problems, yet still keep functional ditches.

After equipment came into service that would allow larger earthwork capacities, in the 1930's certain important highways in the state network were straightened and given more ruling grades. Even though they were not elaborately designed considering present standards deeper cuts were involved, with their related backslopes, that were quite often overly steepened for right of way economy. Just prior to World War II, the design period for straight alignments began which substantially eliminated the curves. This also provided for more ruling minimum grades with wider ditches and flatter shoulder slopes and improved cut slope design. Very few of these designs were constructed prior to the War, which basically stopped all construction.

After the War years there was a period from approximately 1946 to 1950 when much of the available money was applied to the surfacing and dressing up the existing routes to take care of immediate transportation emergencies and many miles, with new base type surfaces, where constructed on the generally existing grades and right of way. Thus, very little earthwork could be done to improve the backslopes and ditches on these many miles of Highways. Because of this improvement these highways are still in use, even though the grades, slopes and ditches are less than desirable.

Following this period from 1950 on, large grading projects were constructed with higher standards of construction, with flatter grades, wider roadways and ditches, and flatter backslopes and shoulders. Economy of right of way now became less involved in cut slope design. Thus, the demand for much wider right of way began. During this period the Interstate Highway Program came into existence. A program for relocating many miles of existing highways due to the construction of several large reservoirs also came along.

The Interstate Highway Program no doubt has had greater influence on the standards for highway design and construction than any in the lifetime of road building since this super highway system was financed on a 90-10 basis by a large Federal Interstate Trust Fund. In the first years there were few restrictions on costs if the expenditure would produce a more quality highway. The higher standards for Interstate Highways influenced the design of the other highway systems and costs increased rapidly during this period of time aside from inflation. Regardless, this period provided the State with a good many miles of excellent newly constructed highways before funding began to cut seriously into this program.

Along with these major highway programs came more serious thinking into the Environmental Aspects of highway location and design. Highway designs were strongly influenced by aesthetic as one of the environmental aspects.

At this time environmentalist and the highway engineers were somewhat apart in their opinions of these early federal programs of which many had a deep seated environmentalist backing. The highway engineer up to this point in time had been pressed to produce a usable highway in the shortest time because of the extreme need to get the road open to traffic and utilize the expenditure as quickly as possible. Often surfacing contracts were let at the same time or immediately following grading and bridge contracts. The Environmental emphasis changed this procedure quite abruptly. Possibly insufficient thought had been given to aesthetics and ecology in earlier years of highway construction, especially in the more rural areas, even though within city areas more consideration had been given to these areas.

During this period of time the aspects of slope design in regard to ecology and aesthetic was brought to the attention of the engineer as much as any other facet of highway design and construction. With the wide differences of opinion at this point there had to develop a workable middle ground of professional ideas. Ecologists on the one hand were strongly heard and wanted to put a halt to anything that would upset the ecology by destruction of existing vegetation which could create erosion and siltation problems. Their demands were that where nature was to be disturbed, it should be most minimal. Erosion and siltation should be kept under complete control and greenery, wherever destroyed by construction, should be returned to green. With this came the national Highway Beautification Program emphasizing that all slopes, as far as possible, should be green and tree covered. This was going to cost money but it was believed then than the public was willing to pay for these things even up to the point of luxury and federal regulation became very pronounced in this area. Now the Engineer was faced with the problems of building the highway, attempting at the same time to design and construct it within tolerances of these new and stringent environmental regulations. This became a difficult time in the design and construction of cut slopes. There was a stress for greener slopes requiring all cut slopes, as far as possible, to be covered back with vegetation. To do this there would have to be flatter slope design in the softer materials and also in some of the more durable rocks. This, in some cases, would require the flattening of slopes containing hard rock to be cut a minimum 3:1 slope. To do this the rock formations had to be undercut and backfilled with a dressing of topsoil. There was a problem of the availability of soil covering in some areas, where badly needed.

In cases where vegetative cover was considered a necessity, as in the case of urban or near urban areas, cut slopes of limited magnitude were sloped back on the minimum 3:1 slopes recommended and covered with a minimum of 6 inches of topsoil to support vegetative cover. The slopes were thus flattened in order to hold the soil covering from slipping in severe wet seasons, as well as to allow mowing without appreciable damage to slope by the mowing equipment. Where soil covered slopes were made steeper than this, occasional slippage occurred and there was often evidence of slope damage caused by mower wheel tracks. In some instances, where soft rock and durable rock was undercut and backfilled with soil covering, there were areas that would support only sparse vegetation other than in the thicker soil cover areas where moisture was available. A criteria should be developed for determining what height of slope durable rock should be undercut and flattened for soil dressing as opposed to using a steeper non-covered slope design.

STEPPED SLOPES

There was the promotional period for steps in the slopes of soft rock formations to provide greener slopes. These employed steps cut at about 3 foot vertical intervals and in nearly level direction. The width of the steps are related to the vertical intervals and are a function of the slope ratio. This slope method is described in a memorandum from the Bureau of Public Roads entitled "Steps in the Stairs to Greener Highway Cut Slopes".

The backslope is stepped as shown on the plan and cross sections and constructed in the following manner:

"Cut in soft rock, as listed on the plans or designed by Engineer, shall be excavated to shape the cut face to a stepped pattern in reasonably close conformity with a typical cross section shown in the detail.

"The steps may vary one foot vertically from the dimensions shown on the plans, with horizontal dimensions being a function of the staked slope. The approximate midpoint of the horizontal tread of the step shall be constructed on the staked slope line.

"The first step shall begin immediately below the soft rock line and continue to the bottom of the designated formation(s). The steps shall be constructed horizontal (± 1%) rather than parallel to the grade line grade. Excavation of each step shall be in opposite direction from the proceeding one to minimize buildup of loose material at the ends of steps. Loose material which collects at the end of the steps shall be removed and the ends blended into the natural ground. Where rock too hard to rip, is encountered within a cut, the step shall be blended into the rock.

"Scaling need not be performed on the stepped slope except for removal of large rocks or material which may fall into the ditch or onto the roadway and create a safety hazard. Construction of stepped slopes shall be subsidiary to other items of the contract.

"Individual benches constructed during the spring and summer months shall be seeded promptly under the temporary seeding provisions of the contract. Benches constructed during winter months shall be seeded as soon as weather permits under the temporary seeding provisions so that vegetation can be established early in the spring. Benches seeded earlier, which have not developed satisfactory vegetative cover, shall be reseeded with temporary seeding: all seeding or reseeding shall be done without disking or other preparation.

"All Steps shall receive permanent seeding without disking or other preparation under permanent seeding contract for the project."

The steps are constructed as the cut is being brought down to grade and cleanup of the backslope is minimal since the purpose of the benches is to catch and hold loose material and moisture which in turn will support vegetation. Topsoil covering if available is not used in order to eliminate the extra expense.

These slopes are seeded but not mowed to allow the slope to return to a natural state of vegetative cover.

SERRATED SLOPES

Consideration has previously been given to the use of serrated slopes in shale cuts but to date none have been completed. This method should be given future consideration. The first proposal for serrating slopes was presented back in 1967 by the Landscape Section and Geology Section for use of 3 different projects that were already under construction, 2 were in Jefferson County and the other in Riley County, but due to various difficulties these were not constructed.

This was a ripping method of treating backslopes that was published in a technical journal several years ago and was modified for use in Kansas Department of Transportation of George Mathews of the Landscape Section.

The ripper grooves run horizontal along the backslope and are spaced approximately 3 feet apart. The ripping is done, as the cut progresses, by running the ripper along the point of intersection of the backslope line and the roadbed line. This allows one dozer track to run on level ground and one up on the slope.

Grooves are cut to a minimum of 14 inches deep leaving the backslope in a rough condition. Topsoil cover is not used since one of the reasons for this special treatment is to eliminate the extra expense of soil cover. The backslopes are seeded but not mowed to allow the slope to return to a natural state of vegetative cover. This item of work is not to be paid for directly but is a subsidiary item to excavation.

Stepped and serrated slopes in soft rock, which usually refers to firm shales, will have limited use due to interbedded alternating durable limestone and sandstone layers that are associated with the shales units. This lack of workable thicknesses of individual shale units will prevent this method from being extensively used.

Any success of vegetative cover on these steepened slopes, regardless of method applied, is dependent upon the amount of rainfall and fertilization.

There are portions of western half of the State that the annual rainfall is such that vegetation, even on good fertile soils, is sometimes impossible and each location becomes a trial basis. Therefore this type of slope construction applies better to the eastern portion of the State where more rainfall is normally present.

PHILOSOPHY OF GREEN CUT SLOPES

Not all persons are in agreement with the philosophy that cut slopes should be covered with greenery, if for beautification purposes. They contend that the word beautification, has become synonymous with vegetation and the main aesthetics of a highway is based on the major design of all the highway components. Vegetation has an important place in highway aesthetic and in erosion control, but is not a beautification panacea. There are many traveling the highways that are extremely interested in rocks and rock formations and it is quite common to see people in rock cuts looking and taking pictures of rocks. There is more importance of vegetation wherever the soil, precipitation and climate warrants but trying to establish vegetation on areas that will not support vegetation naturally always costs money. It is conceivable to grass cover bedrock slopes in specialized sections such as a depressed section through an urban area where seeding and sodding may be more economical, but in their opinion, it is not necessary that bedrock always be covered with vegetation because of the high initial costs and later maintenance costs of which includes mowing, mulching, cultivation and watering. If bedrock backslopes are properly designed and properly constructed they can present a more pleasing and interesting appearance to the passing motorists than grass covered slopes.

SLOPE DESIGN CONSIDERATIONS

Kansas highway cut slopes are constructed in a variety of unconsolidated materials and consolidated sedimentary Geologic formations.

A recommended slope is established for each geologic unit based on the known geologic characteristics of the unit and those of associated companion units.

Slopes in soils and other poorly consolidated materials are not generally troublesome especially in the drier portion of the state where slides are not common.

In these areas erosion would be the greatest problem where quite frequently silt fans from slope runoff causes filling of the ditches.

In the eastern portion of the state with its greater rainfall silting and sliding becomes more of a problem. This is in part due to the amount of clay materials involved. For a prime example a large cut slope in glacial material located on the west side of the Kansas River in the Kansas City area became a slide victim. The slide developed during construction when a soft layer of weak clay probably, not more than three to four feet in thickness, near the bottom of a 70′ cut section, was exposed by the construction. This exposed clay layer was freely extruded out by the weight of the 70 foot soil column above it. The release of this supporting clay allowed the whole cut slope face, composed of silty clay, to drop downward and break up in shear. This major slide resulted in considerable property damage since there was a home at the top of the slope.

SHALE SLOPES

The shales encountered throughout the geologic section are probably the most troublesome of all types of slope materials.

There are a number of shale members that are known for their poor cut slope stability due to jointing and slippery characteristics. These shales present additional slope problems when the companion limestone or sandstone members above or below them carry water.

Some of the most troublesome shale units are the Blue Hill, Graneros, Dakota Formation, Kiowa, Havensville, Stine, the shales of the Douglas group, Weston, Wea, Pleasanton and some of the shale units in the Cherokee group. Some of these shales have bentonitic characteristics while others demonstrate high clay characteristics in the presence of water.

There are both compacted and cemented types of shales to be dealt with across the State. The compacted clay shale types are developed from clay that has been consolidated under overburden pressures. Bonds between clay particles are molecular and are formed as the particles are forced to physically conform to one another. In cemented shales, the bonds are formed by precipitation of cementing agents at the zone of contact between particles.

Weathering of clay shales is a more rapid process by which they are broken down to form residual clays. This results in physical changes by exposure to the atmosphere and to other changes which destroys the bonds and allows the shale to expand.

Weathering is both physical and chemical. Physical disintegration changes the structure of the clay mineral by breaking interparticle bonds; chemical changes and decomposition of the minerals also occur. A color change often accompanies the chemical phase of weathering, but weathering occurs considerably deeper than color change might indicate. In this state of weathering additional water is taken up by the shale to cause weakening.

Since many of the slope problems result from the behavior of shale members it is necessary that careful consideration be given to the slope characteristics of each individual shale unit in cut slope design. It should be determined how the shale unit will perform under these categories of stability; rate of weathering, undercutting, slaking, erodability, jointing, density, void ratio and moisture interbeddedness, permeability, compressive strength, and potential swell. The majority of cut slope difficulties can be solved if they were designed and constructed with these characteristics taken into account.

LIMESTONE AND SANDSTONE SLOPES

Closely related to the shale cut problems are the limestone and sandstone members which are sandwiched between them. As in shales there is varying degree of inherent weakness in the limestone and sandstone members associated with them. The durability characteristics of each limestone and sandstone member need to be closely analyzed and considered.

ROCK FALL

Benches should be constructed at the base of durable rock layers where possible or desirable in order to prevent undercutting the hard formation and to catch the rock fall preventing it from falling into the ditches or onto the roadway.

The recommended cut slope benches should be noted on the typical section sheet of the plans. They should be constructed so that where the grade line intersects a backslope bench elevation, the bench should be at its full width at the point where the ditch cuts the bench. The extra ditch width should be tapered to standard width within 100 feet upgrade. The benches should extend out into the soil mantle at the ends of the cuts and be rounded back to the soil mantle slope. All benches should be covered with soil and seeded.

Safety in cut slope design is of great importance to the highway for the safety of the traveling public.

Certain designs of the cut slope are matters of safety. By procurement of a small amount of additional right of way in shallow cuts; it becomes possible to change a hazardous 1:1 backslope into a gentle 3:1 or flatter slope when this procedure reduces hazardous rock fall that might otherwise tend to litter the ditch and shoulder section of the road.

Rock fall from cut slopes should be prevented in matters of safety along with maintaining an open functional ditch. Certain rock members require a flattening or benching of the softer shale layers beneath them to prevent undercutting and to catch the fall.

Removal of rock fall from the ditches is also an expensive maintenance item. There are records from several states with estimates, which show that the unit cost of removing fallen rock and debris from ditches varied from three to ten times the cost of excavating an equal volume of similar material during the original construction.

IMPROVED CONSTRUCTION METHODS

Improved construction methods in themselves can improve slope stability. Within a short span of time techniques of smooth blasting of durable rock layers have provided better cut slopes. The use of presplitting and preshearing methods and angle drilling of shot holes for excavating rock cuts have provided better quality slope faces.

These methods have reduced the problem connected with overblasting and frost wedging of prefactured rock faces. Contractors have discovered that good presplitting methods reduces their cost of final shaping and cleanup time. The improvement of these drilling and blasting methods are producing a more stable slope with a neater clean appearance without the added cost of excavation.

Inclined blast hole drilling methods are coming more into use due to the advancement in design of drilling equipment and in the use of explosives during the past years. These techniques are capable of producing desired slopes in limestones and sandstones as well as providing many additional advantages including a considerable reduction in cost. Slope angles up to 45 degrees can now be constructed without difficulty.

Where presplitting locations are desired these locations are to be noted on the plans giving the stations involved and the names of the formations as determined from the plans cross sections.

SLOPE EROSION

Erosion is a real problem on cut clopes. To reduce erosion, slope in soil and non-durable rock should be flattened as far as possible to allow the products of weathering to stay in place on the slope to increase the fertility and moisture retention for vegetation cover. The interception of drainage at the top of the slope is of prime importance. Weathered shales sometimes appear barren when the erosion is really at fault causing a gullied and washed seedbed. Like any crop, a failure can result in slope seeding. When a crop failure results the slope should be put back in shape and reseeded.

Drainage from gentle slopes above the backslope proper should be prevented from flowing down the slope face by construction of diversion ditches. These ditches should be located some distance behind the top of the slope and in some cases should be lined with impermeable material. Failure of slopes due to earth flowage or sliding is always preceded by development of tensional cracks on the ground behind the slope. Inducing water into these cracks from a poorly located diversion ditch would rapidly worsen the situation.

In many areas, construction of subdrains might be considered behind the top of a slope to cut off the troublesome concentration of seepage which often occurs at the soil mantle-bedrock contact.

RECOMMENDATIONS FOR SEEDING SLOPES

Considerable erosion and consequent siltation of ditches and drainage structures occurs after completion of grading and prior to establishment of an effective vegetative cover. A 50% reduction in drainage structure capacities is not uncommon due to siltation during this period.

Where backslopes and shoulders are to be constructed in highly erodable deposits, consideration should be given to the proper timing of construction to reduce the period of exposure to erosion. In extreme instances, two stage seeding of backslopes and shoulders might be advisable with backslopes and ditches being seeded the fall following completion of grading and shoulders seeded after surfacing is completed. The use of temporary retards might be also considered. At some critical locations, oversize drainage structures might be necessary to assure adequate capacities after siltation.

Seeding should seldom be attempted on shale slopes of 1:1 or steeper which are not stepped or serrated. Shale is of poor fertility and supports vegetation sparsely on such slopes even when mulched. Further, it has been noted that in the past maintenance crews will attempt to machine mow any slope that has vegetation on it. This p ractice sometimes results in unfortunate accidents and almost always results in rutting or tearing up of the mulch or vegetative cover which eventually sloughs off, leaving patches of backslope open to erosion and further destruction.

ECONOMY IN CUTSLOPE DESIGN

As mentioned previously, the factor of economy must be carefully considered in cut slope design.

Through the years there has been a changing evaluation of certain factors in economy of cut slope design.

This has ranged from the original opinion that cut slopes be constructed on the steepest slope allowed for taking of the right of way to later years where possibly slopes were constructed that required excessive demands on right of way and excavation costs.

In the highly steepened slope situation, these initial savings were usually overshadowed by high maintenance costs throughout the lifetime of the road due to the maintenance of ditches, scaling the slope and often repairing the adjacent road surface due to poor ditch drainage.

On the other hand the design of a slope that is overly flattened to require needless right of way and rock excavation is not a good buy. Dollars spent unnecessarily in either of the situations in added maintenance and added construction costs certainly will not buy other new roads.

There probably always will be special situations where the most desirable slope cannot be constructed due to narrow right of way, since acquiring adequate right of way would require purchasing very expensive property or would be too upsetting to the ecological balance. In such cases the engineering geologist and design engineer must work closely together in making the best possible slope design on the right of way available to them. These special practices should be avoided whereever possible and their occasional use should not, in any way, promote use of an erroneous flexible slope design criteria.

It can become easy to oversimplify cutslope design and hurriedly use a rule of thumb standard, that has developed through the years, with the misgiving that it is a good economical slope design.

General slope criteria often used in such cases are as follows: Soil Mantle 3:1; Shales, Common excavation 2:1; Rock excavation 1:1; Limestone and Sandstone (rock excavation), Firm ½:1, or vertical. Along with these slopes a bench might automatically be placed below almost every resistant bed, with any thickness, to catch the fall of loose material. Designing the most stable and economical slope by such simple criteria would be only accidental since there are too many other facets of slope design to be considered to make the best decision.

In the view of economy of cut slopes we have experienced considerable savings of right of way by careful consideration of soil mantle and weathered shale slope design. In areas of rougher topography the placement of rock buttresses in the slope at the base of the weaker seasonally wet mantle materials has often allowed steepening of the mantle slope.

These buttresses are built from the limestone excavation taken from the balances and placed within the slope as the cut is being brought down. Mantle slopes can often be made steeper due to the added stability, internal mantle drainage and the deterrent to slope erosion provided by the buttress. Thus, a savings can be made since many times soil and weathered shale slopes at the top of the slope face are the important determinant for right of way needs.

Slopes built on too narrow right of way are still in evidence due to their instability with silted and/or rock strewn ditches and deteriorated road surfaces caused by improper drainage. Also it is not uncommon to have these unstable slopes eventually develop into a costly legal problem or with additional right of way requirement problems where they encroach upon the adjoining property.

In most of these steep slope situations there are considerable extra maintenance costs over the years. Added to this is the general nuisance, traffic, safety, mowing difficulties and the property damage potential.

It should never be accepted that unstable slopes become stable with time for most often they will require periodic maintenance over the lifetime of the road.

We should, therefore, stress economy, yet emphasize a slope design that weighs heavily toward producing slopes which, for the design life of the roadway, will be entirely free of dangerous rock fall, slippage or slides, will require little maintenance and will be reasonably attractive to look at.

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REINFORCED EARTH FILL ON STEEP MOUNTAIN TERRAIN HIGHWAY 14A, BIG HORN COUNTY WYOMING

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ABSTRACT

The Wyoming Highway Department has utilized Reinforced Earth to contain a large side-hill fill on steep mountain terrain on the construction project for the relocation of portions of U.S. Highway 14 alternate along the west side of the Big Horn Mountains.

The purpose of this report is to discuss the Reinforced Earth technique and its applicability for road construction projects on mountainous terrain where other types of retaining walls are unsatisfactory or where the costs of bridge structures to span the problem areas are prohibitive.

Moderately to heavily jointed granite comprises the foundation at the project site. Preliminary field investigations concluded that joint orientations were generally favorable for stable foundation conditions. Consequently, a bench design for the wall base was used to reduce hard rock excavation quantities. The resulting Reinforced Earth Wall has fulfilled the expectations of the Highway Department to provide a sound fill on steep mountain terrain where the possible alternative would be a costly bridge structure.

INTRODUCTION

Reinforced Earth structures or embankments have been used on road construction projects in many areas in the United States and other countries as corrective solutions where slides have occurred or where poor soils in the foundation required a structural bridging effect afforded by reinforcement of the fill volume.

The Wyoming Highway Department has recently applied the Reinforced Earth technique to retain a side-hill fill on very steep mountainous terrain (1:1 slopes) underlain by fractured, Pre-Cambrian Granite on the construction project for the relocation of portions of U.S. Highway 14 alternate (Figure 1). This section of the new alignment descends the west face of the Big Horn Mountains with a vertical drop of 2316 feet in a distance of 5.3 miles. Consequently, limitations for lateral line shifts exist in certain areas where normal cut-fill construction cannot be utilized because of steep terrain.

REINFORCED EARTH DESIGN

Reinforced Earth consists of an association of earth and reinforcements, the latter being linear elements (metal strips) to withstand important traction forces. At the free boundaries of the structure, it is necessary to provide facing elements or "skins" to prevent earth from flowing away between reinforcements. The earth backfill consists of granular material from sand size to cobbles up to ten inches in diameter. The amount of silt or clay size constituent should not exceed fifteen percent of total quantity. Minimum required internal friction angle is 25 degrees.

Metal and concrete facing elements are available; but because of the difficult terrain and limited access at the project site, easily handled metal facing elements were utilized. For most projects the width of the Reinforced Earth is equal to 80 percent of the wall height. Modification of this width requirement to reduce the amount of hard rock excavation was determined to be feasible, and the cross sectional design then included a stair-step benching arrangement in granite with benches up to fifteen feet wide (Figure 2).

GEOLOGICAL CONDITIONS AT THE SITE

Moderately to heavily jointed granite comprises the foundation at the project site. Preliminary foundation investigations consisted of measuring the attitude (dip and strike) of fracture planes in the granite outcrop and noting the physical characteristics of the planes, particularly, the amount of roughness or associated undulations. These latter features create sliding resistance and are important in an analysis of the overall stability of the rock mass. Stability is also dependent upon the orientation of joint planes, i.e., planes that dip steeply (more than 30 degrees) away from a rock face form a potential sliding surface; and block failure can occur. Planes that dip into the rock face provide stable conditions.

Photo 1 shows typical jointed granite at the project site. Inclined joint planes (a) dip into the hillside and form stable surfaces, while intersecting vertical planes (b) create slab blocks that topple from the outcrop or cut face if external forces act upon the inner joint planes. These forces may be natural occurring, as ice build up in open fissures, or man created, as expansion gases in open fissures during blasting. Relatively stable, near vertical, presplit faces (Photo 2) created during excavation confirm, in general, the favorable joint plane orientations as determined by the initial foundation investigations. However, excavation uncovered occasional decomposed granite (a) underlying unaltered granite; and in several localities this condition created unstable conditions that were corrected by selective rock bolting (b).

Decomposed granite, talus, and carbonaceous soils covered portions of the structure site. A maximum thickness of nineteen feet of this material was drilled in the hillside drainage (center of Reinforced Earth wall) after large pine trees were removed during the site preparation phase. This unexpected thickness of soil made it necessary to modify the original design by shifting the alignment toward the hillside to reduce the quantity of additional facing element needed to establish the base in solid granite.

MATERIALS AND CONSTRUCTION

Excavated granite was stockpiled in the project area and utilized as backfill in the wall after screening the material through a grizzly (Photo 3) to provide backfill that met size specifications set by the Reinforced Earth Company, i.e., 100 percent passing the 10 inch screen, 75-100 percent passing the 4 inch, and not more than 15 percent passing the 200 mesh sieve.

Approximately 82 percent of the total man hour input by the contractor was spent on site preparation, primarily rock excavation. The time consuming special bench design prolonged this phase of operation through the summer and well into the fall of 1974 until inclement weather closed down all work for the season.

Actual erection of the wall began July 21, 1975, and was completed on August 8, 1975, utilizing a front end loader and compactors in the backfill operation (Photo 5). The facing elements were lifted inplace by hand and bolted together two or three lifts at a time (Photo 4). Compaction of each 13 inch lift of backfill material was followed by bolting the variable length reinforcing elements to the face "skins" in accordance with a specified schedule (Photo 6). Wall erection proceeded without problems and at a rapid pace as the crew became familiar with the procedures shown initially by a representative of the Reinforced Earth Company. The facing elements were coated with a forest brown paint after the wall was completed. The color blends with the natural slopes to form an aesthetically pleasing retaining structure conforming with environmental considerations (Photo 7).

Given:

Granite debris backfill with $\gamma = 0.125$ Kips/ft^3 and internal friction angle = 30°

Assume:

Lateral-vertical pressure ratio (Ka) = 0.33

Coefficient of friction (f_{sr}) soil to rock = 0.65

$W_1 = (44.7)(15)(0.125) = 83.8$ Kips

$W_2 = (33.9)(15)(0.125) = 63.6$ Kips

$W_3 = (24)(6)(0.125) = 18$ Kips

$$d = \frac{(83.8)(7.5) + (63.6)(22.5) + (18)(33)}{165.4} = 16.04 \text{ ft.}$$

$$E = \tfrac{1}{2}\gamma H^2 Ka = \tfrac{1}{2}(0.125)(18^2)(0.33) = 6.68 \text{ (Coulomb)}$$

1. Overturning

$$F.S. = \frac{Wd}{Ed_1} = \frac{(165.4)(16.04)}{(6.68)(32.7)} = 12.1$$

2. Sliding

$$F.S. = \frac{f_{sr} W}{E} = \frac{(0.65)(165.4)}{6.68} = 16.1$$

3. Bond Stress (Reinforcing Strip to Soil)

$$\text{Minimum strip length (L)} = \frac{(F.S.)\, P_h}{2\, bf\, P_v}$$

where: F.S. = Factor of safety needed

b = width of reinforcing strip in feet per ft^2 of wall

f = coefficient of friction (soil to strip) = 0.5

P_h/P_v = Ka

a. at base of wall, when b = 0.082, and L = 15 ft.

$$F.S. = \frac{(2)(0.082)(0.5)(15)}{0.33} = 3.73$$

b. at 33.9 ft. depth, when b = 0.055, and L= 30 ft.

$$F.S. = \frac{(2)(0.055)(0.5)(30)}{0.33} = 5.0$$

c. at 24 ft. depth, when b = 0.04, and L = 36 ft.

$$F.S. = \frac{(2)(0.04)(0.5)(36)}{0.33} = 4.36$$

4. Foundation Pressure and Steel Stress

a. at base of wall, where d = 16.04 ft., and L = 36 ft.

$$\Sigma M_{\substack{\text{centroid}\\ \text{at base}}} = Ed_1 = (6.68)(32.7) = 218.4 \text{ Kips}$$

$$e_{\text{from centroid}} = \frac{218.4}{165.4} = 1.32$$

$$e_{\text{centroid from } \mathbb{C}} = 18 - 16.04 = 1.96$$

$$e_{\text{total}} = 1.32 + 1.96 = 3.28$$

Since $e_{\text{total}} < L/6$, $P_v = \dfrac{W}{L-2e} = \dfrac{165.4}{36 - 2(3.28)} = 5.62$ Kips/ft^2

and, $P_h = Ka\, P_v = (0.33)(5.62) = 1.85$ Kips/ft^2

Also at base of wall, there are 3 Reinforcing Strips/10.76 ft^2, and 1 R.S. = 90 x 3mm = 3.543 in. x 0.1181 in. = 0.418 in^2, ∴ Steel Cross Section = 3 R.S. x 0.418/10.76 = 0.1165 in^2/ft^2.

Also, steel stress $= \dfrac{1.85}{0.1165} = 15{,}880$ psi tensile strength

and for A446 steel, tensile strength = 40,000 psi

$$\therefore F.S. = \frac{40{,}000}{15{,}880} = 2.53$$

b. At 33.9 ft. depth, where d = 17.23 ft., L = 36 ft., $\Sigma M_{\substack{\text{centroid}\\ \text{at base}}} = 146.3$, $e_{\text{total}} = 1.78$ and W = 145

Since $e_{\text{total}} < L/6$, $P_v = \dfrac{145}{36 - 2(1.78)} = 4.47$ Kips/ft^2

and $P_h = (0.33)(4.47) = 1.48$ Kips/ft^2

Also at this depth, there are 3 Reinforcing Strips/10.76 ft^2, and 1 R.S. = 60 x 3mm = 2.362 in. x 0.1181 in. = 0.279 in^2, ∴ Steel Cross Section = 3 R.S. x 0.279/10.76 = 0.0778 in^2/ft^2

Also, steel stress $= \dfrac{1.48}{0.0778} = 19{,}000$ psi

$$\therefore F.S. = \frac{40{,}000}{19{,}000} = 2.1$$

c. At 24 ft. depth, where d = 18 ft., L = 36 ft., $\Sigma M_{\text{centroid at base}} = 80.2$, $e_{\text{total}} = 0.74$, and W = 108

Since $e_{\text{total}} < L/6$, $P_v = \dfrac{108}{36-2(0.74)} = 3.13$ Kips/ft^2

and, $P_h = (0.33)(3.13) = 1.03$ Kips/ft^2.

Also at this depth, steel density is the same as at 33.9 ft. depth, and, therefore:

steel stress $= \dfrac{1.03}{0.778} = 13{,}240$ psi

and, F.S. $= \dfrac{40{,}000}{13{,}240} = 3.02$

INSTRUMENTATION

Analyses of instrumentation of Reinforced Earth walls elsewhere illustrates conclusively that the design theory normally utilized for Reinforced Earth is adequately proven. The mathematical analysis of a benched cross sectional configuration also shows that the internal structure integrity of the modified design can be maintained by changing the width and spacing of the reinforcing elements even though the length of the bottom strips are less than 80 percent of the wall height. In view of this, it was proposed to use instrumentation to determine the following conditions in the foundation and backfill; a) the load transferred to the planes of weakness (joints) within bedrock, and b) the ability of the granite backfill material to maintain sufficient internal drainage for minimizing hydrostratic pressures created by groundwater entering the backfill from open joints in the granite back wall or by surface water percolating downward through the granular volume. Federal participation for instrumentation was approved by the Federal Highway Administration based upon the following premise – the behavior of jointed granite foundation in steep terrain under stresses of Reinforced Earth was essentially unknown, because most structures of this type have been constructed full width on unconsolidated material. Periodic monitoring of the instruments will provide data to aid in the design of other Reinforced Earth walls on the Highway 14a relocation project.

Location of instruments is shown on Figure 4, which is a schematic illustration of the basal concrete pad. Load cells were placed at the base of the concrete on a shear zone in granite and on the down thrown side of a joint block with the controlling joint plane dipping 45 degrees south or near parallel to the original ground slope. One load cell was imbedded in the top of the concrete pad to monitor changes in stress of the fill load in the critical area near the base of the wall face. A pore pressure cell is located behind the concrete pad and on the lowest bench in the granite foundation to record hydrostatic pressure, although the six inch perforated underdrain should normally relieve groundwater pressures.

Slope inclinometer tubing at two locations extends through the backfill and into the granite foundation behind the concrete pad. The tubing is one and one half inch I.D. square steel in 20 foot lengths for use with the Soiltest Probe, Model C-350. The inclinometers were installed to detect any shearing at the contact of the reinforced backfill and foundation and also possible tilting of the Reinforced Earth mass as the result of downslope rotational sliding.

Instrumentation indicates that although a small adjustment in the jointed granite foundation is evident because of the fill load, no incipient failure is suggested. The Reinforced Earth mass is effectively bridging any minor settlement indicated thus far by instrumentation. Survey control on top of the wall face by field personnel concludes that any movement that may have occurred is essentially too small to detect.

CONCLUSIONS

The Reinforced Earth wall has fulfilled the expectations of the Highway Department to provide a sound fill in steep mountainous terrain underlain by highly fractured granite where the possible alternative is a bridge structure that would cost at least $325,000. Although the final cost of the Reinforced Earth project was $308,269.00, elimination or simplification of the bench design would probably reduce this cost substantially at other locations on the highway realignment project.

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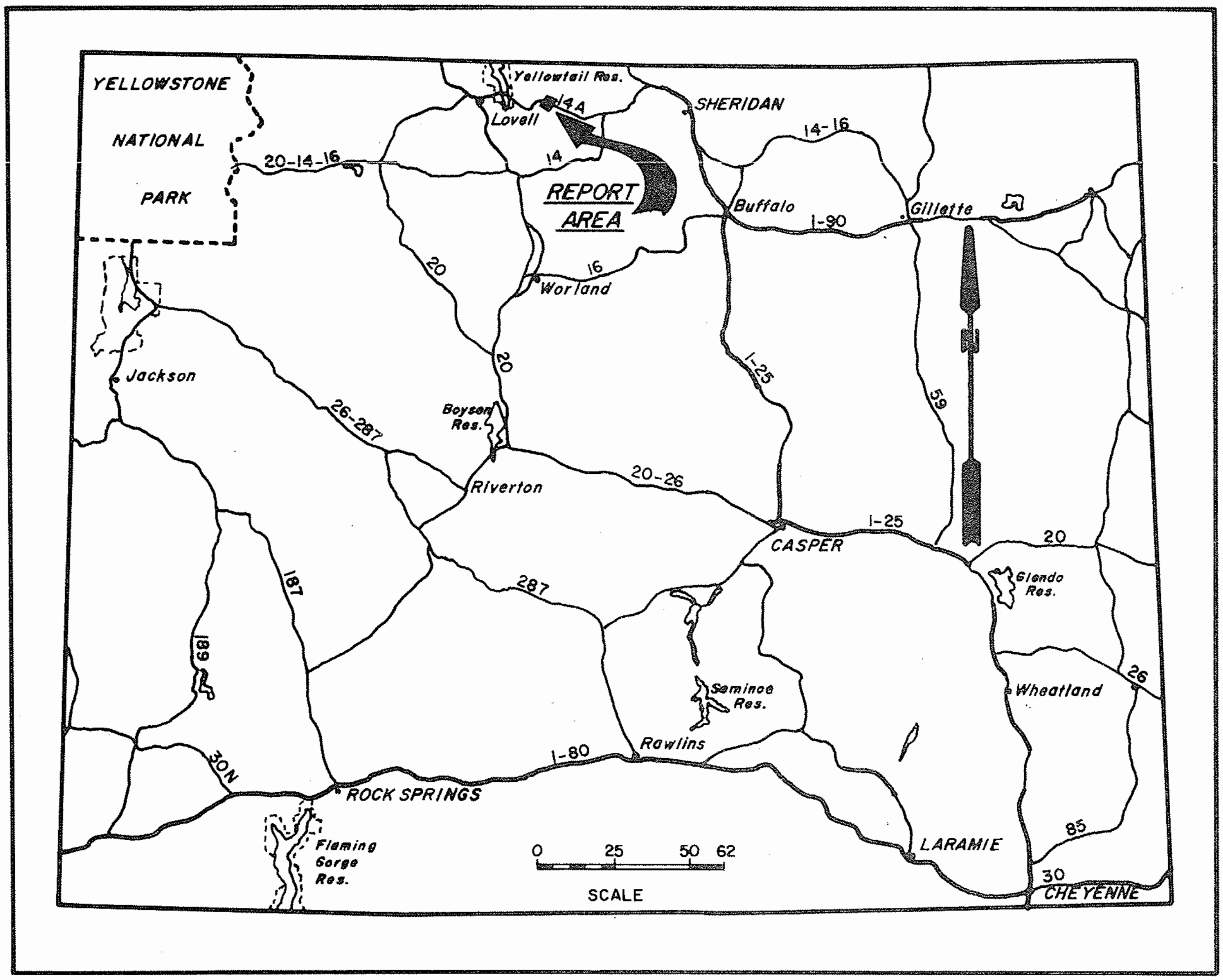

Figure 1 - Index Map of Project Area

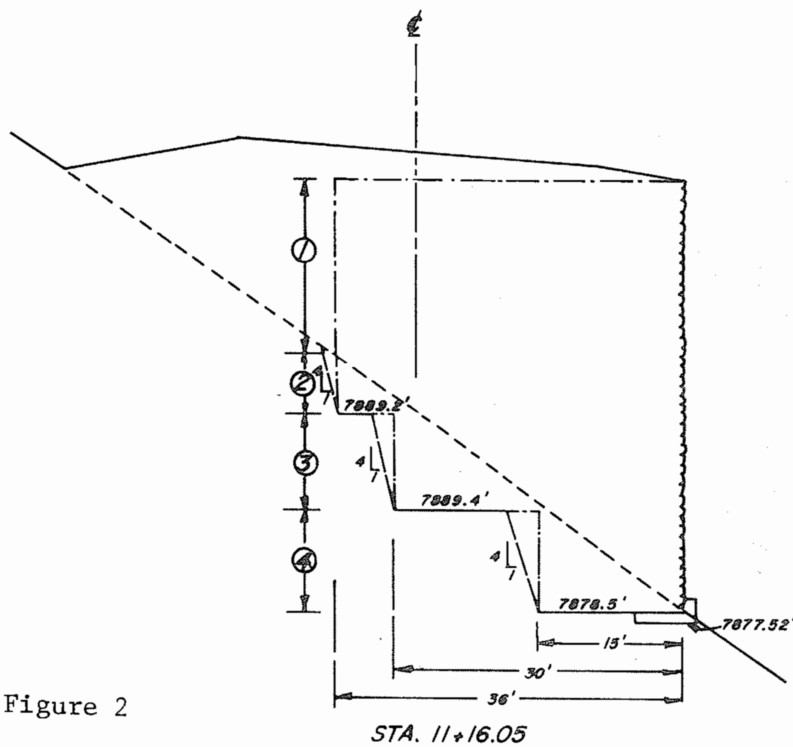

Figure 2

Mathematical Analysis of Stresses[1]

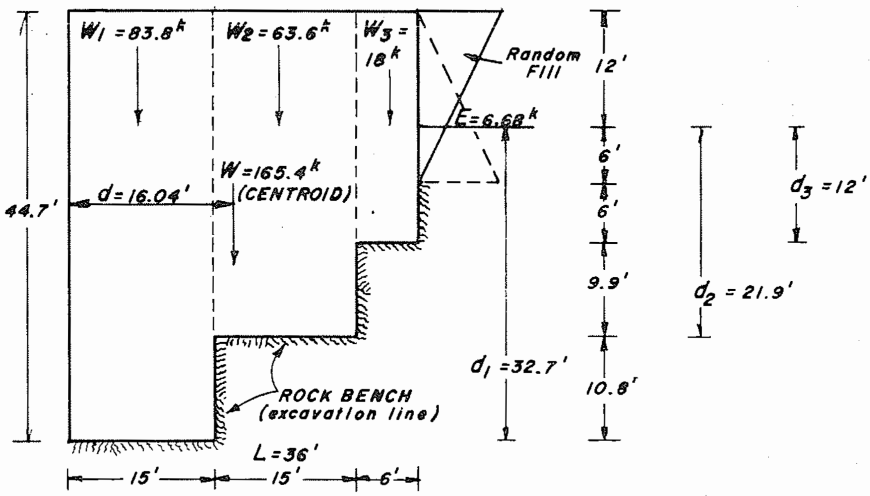

Figure 3. Reinforced Cross Section, Station 11+00

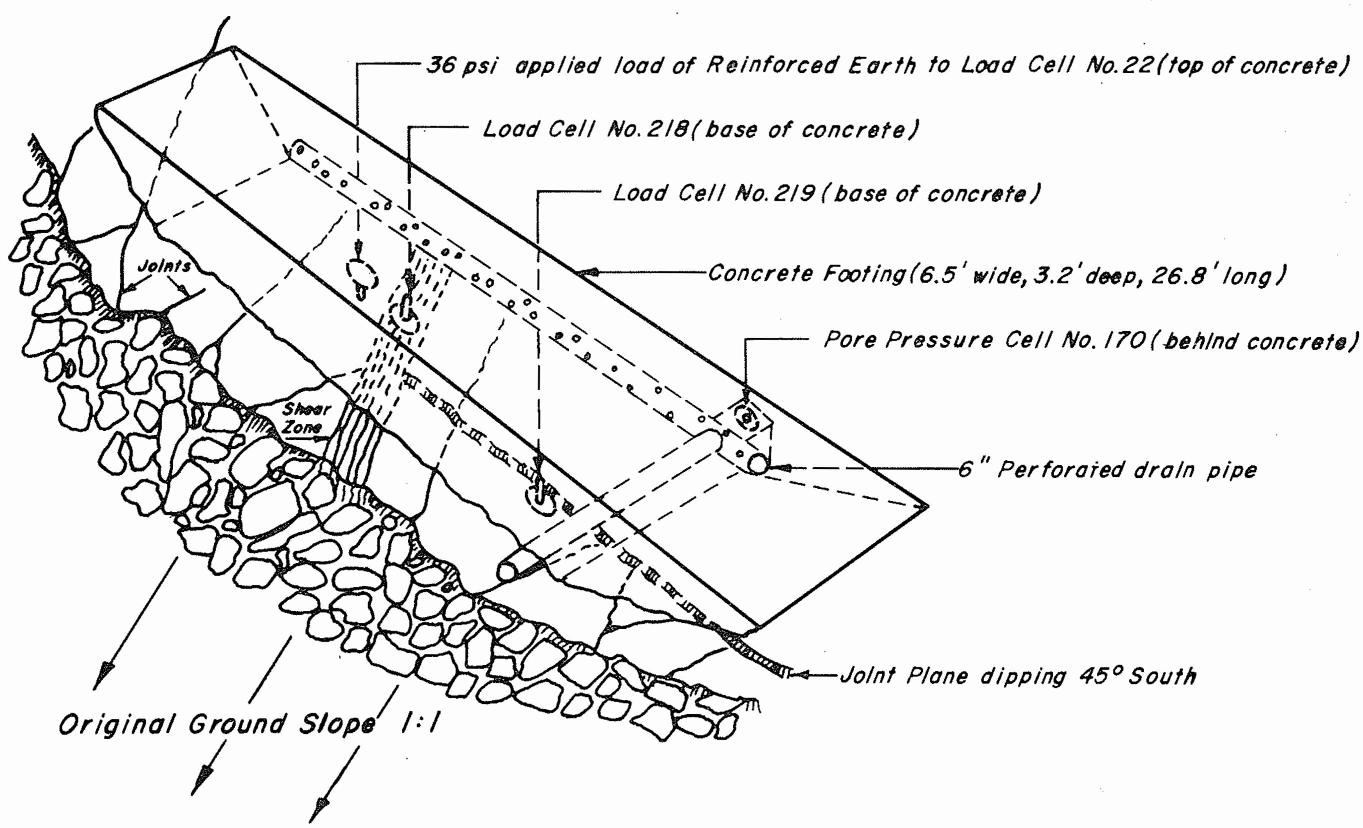

Figure 4. Schematic illustration of concrete pad at base of Reinforced Earth, with location of stress meters.

[1]Reinforced Earth Company

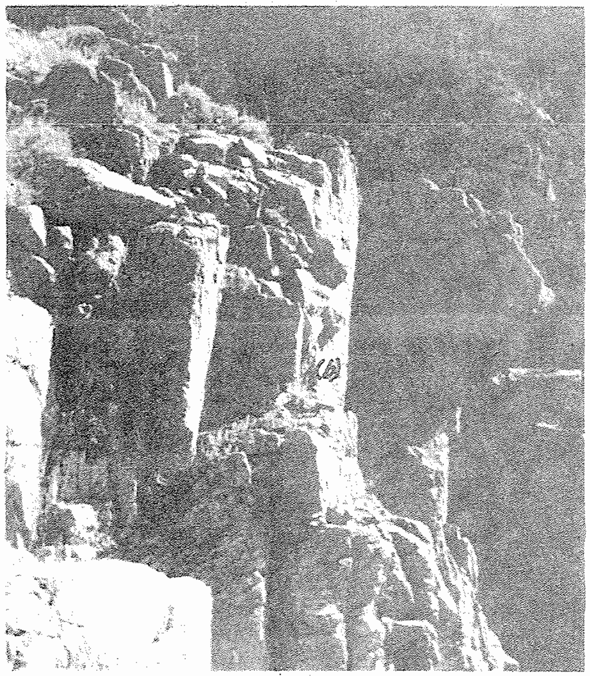

Photo 1

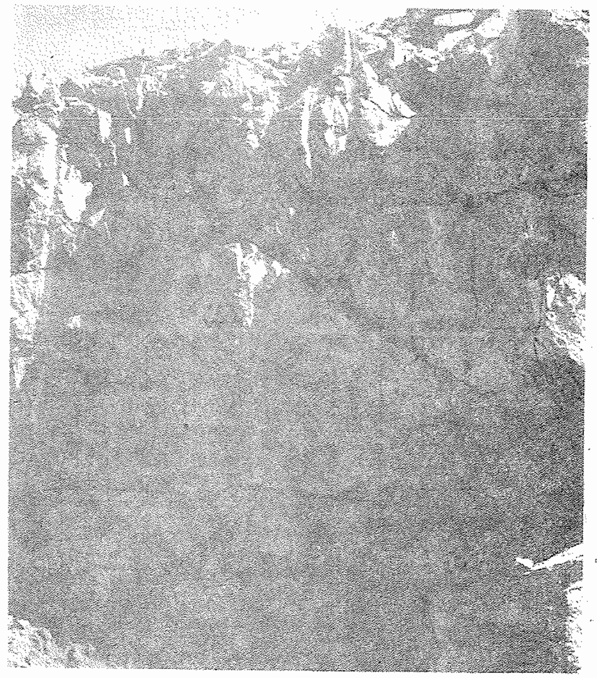

Photo 2

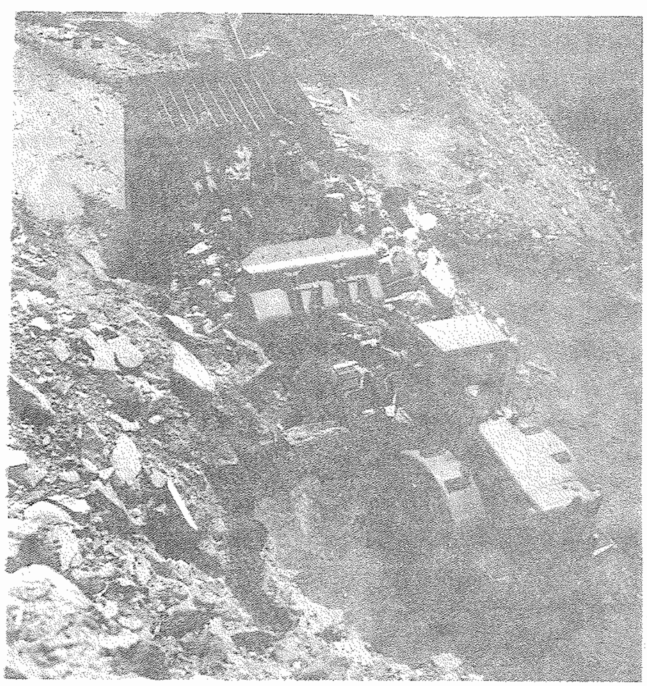

Photo 3

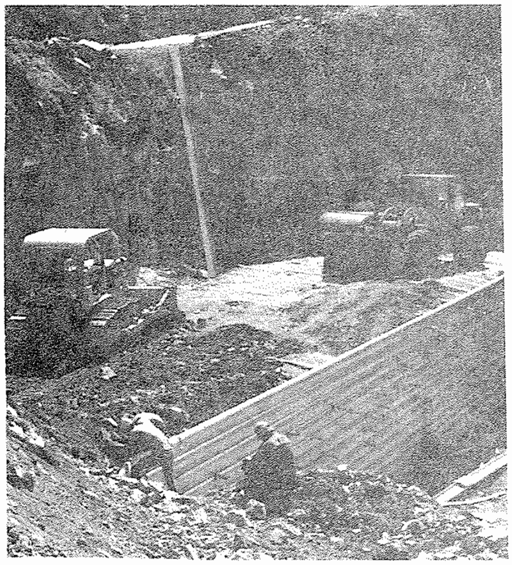

Photo 4. Erection of facing elements and backfill operations.

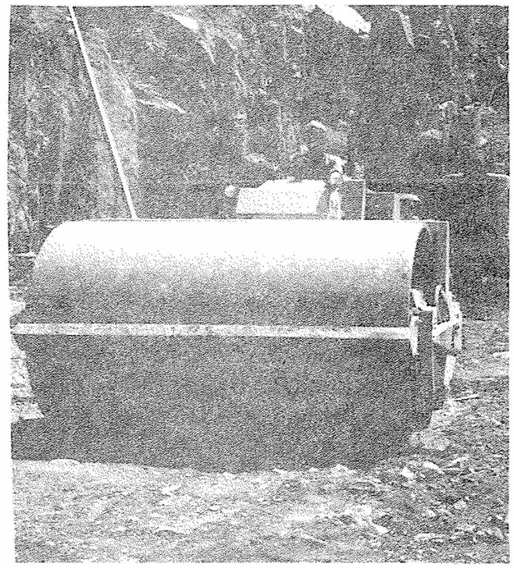

Photo 5. Compaction of backfill material prior to next reinforcing element layout.

Photo 6. Attaching reinforcing elements to 13 inch face "skins".

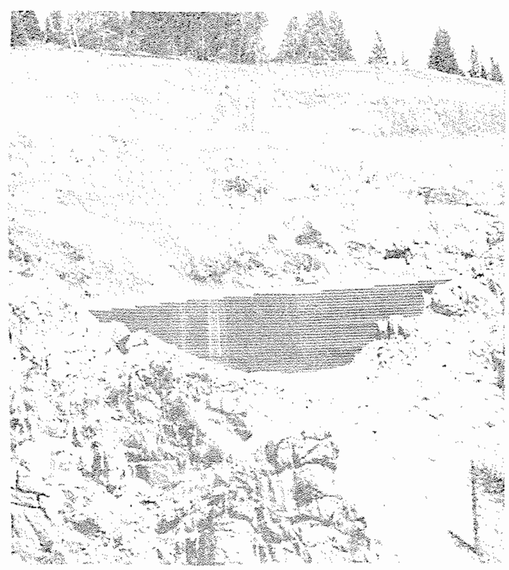

Photo 7. Completed wall coated with forest brown paint.

THE USE OF REINFORCED EARTH WALLS AS BRIDGE ABUTMENTS

By

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Rocky Mountain Region
The Reinforced Earth Company

During the past seven years over 120 structures have been built in the United States utilizing Reinforced Earth as a material of construction. Earlier applications of Reinforced Earth were done as part of the Federal Highway Administration demonstration program and included slide corrections, retaining walls, foundation slabs and bridge abutments. It is the latter application which is being discussed today. In this presentation I will briefly discuss Reinforced Earth abutment concept in general, design procedures and construction methods. Then as an example I will describe a specific project that has been built with Reinforced Earth abutments - the first of its kind in the United States.

A Reinforced Earth abutment is simply a Reinforced Earth wall having been designed to accommodate a highly concentrated loading near its facing system. Reinforced Earth is a composite material formed by the association of soil and reinforcements whereby the mechanical properties of soil are improved by reinforcing it in the direction in which it is subjected to the greatest stresses. The resulting material behaves like a soil which has an anisotropic cohesion, the value of which is directly proportional of the potential strength of the reinforcements.

The Reinforced Earth theory is based on the proven hypothesis that an active Rankine state of stress exists within the Reinforced Earth volume. The earth pressures mainly associated with this stress is taken in tension with reinforcements. The basic mechanism effecting this stress transfer is friction between soil and reinforcements.

Ordinarily a wall would be proportioned such that the width of the reinforced section perpendicular to the face of the wall (or strip length) would be approximately 0.8H. In an abutment, however, with the bridge seat loading, higher stresses occur in the upper portions of the wall so that strip length in these zones is usually greater, and our rule of thumb no longer applies. A roughly square section then is assumed to test the structure under imposed loading for external stability.

An analysis of the bridge dead and live loads is made, and the depth and position of girders is considered so that a bridge seat can be dimensioned. The seat is designed such that the unit loading is limited to a maximum bearing pressure on top of the Reinforced Earth wall. Its forward edge is located at a minimum of 1 foot inboard of the top facing panel. External stability of this unit is examined and safety factors in overturning and sliding are calculated. Next we move to the foot of the Reinforced Earth structure itself where, after having determined external stability we calculate horizontal stresses by means of a resolution of forces and moments producing a resultant reaction. This is reduced to a component vertical stress using the Meyerhof Theorem which allows for a uniform stress distribution over a portion of the base.

The horizontal stresses are determined by applying the coefficient of active earth pressure to the vertical stress. This is done at the base of the structure as well as at various levels within the structure.

At the upper levels of the wall, however, stresses due to the loading of the bridge are the factors that control the design. In this zone the horizontal pressures due to loading of the bridge are maximum along a shear plane which is defined as a Rankine failure surface and are calculated at the intersection of the shear plane and the wall face, five feet below the top of the wall and at the top of the wall itself.

All the calculated horizontal pressure values are then plotted to describe a design pressure envelope. Reinforcements are then designed to provide a cross sectional area of steel per square foot of wall face to accommodate these stresses with an adequate factor of safety. This is done by varying the width and/or the density of strips at various levels within the wall to economize the amount of steel being used. These densities, widths and lengths of reinforcing steel are then detailed on the design drawings. Friction plays a large part in determining the length of the steel in the upper portions of the wall.

As a proprietary system the Reinforced Earth Company not only designs but furnishes the materials of construction as a material supplier during the construction of the abutments. The construction of a Reinforced Earth abutment using a four or five man crew usually proceeds at rates averaging 1,000 to 1,200 square feet of finished wall per day. This is a function of the contractor's rate of haul, placement, and compaction of the granular backfill material. Since the Reinforced Earth fill is built up in the same manner as an embankment, this rate relates to the equipment spread and earth work management techniques of the contractor. The placement and erection of the facing system and the reinforcing strips is done behind the face of the wall at workman level throughout construction. It is a repetitive process and normally can keep well ahead of the earthwork operation. Progress and erection costs are not dependent on wall height.

The project to which I will be speaking is located on I-80 in an area where it crosses Big Meadow Ranch Road, northeast of Reno, Nevada near Lovelock. Here an overpass was required to span a ranch access road. Normally this would have been a routine construction project until a structure foundation investigation exposed an unusual foundation situation where weak, compressible soils occurred in deep layers.

In a recently published paper entitled **Reinforced Earth Bridge Abutments** prepared by Dale S. Mosher of the Nevada Department of Highways, published at the Second Southwest Geotechnical Engineer's Conference in Las Vegas, Nevada, March 22 and 23, 1977, the soil and subsurface conditions at the site were described in detail. The factors leading to the selection of the Reinforced Earth Abutments were described and the results of the installation were outlined.

In summary, that study pointed out the following: The location of the highway was not flexible at the time that the poor foundation conditions were discovered. Throughout the Lovelock Valley, there is an upper strata of fine sandy silts and clays, low plasticity silty clays, and clayey silts to a depth of 40 to 50 feet. Next lies a 50 foot clean sand strata, and below that extending past 200 feet there are highly compressible organic and inorganic clays and silts having low permeability. Depth to ground water surface averages approximately 12 feet.

The founding of a conventional three span structure on piles driven into the clean sand strata was rejected mainly due to the possibility of settlement in the lower compressible soils. Longer piles were also ruled out. Approach fill settlement was also a major problem and light weight fill materials were found not practical as were sand drains.

It was finally decided to preload by construction of the embankment, which included filling over the proposed bridge site. Later the crossing site was to be excavated after primary consolidation had taken place, and then a short span structure would be built. The secondary consolidation would have been damaging to a conventional reinforcing concrete structure or binwall. Therefore, the Reinforced Earth wall was selected with its flexibility in differential settlement as an important factor in the decision.

During the stage of construction that primary consolidation was to take place (two years) the embankment was instrumented and evaluated as a test fill for the Lovelock Valley. The fill was constructed in stages of two vertical feet per week. As predicted the primary consolidation was 90% complete in the two-year loading period and the settlement rate decreased to 0.2 feet per year.

The Reinforced Earth structure that was designed for the Big Meadows crossing was to support a precast bridge span of 70 feet. The bridge seat was designed to be placed 1 foot back from the facing panels with a total width of 2 feet 9 inches and a uniform bearing pressure of 3 KSF. Stresses at the base of the Reinforced Earth structures were determined to be about 1.5 KSF and maximum unit horizontal stresses in the zone of influence of the bridge seat were approximately 1.4 KSF. Reinforcing below the bridges was designed to accommodate these stresses and to develop adequate friction to resist the higher than normal stresses in the upper zone of the walls. Slightly beyond the bridges, in the wing walls and in the median, reinforcing was designed to accommodate only horizontal stress due to earth pressure.

The footing of the Reinforced Earth structure, as in most similar structures, was a 6 inch x 1 foot reinforced concrete leveling footing. Its function is to give proper alignment and grade of the first row of facing panels as they are set. This is by no means a rigid footing, and it is important that the footing be allowed to break up as differential settlement should occur along the length of the structure. Since secondary consolidation is to take place over a period of twenty years a limited settlement of the structure is anticipated. Each panel in the facing system is relatively free of contact with surrounding panels so that a slight articulation can occur which will accommodate a differential settlement over the length of the structure (the Reinforced Earth behind the facing system is a flexible composite material and, therefore, the facing system must also exhibit flexibility).

The phase of the project that included Reinforced Earth walls was bid in June, 1974, and the successful contractor was J. C. Compton Construction Company of McMinnville, Oregon. Construction was begun in July, 1974 after 50 feet of the embankment on both sides of Big Meadows Road was excavated. The 8,079 square feet of abutment walls were completed in August. The bridge seat was then placed and cured atop the abutments and the superstructure beams were set. The entire structure was completed in September, 1974 and the roadway was open to traffic shortly thereafter. Since the placement of the Reinforced Earth abutments, there has been further settlement of approximately 0.5 feet with the predicated further settlement of 0.5 feet within the next twenty years. Settlement is occurring primarily between the shoulders of the freeway with little movement outside the fill slope. This project has been considered very successful by the Nevada Department of Highways, and another similar structure is being planned in the Lovelock Valley.

I will describe briefly, at this point, other Reinforced Earth abutment projects that are actively being pursued.

RENO JUNCTION, WY – The Wyoming Highway Department has recently completed an abutment for a single bridge on a Wyoming State Highway near Reno Junction (about 40 miles south of Gillette) for a crossing over a haul road for the Black Thunder Coal Mine. This 90 foot steel span is set back on the abutments so that a clear distance of 78 feet is provided between Reinforced Earth walls and a minimum clearance of 30 feet is provided for the large haul vehicles to travel under the structure.

I-40 GALLUP, NM – This is a project recently bid by the New Mexico State Highway Department in providing a solution to a weak foundation situation similar to that found in Lovelock Valley. Strata of weak compressible material of a former channel Rio Puerco underlays the location of the major portion of I-40 through the middle of the town of Gallup. Three grade separations are involved in this location; two of which are over streets, and a third is a crossing of a Santa Fe rail spur. This project is to be constructed in two stages, the first of which will be the construction of the embankment including the Reinforced Earth walls at the crossings. A six month period of no activity will follow, where primary consolidation of up to 2 feet is expected to occur. Sixty percent of the total ultimate consolidation will occur during this period with further settlement expected over a twenty year period. Final elevations of bridges with respect to the top of the Reinforced Earth abutments will be established after the initial settlement period, and structures and remaining roadway will be constructed.

ANCHOR DAM, WY – The first U.S. stream crossing of a bridge with Reinforced Earth abutments was recently bid in Wyoming in a remote site near Thermopolis. The abutments are small with maximum height of 15 feet each having sloped wing walls angled at 45 degrees. The principal consideration was to enable the construction of a "prefabricated" bridge system whereby abutments and superstructures could be taken to the site and rapidly erected with a minimum of cast-in-place concrete. The ability to quickly erect such a structure has considerable cost advantages over past projects using conventional designs in the same general location.

LIBBY DAM, MT – The Corps of Engineers is in the process of designing two crossing structures for a haul road to be used in the construction of Libby Dam. One structure crosses the Burlington Northern main line and another crosses a Montana State Highway. Both have approximately 43 feet of clearance between abutments. One of the primary considerations here is that the haul roads, being temporary, will be relocated after five years, therefore, it will be economically important to be able to reclaim and reuse the bridge materials including abutments. It is intended that at the end of the haul road's life the structure will be disassembled, the Reinforced Earth panels will be excavated to free them of the strips and the structure will be reconstructed elsewhere with only new reinforcing strips as a further requirement.

In summary, the experience of highway departments and other agencies with Reinforced Earth abutments has shown apparent technical as well as economical advantages in conditions where settlement, remote location, or reuseability are primary concerns. Often there has been demonstrable economy in the elimination of end spans. There are two projects on this nature completed; seven under construction, and twenty in the design stages. It is my prediction that we will see a great number of these strutures being built in the next few years.

The following 35 mm slides were presented with this paper.

1. Heart-O-Hills Slide Correction, Washington
2. Lovell-Burgess Retaining Wall, Wyoming
3. Route 202, Two-Way Slab, Pennsylvania
4. Lovelock Abutment, Nevada
5. Reinforced Earth Cross-section (typical)
6. Reinforced Earth Cross-section (proportion)
7. Abutment Cross-section
8. Reinforced Earth Cross-section (vertical stress)
9. Reinforced Earth Cross-section (horizontal stress)
10. Abutment Cross-section
11. Reinforced Earth Panels
12. Construction of Reinforced Earth Wall

13. Granular Backfill Placement

14. Erection of Panels

15. Lovelock Abutments, Nevada

16. Lovelock Cross Section

17. Reinforced Earth Concrete Leveling Footing

18. Cork Joint Membrane

19. Lovelock Abutment under Construction

20. Reno Junction Abutment, Wyoming

21. Reno Junction Abutment, Wyoming

22. I-40 Gallup Abutments (design), New Mexico

23. Anchor Dam Abutment (design), Wyoming

24. Libby Dam Abutments (design), Montana

25. European Abutment

Information pertaining to the above slides can be obtained from the author at the following address:

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THE GEOLOGY AND CONSTRUCTION TECHNIQUES OF THE SECOND HAMPTON ROADS CROSSING, NORFOLK, VIRGINIA

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The first Hampton Roads Bridge Tunnel between West Ocean View Street in Norfolk and the Hampton Shoreline near Old Point Comfort in Hampton was constructed during the period November, 1954 through November, 1957 (see Figure 1). The project was financed by part of a 95 million dollar bond issue and replaced the Newport News and Old Point Comfort ferries. The crossing consists of 23 tubes totaling 6,859 ft. in length, 2 man made islands, open approaches each approximately 600 feet in length and 2 approach trestles, the north trestle is 3,250 feet in length and the south is 6,150 feet. The elimination of tolls in 1974, due to retirement of the bonds indicated an increased traffic volume, consequently, the need for a second crossing.

The tunnel portion of the project consists of prefabricated tubes, cut and cover box sections, ventilation buildings and open approaches (see Figure 2). The 23 prefabricated tubes were sunk in place in a dredged trench and backfilled to provide a 5 foot minimum cover over the tunnel. At each end of the prefabricated tube section, box sections, which are each about 90 feet long and constructed by the cut and cover method, connect the prefabricated tube sections to the ventilation building structures. The open approaches carry the tunnel roadway to the trestles section.

The tunnel roadway descends on a 4.0 percent grade from a roadway elevation of +16.4 mean sea level at the north trestle to a low point of elevation -105.4 MSL in the tunnel, ascends on a grade of 0.5 percent under the main channel, and then continues on an ascending grade of 4.0 percent to a roadway elevation of +16.4 MSL at the south trestle (see Figure 3). The widths of roadway through the tunnel and open approaches is 23 feet between curbs and the minimum vertical clearance between the roadway and the ceiling in the tunnel is 14 feet.

Electric power for ventilation, drainage, and lighting is provided by submarine cables from each shore, and water for fire protection and cleaning is piped over each trestle. The roadway within the tunnel is a flexible bituminous concrete pavement on a reinforced concrete slab, and the interior finish consists of ceramic tiled walls and a porcelain enamelled aluminum pan ceiling.

The trestles both north and south are precast, pretressed beams 50 feet long, supported on precast, pretressed concrete pile bents. The width of roadway is 30 feet between curbs, thus permitting traffic in either direction to bypass disabled vehicles. Two feet wide safety walks are provided on each side of the trestles and the concrete parapet walls are surmounted by aluminum posts and railings.

The second bridge tunnel is located inshore (west) of the first crossing, with parallel trestles joining enlarged portal islands (see Figure 4).

Hampton Roads is a large naval and commercial harbor in the Chesapeake Bay which falls in the Coastal Plain of Virginia. The Bay is about 10,000 years old. Its origin came during early Pleistocene times when the Atlantic Ocean advanced westward and drowned the valleys of the Susquehanna River. The tidal shore-line of the bay and its tributaries is estimated at 4,600 miles with a surface area of about 4,300 square miles. The waterways have a combined navigable length of approximately 1,800 miles.

Geologically, the Coastal Plain dips gently eastward at about 5°. It continues east beneath the Atlantic Ocean for about 200 miles where the continental shelf forms (see Figure 5).

The bedrock of the Coastal Plain is composed of partially consolidated detritus. The rocks consist of Cretaceous, Eocene and Miocene marine sands; commonly containing a large percentage of glauconite, gravels, clays, coquina and diatomaceous sediments. In turn, these sediments are sporadically overlain by Pleistocene and recent sand and gravel deposits.

The Paleocene period followed the Cretaceous. There is no evidence that indicates Paleocene sediments were deposited, which suggests a period of erosion or withdrawal of the sea. It has been argued that possibly the lower and older Eocene sediments may represent the Paleocene Period.

The Eocene began approximately 60 million years ago during another period of advancing seas. Depths of the sediments from the Eocene range from 150 feet in the west to 850 feet between Norfolk and the Cape Charles. The sediments are glauconitic sand beds with discontinuous interbedded clays. The formation abounds in fossil shark teeth and whale bones.

The absence of the Oligocene material in the column indicates another era of either nondeposition or active erosion in recessed seas.

To the east, around Norfolk, the Miocene sediments reach thickness greater than 900 feet. The composition of the Miocene formation vary from sand to diatomaceous clays with some glauconitic clays and shell marls. The formation abounds in fossil remains of whales, seals, porpoises, mollusks, shark, ray and other fish.

The most recent system of rocks came during the Pleistocene Period. They may be simply described as unconsolidated, yellow cross bedded sand and gravel up to 100 feet thick near the Eastern Shore with a veneer of sand, silt and gravel overlying it.

The surface is only slightly dissected by streams. Swamps and marsh lands are common, with the Great Dismal Swamp being the largest. Because the coastline is so deeply cut by bays and estuaries, the appearance has been described as a "fringe of peninsulas". Bluffs up to 100 feet high commonly border the estuaries.

The stratification of the silt-clay and upper sand deposits was explored during the design of the first Hampton Roads Crossing by 46 borings (see Figure 6). Undisturbed samples were secured and tested to determine the soil properties. An additional program, consisting of 16 borings, was undertaken to further verify the soil profile. The later program substantiated the first. These indicated that the north 2,000 feet of the tunnel, portal island and trestle would be founded on sandy soils and would not present any special design or construction problems.

To the south, however, there is a thick layer of compressible organic silty clay which in turn is overlain by sand. The elevation at the bottom of the silty clay ranges from -30 feet at the south trestle to -130 at the deepest portion of the channel. Dense sandy soils are found below the silty clay. The compressible material was a problem at the south Portal Island (see Figure 7).

The properties of the silty clay strata were investigated by means of field vane shear tests and laboratory tests on undisturbed samples, consisting of unconfined, undrained triaxial tests. A shear strength profile was developed from the data. Strengths varied from 300 psf, at the top to 800 psf at the bottom of the silty clay layer.

Compressibility of the silty clay was determined by consolidation tests in conjunction with evaluation of settlements observed at the final South Portal Island. The tests indicated that the silty clay layer was highly compressible and the required load would result in substantial settlements. Furthermore, the rate at which consolidation occurs is very slow; therefore, any design which is dependent on preloading would require some means of accelerating the consolidation.

As the construction of the new South Island would impose large additional loads on the compressible subsoils, then 3 schemes were studied to determine the method that would result in minimum cost and risk to the first island.

These schemes as discussed in the following sections are (1) the sheetpile trench method, (2) the undercut and backfill method, and (3) the sand drain and surcharge method (see Figure 8).

(1) With the sheetpile trench method of construction, the new tunnel and open approach structures would be located about 80 feet inshore (west) of the existing center line of the tunnel on the South Portal Island. Construction of the new facilities would be carried out within an open sheetpile trench parallel to the existing tunnel and within the confines of the existing dredged and filled island. This method would limit the need for extending the existing island and hence limit the new loads resulting from sand fill on the compressible substrata. However, the cofferdam methods requires the alignment on the new tunnel to be so close to the existing tunnel that the trench excavation would uncover the top of the existing tube. Therefore, any further consideration of this method was abandoned because of the high risk of disturbance.

(2) The construction of the original South Portal Island was accomplished by the dredging and disposal of 90 feet of compressible materials, and hydraulically backfilled with sand. Since completion of the construction 20 years ago, settlements have been within the predicted acceptable values, and have resulted in minimal corrective measures to the island and its protection, with no significant damage to the structures. Enlargement of the existing island by dredge and fill methods would require that the center lines of the existing and new island be 300 to 400 feet apart in order to avoid problems of slope stability during the dredging of the enlarged island. This method of construction, with suitable separation of the island, presented no undue hazards to the existing structures, but would require the removal and disposal of about 1,000,000 cubic yards of silts and replacement with approximately 1,500,000 cubic yards of hydraulic sand fill. Disposal at sea of unsuitable material is no longer economical and disposal in the Corps of Engineers Craney Island disposal area 5 miles distant requires a substantial rehandling charge. Further, the hydraulic fill to a depth of 20 feet below the new structures will require vibro-compaction for a distance of 25 feet each side of the tunnel center line. The total estimated cost is $5 million.

(3) Widening of the existing island utilizing the sand drain and surcharge method of construction would require placing hydraulic fill between the existing bottom (elevation -15 msl) and the top of the island (elevation +12 msl) within the enclosure of previously placed stone dikes. This procedure will limit silting and provide a dry land surface from which sand drains can be installed prior to surcharging to elevation +37 msl. After completion of the consolidation period, the surcharge would be removed and used as tunnel backfill. Settlement analyses were made using the Boussinesque Theory, and these were verified by the Finite Element Method using computer techniques. These studies indicate negligible additional settlement under the existing structures when the center lines between islands are 250 feet apart. This method would require 1,350,000 cubic yards of fill, 250,000 cubic yards of surcharge removal, 525,000 lin. feet of sand drains, and instrumentation. Estimated cost: $4.05 million.

Upon evaluation of the above factors it was decided to use the surcharge and sand drain method of construction. No dredging and disposal of unsuitable material is required in this method. However, the dredge and fill method is a positive and relatively foolproof plan while the sand drain construction and operation require considerable care and control of technique to produce completely satisfactory results. The South Island was constructed under a separate advanced contract to permit, under a subsequent contract, the trench to be dredged and deposited to construct the North Island and the tubes to be fabricated while the 12 months of surcharge on the South Island progressed.

The problems after the selection of the surcharge and sand drain method, became the criticality of design, care in installing the sand drains, and the evaluation of field instrument readings.

In general terms, the soil profile at the South Island site consists of a thin layer of loose silty sand at the bay bottom at about elevation -15. This is followed by a layer of clayey silt with seams and pockets of sand down to elevation -90. The lower portion of the silty clay contains some organic matter. Below the silty clay are layers of dense silty sand and sand (see Figure 9).

The clayey silt is slightly overly consolidated at the upper portion and normally consolidated lower down. It varies gradually in consistency from soft at the top to stiff at the bottom.

The presence of this clayey silt layer presents problems of stability and settlement.

Laboratory tests of undisturbed samples of the clay silt were made in 1954 for the first bridge-tunnel. Additional laboratory tests as well as field vane tests were made in 1969. The 1954 tests consisted of unconfined compression tests and triaxial tests. Only the 1954 unconfined compression test results were used for the design.

In the 1969 test 2 consolidated, undrained triaxial tests were made at chamber pressures equal to ½ the overburden pressure. These yielded results lower than those of the unconfined compression tests of the same sample tubes.

A plot of peak cohesion results versus depth showed that soil cohesion increased with depth. From this finding, plotting values of cohesion were assumed for design. The basis of the assumption was to follow the average lower limit of the test results, neglecting extreme values.

From a plot of Atterberg limits and natural water content versus elevation and stress-strain curves for laboratory and vane shear tests, it appeared that the clayey silt was quite sensitive, particularly in the upper layers.

Consolidation tests were performed during both the 1954 and 1969 investigations. Consolidation curves were produced by plotting preconsolidation pressure and existing overburden pressure versus depth and coefficient of consolidation, Cv, at applicable pressure ranges, versus elevation.

From these tests it appeared that:

1. The clayey silt is somewhat overly consolidated from its surface at -15 to elevation -50 and normally consolidated below that.

2. The time rate of consolidation is much higher for the soils above -50 than for those below.

For consolidation by means of sand drains, it was conservatively assumed that the horizontal coefficient of consolidation is equal to the vertical. Field permeability and pumping tests were performed during the initial stages of construction. It was planned that if it was confirmed that the horizontal coefficient of consolidation was substantially greater, the sand drain spacing would be revised. The sand drains were installed by jetting for minimum displacement, minimum smear.

On the basis of the relationship of the cohesion test results with overburden pressure and preconsolidation pressure, it was assumed that

$$C = 0.3\ p.$$

The initial cohesion is the product of existing preconsolidation, whether under existing overburden pressure or other geological preconsolidation pressure. When the soil consolidates under the imposed load to a value higher than its initial cohesion, an increase in consolidation will take place.

The devices used to monitor the consolidation of the in situ foundation material were: (see Figure 10)

Casagrande Piezometers
Pneumatic Piezometers
Deep Settlement Points
Control Stakes
Inclinometer Casings

Slope stability analyses were performed for various stages of the proposed construction. These consisted of complete solutions of the Swedish Slide Circle and manual solutions of sliding wedges.

The minimum factors of safety are tabulated below:[1]

It should be noted that an approximate slope stability analysis by means of Taylor's charts indicates that the fill to +12 requires the same average cohesion value as did the excavation for the existing South Island. It was reported that the excavation was made without any problems of instability.

For fills above elevation +12, the increase in cohesion due to consolidation of the sand-drained area was considered in the stability analysis.

The factors of safety tabulated above are considered adequate in view of the fact that the fill was kept under careful observation. It should be noted that the lowest factors of safety occur at early stages in the construction. For this reason, remedial action such as extending berms will be possible if the need is indicated.

Rather than introducing a mandatory waiting period of 3 months, which would interfere with the contractors' schedule, the contract specifications provided that the engineer could direct suspension of all fill operations when the surcharge had been placed to the critical elevation of +20, should field observation indicate a need for additional consolidation time. A separate pay item for costs of delay time was provided. The decision whether to use the delay time depended on shear test results from borings drilled after the installation of the sand drains.

When the island fill reached elevation +12 piezometers were installed and read periodically. The sand drain installation proceeded using the spacing computed from low permeability values.

The new island consists of hydraulic fill confined within a rock dike and separated from it by a filter course. The rock dike with the filter course was placed on top of an underwater gravel blanket (see Figure 11).

In the past, an island of this nature would have been constructed by placing the hydraulic fill first and then protecting it with rip-rap. This procedure invaribly pollutes the surrounding water, therefore, the dike was first placed around the entire perimeter of the new island and hydraulically filled inside. Color aerial photos were taken regularly during this operation, but no pollution was detected.

During the filling operation, periodic settlement readings were taken from settlement plates and deep points (see Figure 12). The effect of the sand drains were clearly distinquishable in the records with a total of 13 feet of final settlement (see Figure 13).

The island covered 23 acres with 17 acres in sand drains. Six thousand drains averaging 100 feet in depth, on 3 feet - 11 feet centers were installed.

Conferences with the U.S. Army Corps of Engineers, the U.S. Navy and U.S. Coast Guard established that adequate dredging spoil and borrow areas were available for the project. The U.S. Government has a disposal area known as Craney Island about 5 miles from the project that can accommodate pumped or bottom dump material. Approximately 100 acres of sand located just east of the South Island was established as suitable borrow.

The locations of leased oyster beds and public fishing areas in the vicinity of the project presented a problem to the dredging operation.

1

Maximum Fill Elevation	**Minimum F. S. which includes fill to the maximum elevation**	
	Slide Circle	**Wedge**
+ 5	1.09	1.17
+12	1.18	1.23
+20	1.37	1.20
+30	1.44	1.65
+37	1.27	1.27

In addition to a survey of oyster bed ownership and yield, the Department requested the Virginia Institute of Marine Sciences (VIMS) to make a study of the shellfish population and to determine what effect the tunnel construction would have on these. VIMS made such a study before, during and after construction and reported that there was no noticeable effect on the bottom conditions (see Figure 14).

When the instrumentation indicated that the initial settlement has been achieved at the South Island, then the surcharge was removed by mixing the sand with water and pumping it across the bay with submerged lines to the North Island Site. The North Island had the same design except it had good foundation. A problem developed at the North Island when the cut and cover section was being made to transition the trestle into the tunnel. The island was hydraulically placed in such a fashion that any silt was pushed ahead of the filling operation. Unfortunately the island was so narrow that some of the silt was captured in the area of the transition and created a problem when de-watering for construction.

The photographs (following figures) illustrate the techniques by which the tubes for the tunnel were fabricated, filled and sunk into position.

Traffic counts indicate a 41% increase since the tolls were removed consequently the predications are reality.

The Department is making preliminary engineering studies for another similar crossing to Hampton Roads between Newport News and Norfolk, about 5 miles to the west of the present tunnels.

Acknowledgments

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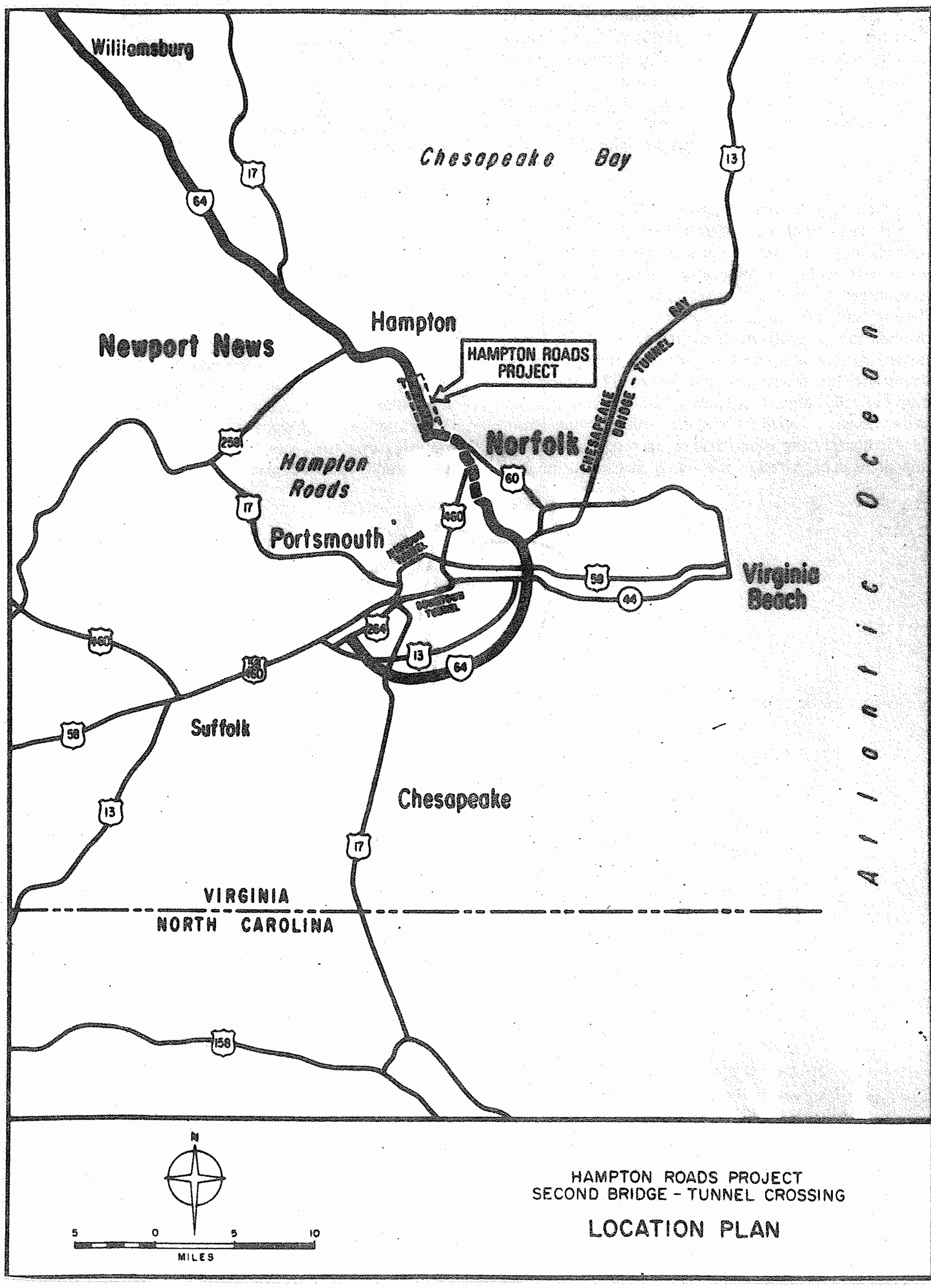
Williamsburg
Chesapeake Bay
17
64
13
Hampton
Newport News
HAMPTON ROADS PROJECT
BAY
TUNNEL
BRIDGE
CHESAPEAKE
258
Norfolk
Hampton Roads
60
17
460
Portsmouth
Virginia Beach
58
64
460
264
13
64
58
460
Suffolk
58
13
Chesapeake
17
Atlantic Ocean
VIRGINIA
NORTH CAROLINA
158
N
5
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5
10
MILES
HAMPTON ROADS PROJECT
SECOND BRIDGE - TUNNEL CROSSING
LOCATION PLAN

FIGURE 1

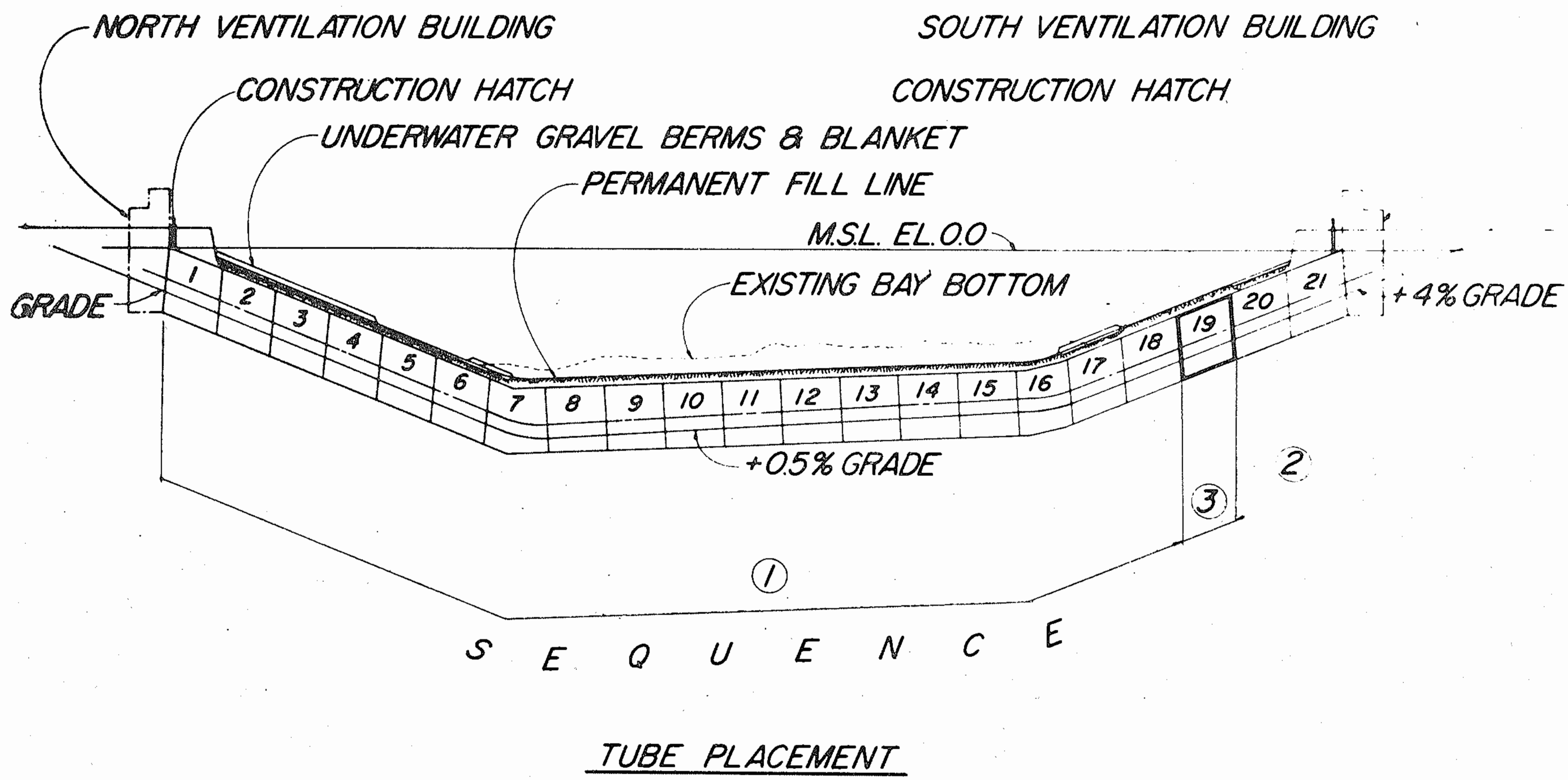
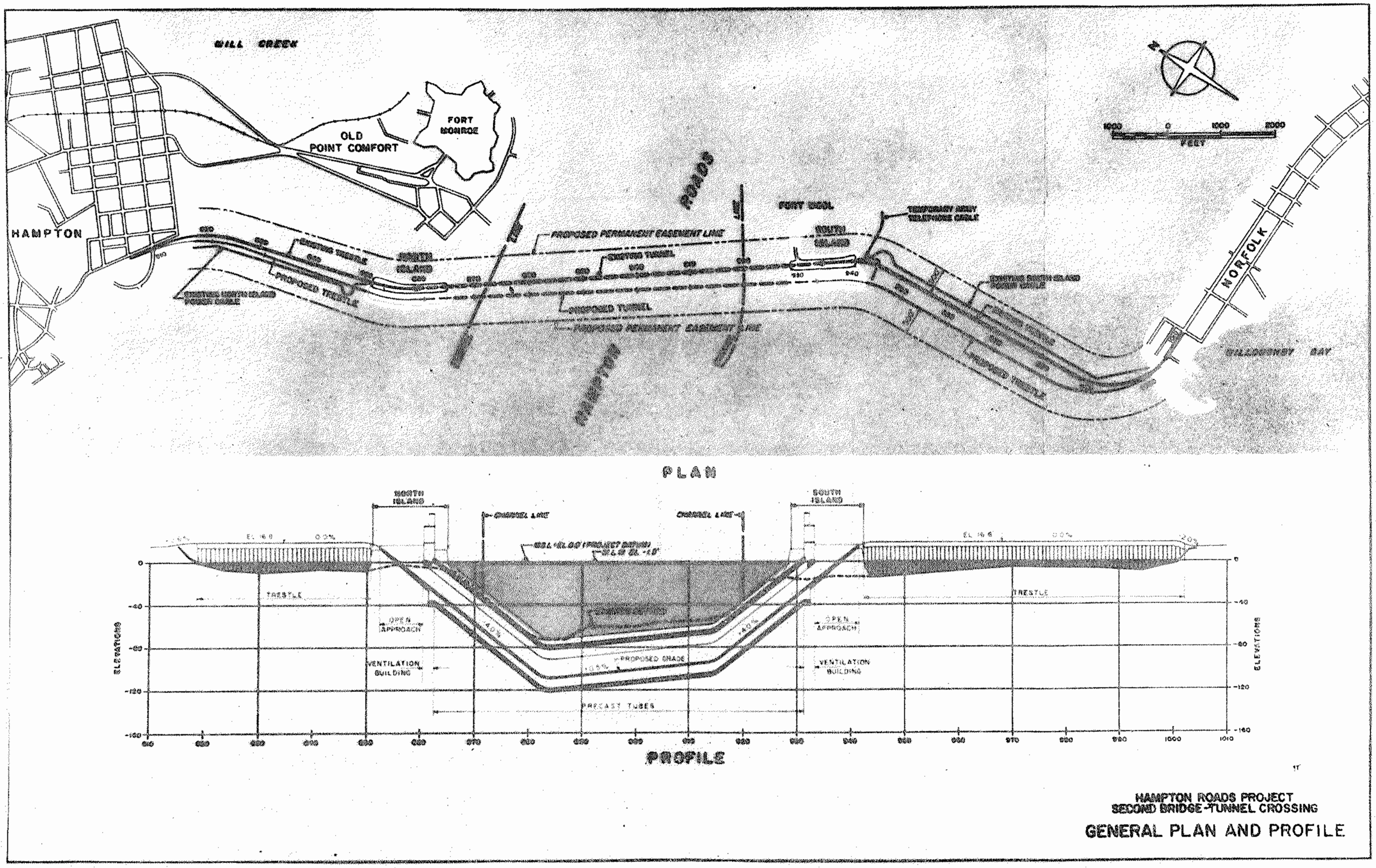

NORTH VENTILATION BUILDING
SOUTH VENTILATION BUILDING
CONSTRUCTION HATCH
CONSTRUCTION HATCH
UNDERWATER GRAVEL BERMS & BLANKET
PERMANENT FILL LINE
M.S.L. EL.0.0
EXISTING BAY BOTTOM
GRADE
+4% GRADE
+0.5% GRADE
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21
1
2
3
S E Q U E N C E
TUBE PLACEMENT

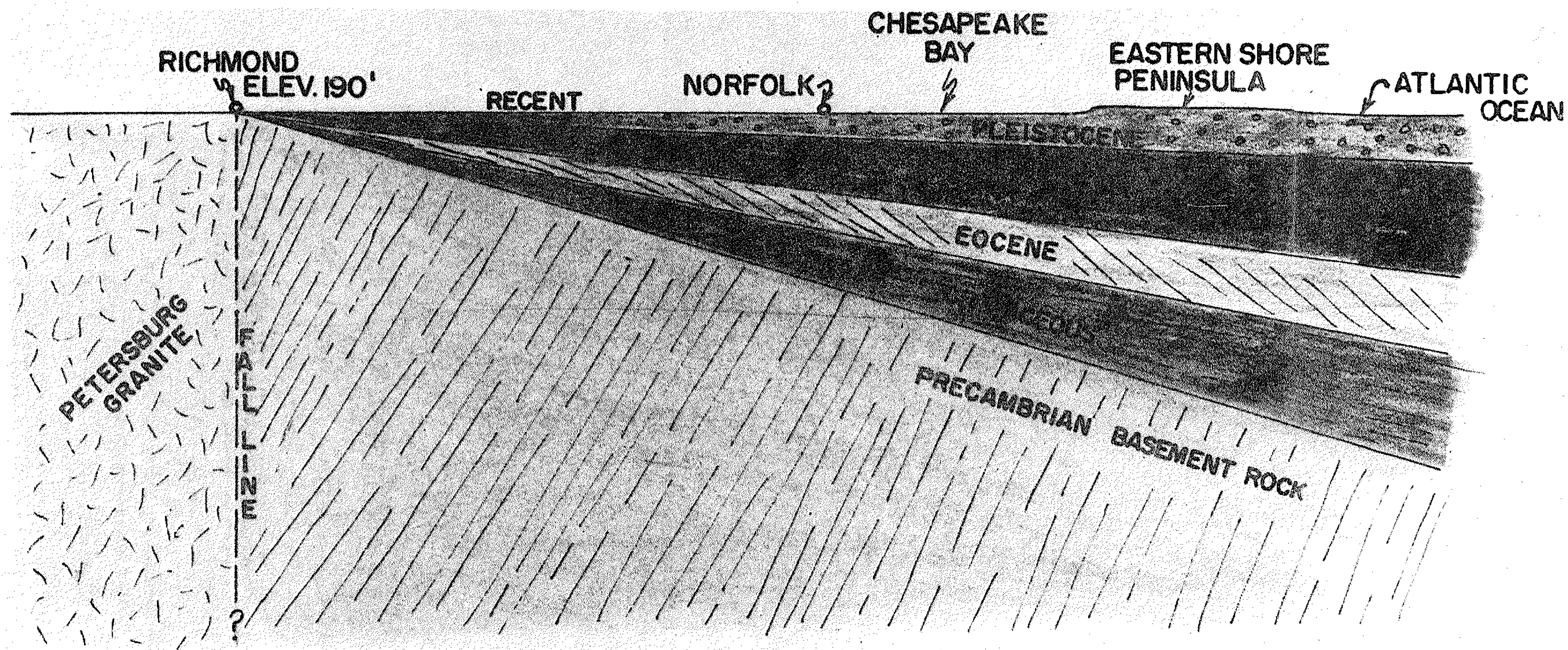
FIGURE 2



MILL CREEK
OLD POINT COMFORT
FORT MONROE
HAMPTON
ROADS
FORT WOOL
NORFOLK
WILLOUGHBY BAY
FEET
PROPOSED PERMANENT EASEMENT LINE
EXISTING TUNNEL
PROPOSED TUNNEL
EXISTING TRESTLE
PROPOSED TRESTLE
NORTH ISLAND
SOUTH ISLAND
PLAN
CHANNEL LINE
TRESTLE
OPEN APPROACH
VENTILATION BUILDING
PROPOSED GRADE
PRECAST TUBES
ELEVATIONS
PROFILE
HAMPTON ROADS PROJECT
SECOND BRIDGE-TUNNEL CROSSING
GENERAL PLAN AND PROFILE

FIGURE 3

FIGURE 4



GEOLOGICAL CROSS SECTION
RICHMOND TO NORFOLK

FIGURE 5

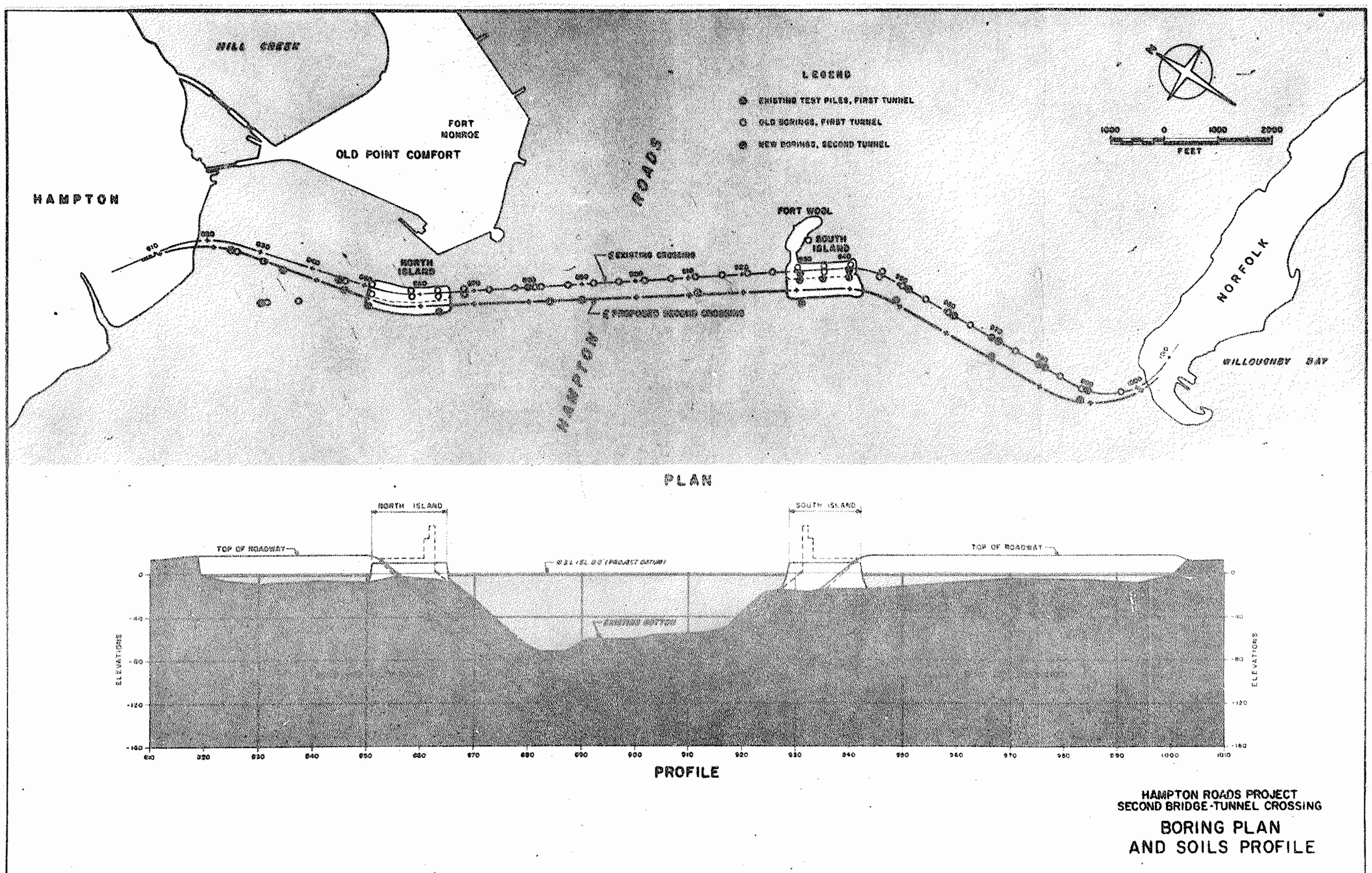
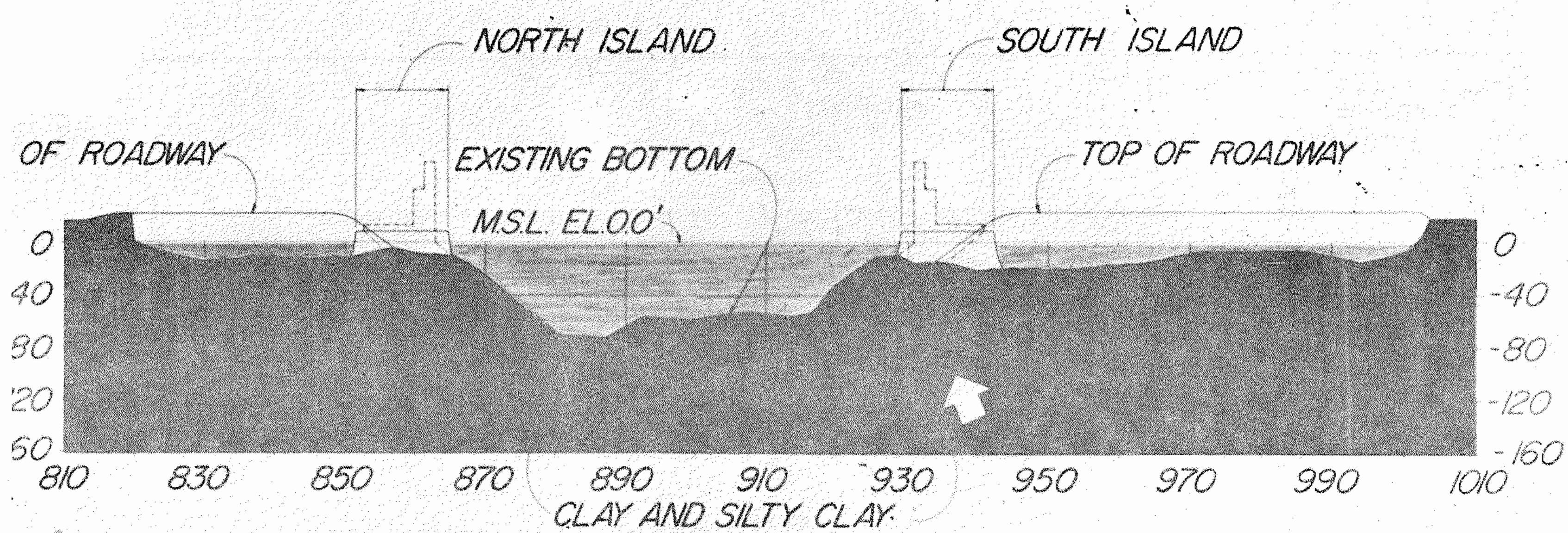

HILL CREEK
FORT MONROE
OLD POINT COMFORT
HAMPTON
ROADS
LEGEND
EXISTING TEST PILES, FIRST TUNNEL
OLD BORINGS, FIRST TUNNEL
NEW BORINGS, SECOND TUNNEL
1000 0 1000 2000
FEET
FORT WOOL
SOUTH ISLAND
NORTH ISLAND
EXISTING CROSSING
PROPOSED SECOND CROSSING
HAMPTON
NORFOLK
WILLOUGHBY BAY
PLAN
NORTH ISLAND
SOUTH ISLAND
TOP OF ROADWAY
TOP OF ROADWAY
EXISTING BOTTOM
ELEVATIONS
ELEVATIONS
PROFILE
HAMPTON ROADS PROJECT
SECOND BRIDGE-TUNNEL CROSSING
BORING PLAN
AND SOILS PROFILE

FIGURE 6



SOILS PROFILE

FIGURE 7

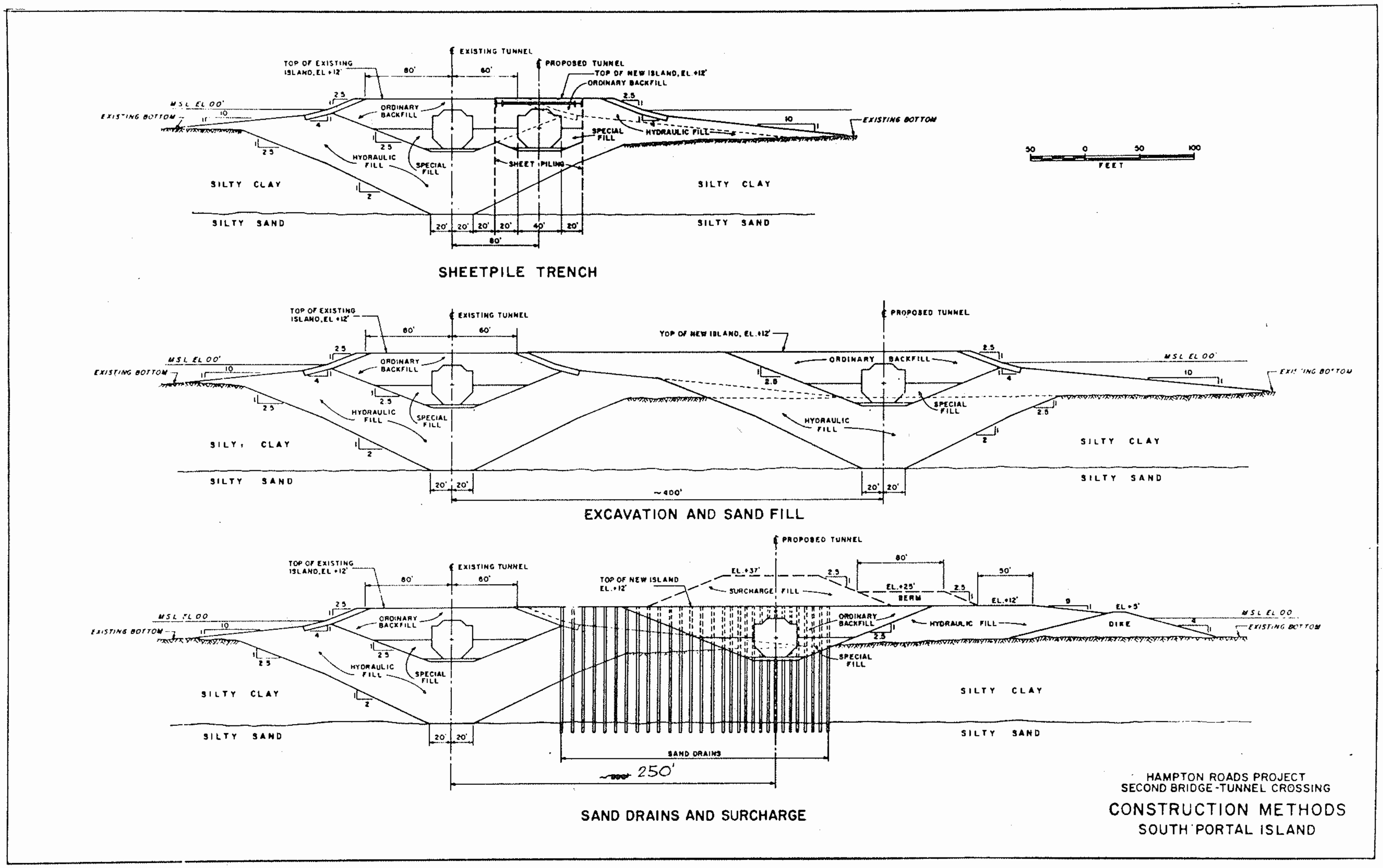
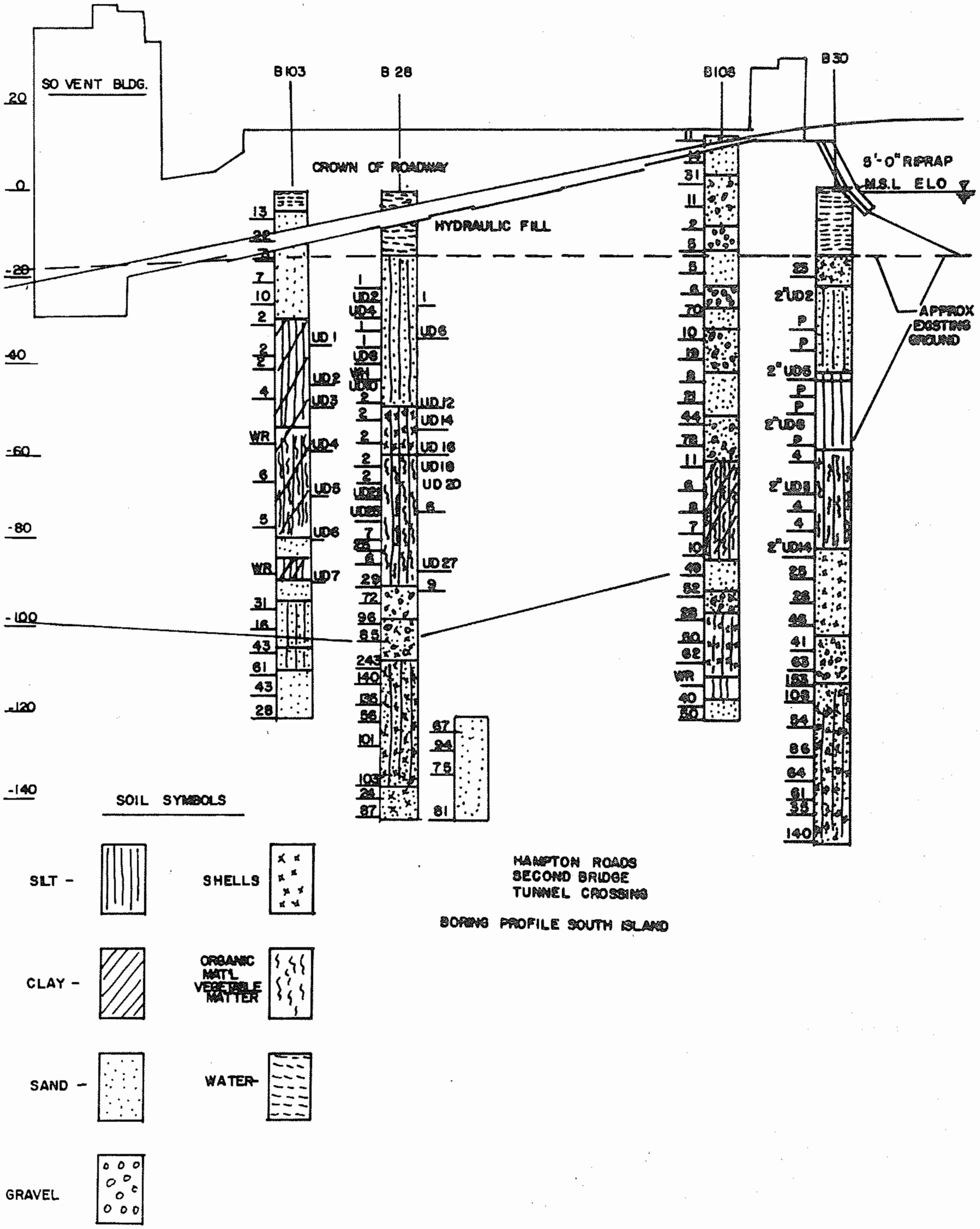

SHEETPILE TRENCH
EXCAVATION AND SAND FILL
SAND DRAINS AND SURCHARGE
EXISTING TUNNEL
PROPOSED TUNNEL
TOP OF EXISTING ISLAND, EL +12'
TOP OF NEW ISLAND, EL +12'
ORDINARY BACKFILL
HYDRAULIC FILL
SPECIAL FILL
SHEET PILING
SURCHARGE FILL
BERM
DIKE
SAND DRAINS
MSL EL 00'
EXISTING BOTTOM
SILTY CLAY
SILTY SAND
EL +37'
EL +25'
EL +5'
~400'
250'
FEET
HAMPTON ROADS PROJECT
SECOND BRIDGE-TUNNEL CROSSING
CONSTRUCTION METHODS
SOUTH PORTAL ISLAND

FIGURE 8



SO VENT BLDG.
B103
B 28
B108
B30
CROWN OF ROADWAY
HYDRAULIC FILL
8'-0" RIPRAP
M.S.L. EL.0
APPROX EXISTING GROUND
HAMPTON ROADS
SECOND BRIDGE
TUNNEL CROSSING
BORING PROFILE SOUTH ISLAND
SOIL SYMBOLS
SILT -
CLAY -
SAND -
GRAVEL
SHELLS
ORGANIC MAT'L VEGETABLE MATTER
WATER-

FIGURE 9

FIGURE 10

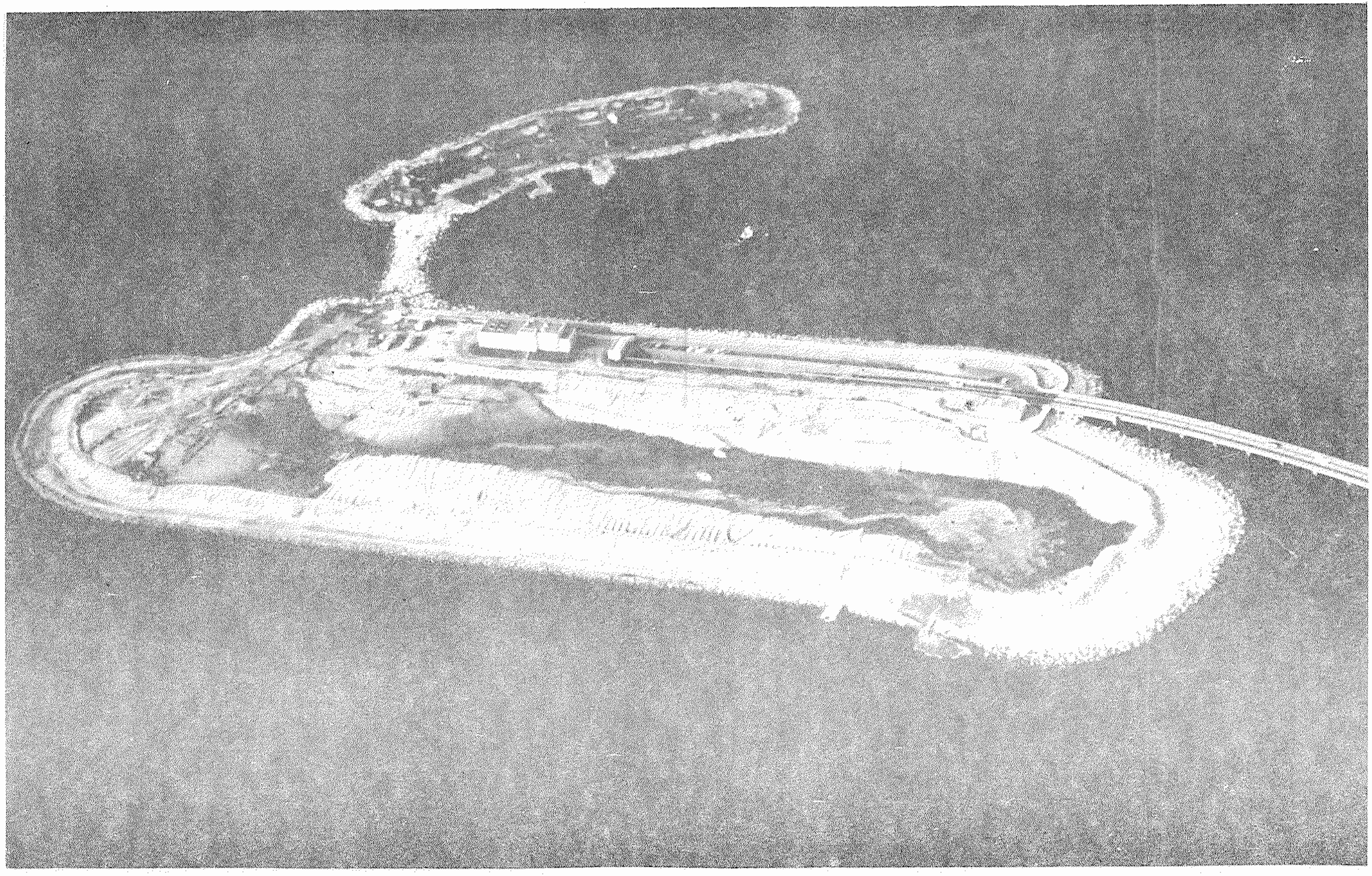
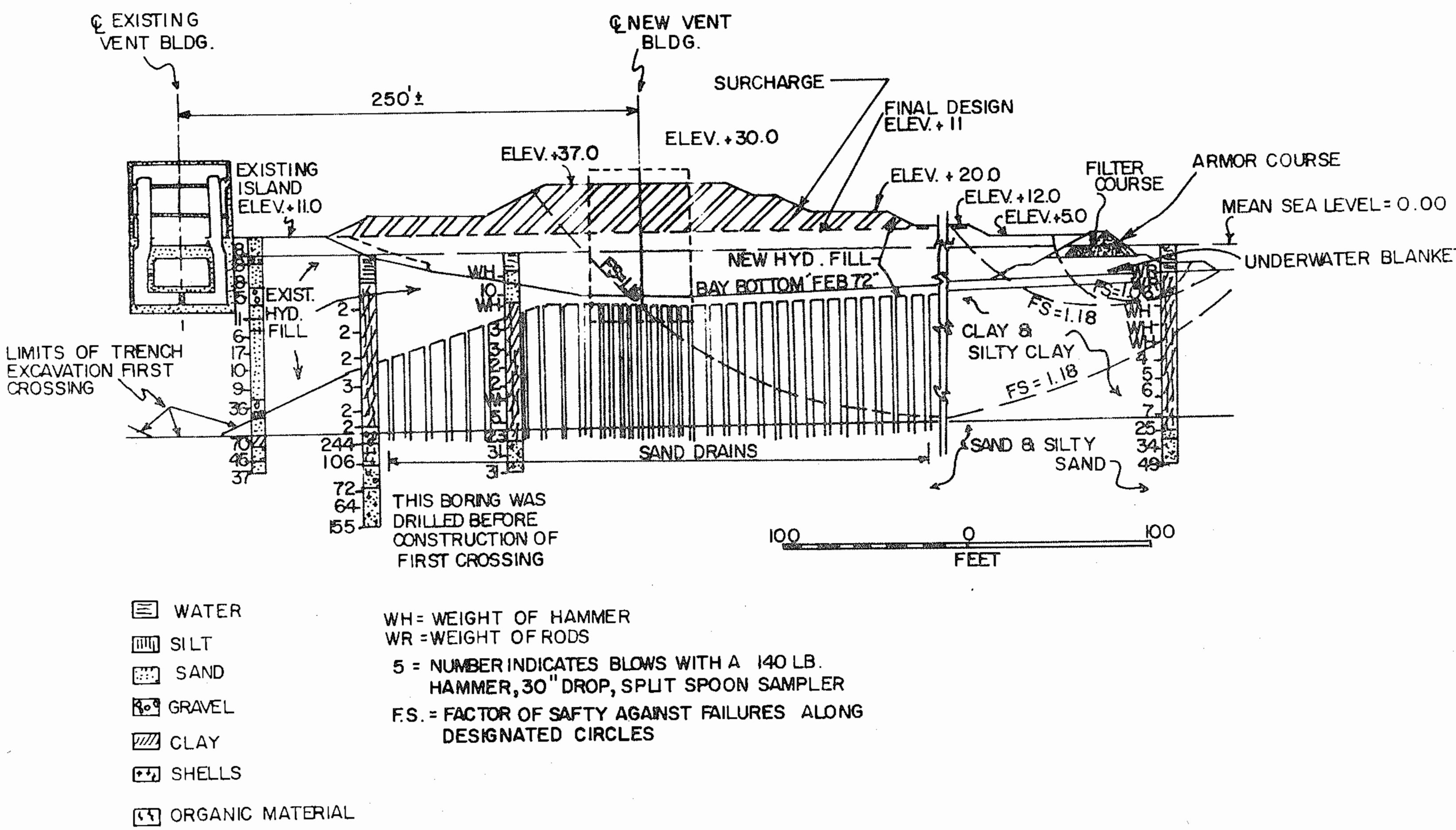
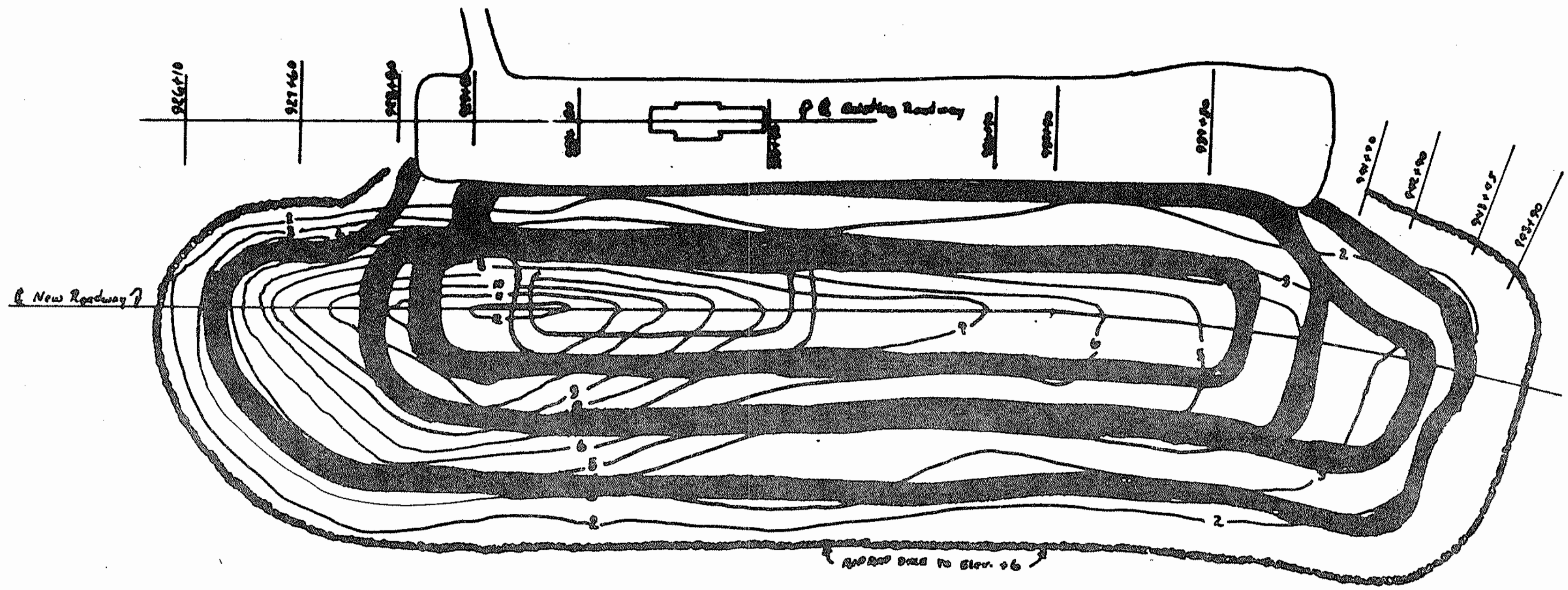


FIGURE 11



CROSS SECTION (A-A) OF SOUTH ISLAND

FIGURE 12

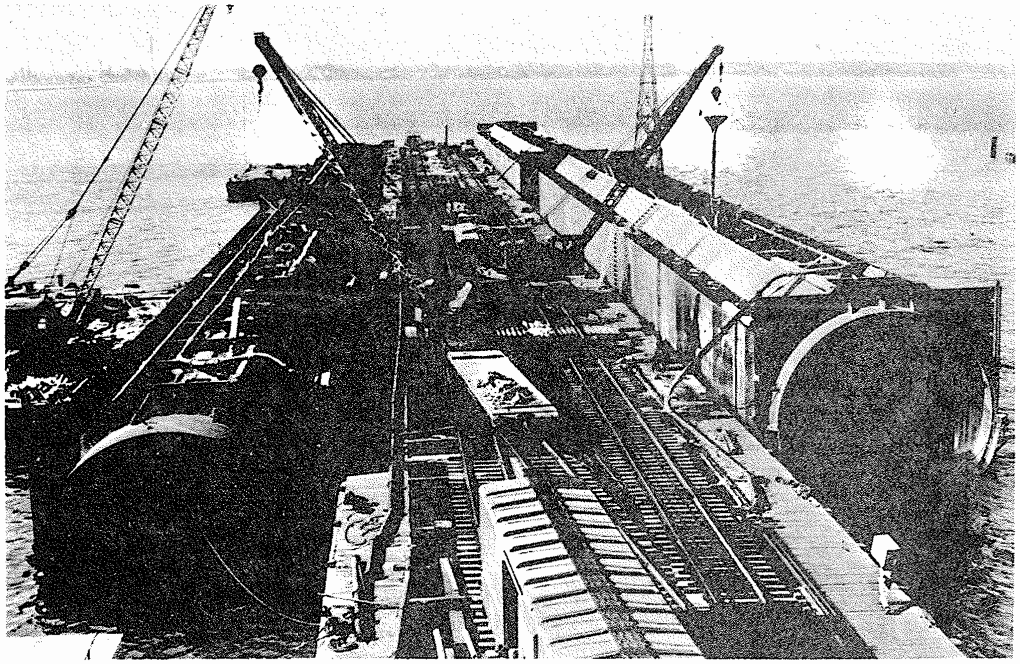


HAMPTON ROADS TUNNEL NO. 2 - SOUTH ISLAND
TOTAL SETTLEMENT AS OF DECEMBER 1, 1971

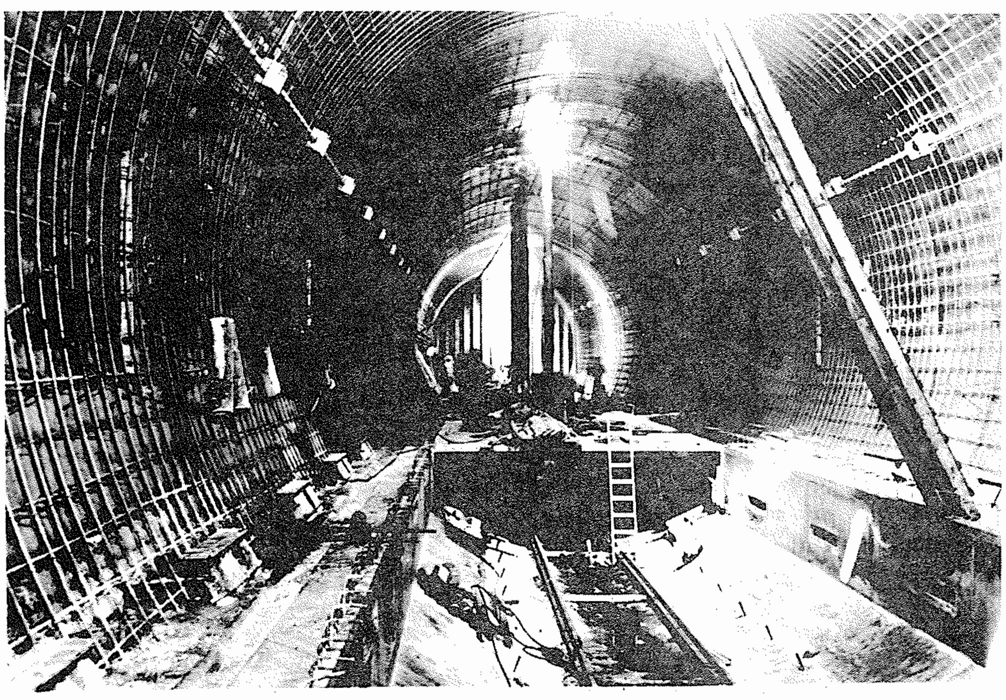
FIGURE 13


HAMPTON ROADS
COMMONWEALTH OF
DEPARTMENT OF HI
HAMPTON ROADS
SECOND BRIDGE TUNNEL

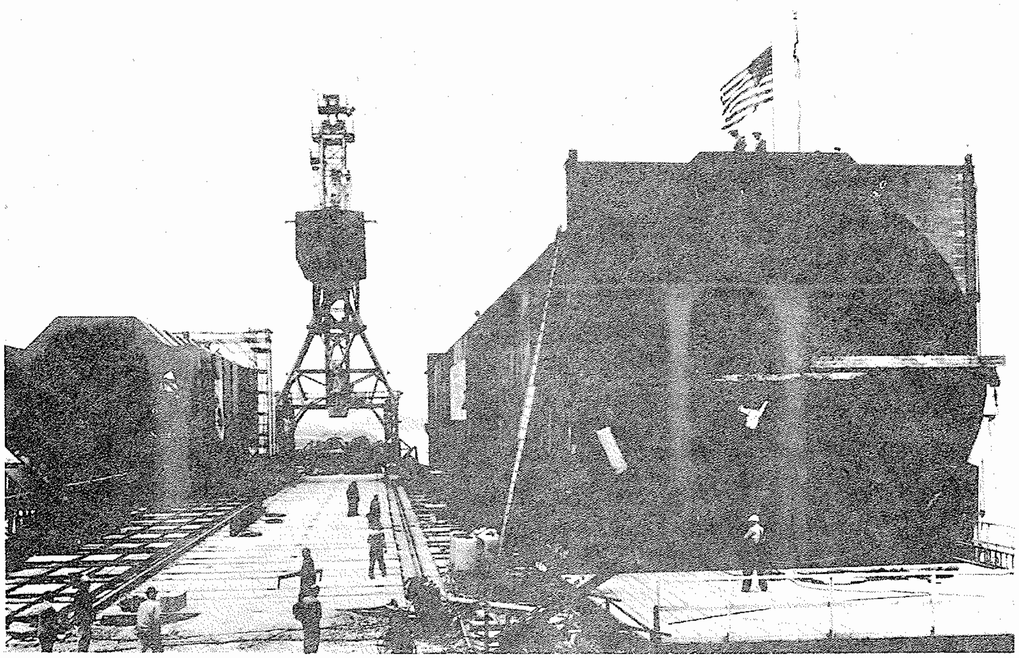
FIGURE 14



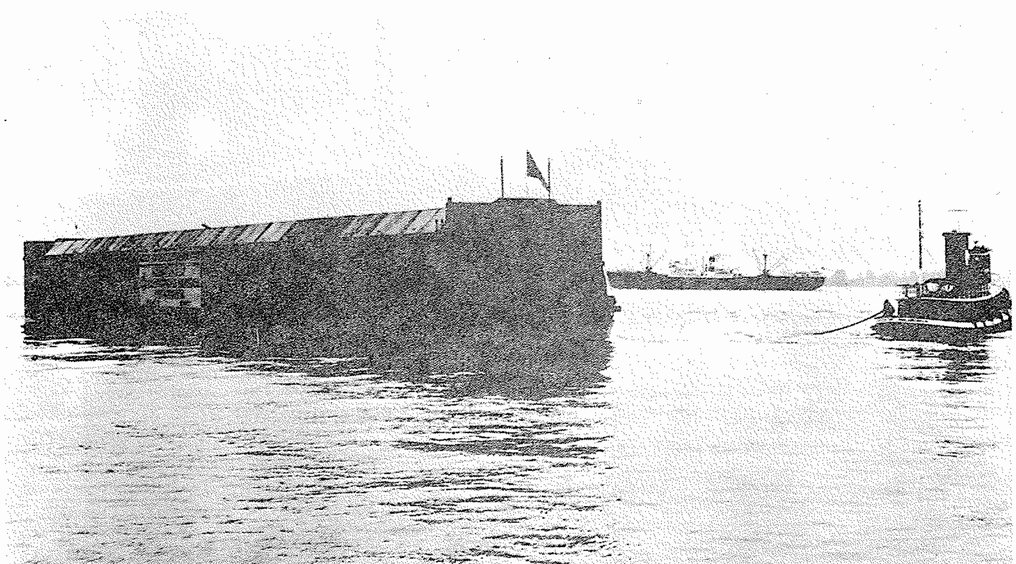
The "outfitting" basin was located at the old coal peirs in Norfolk harbor. The tubes, still afloat, were lined with an inner ring of concrete and fitted under a 23 foot roadway. This also included all utility and ventilation ducts. By the time all of the outfitting was done only 3 feet of the tube was above water.



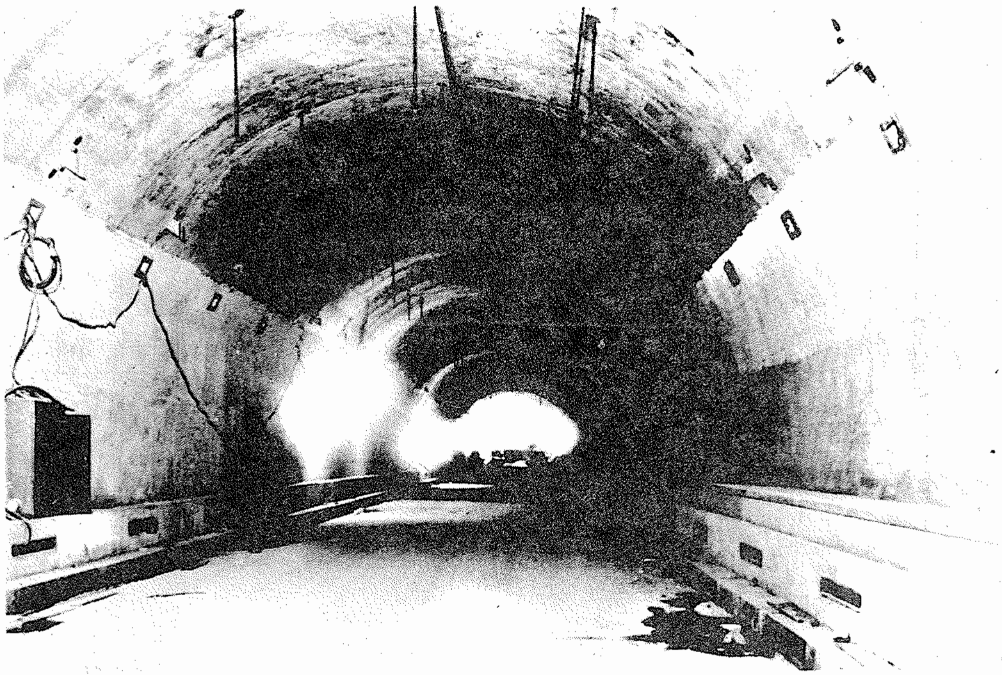
The interior of the tube of the outfitting basin shows forms used for placement of the two lane roadway slab atop supporting haunches. Concrete was placed through a snorkel (center) via openings spaced along the top. Slats at lower right were for ventilation system's air ducts. When completely outfitted, a section weighted about 12,000 tons and worth about 2 million dollars.



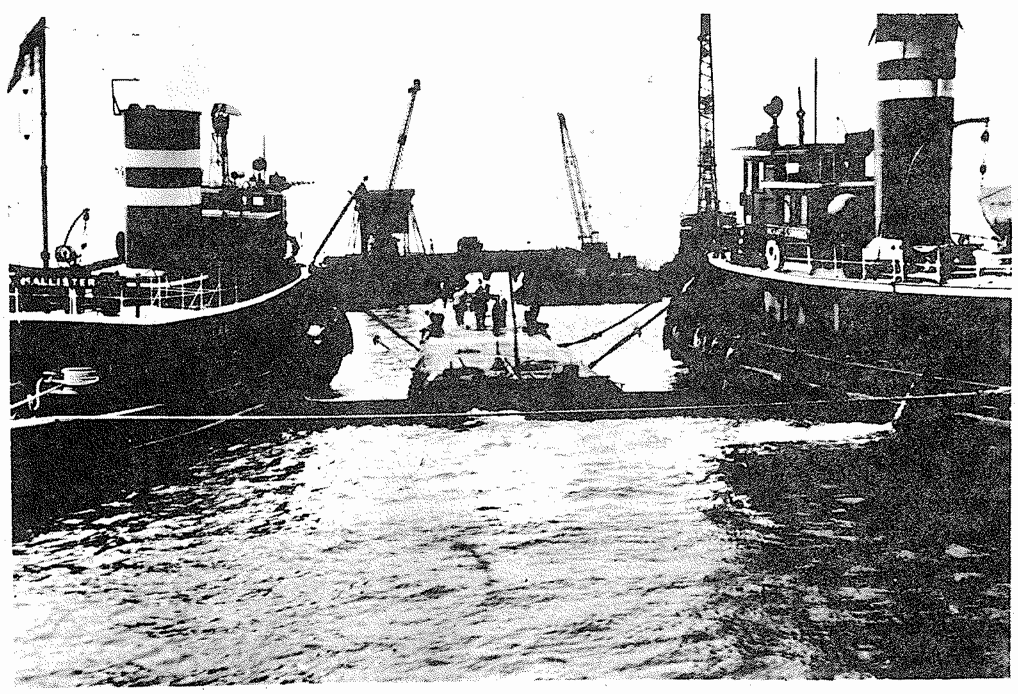
The core of the tunnel consists of 23 double-shell steel tube sections, each about 350 feet long. The octagonial shaped outer shell of the tube measures 37 feet in diameter while the circular inner shell measures 33 feet in diameter. The sections were fabricated at Port Deposit, Maryland on special ways. Tube at right of photo is ready to be launched.



Enroute to the tunnel site, the completed steel tube has been fitted with watertight bulkheads at boths ends and launched into the Susquahana River which flows into the Chesapeake Bay. Fabrication of each section received approximately 600 tons of structural steel. About 1,500 tons of concrete was placed in the "Keel" between the inner and outer shells to serve as ballast.



After being outfitted, the almost completely submerged tube is maneuvered into position and lowered into a gravel bedded trench dredged across the bottom of Hampton Roads. To sink a section, additional concrete was pumped into pockets between the inner and outer shells. Once the tube became heavy enough to reach negative buoyancy it was lowered into position by the "lay down barge".



Once in position on the bottom, the tubes were backed into position with hydraulic jacks and bolted and the remaining space between inner and outer shells filled with concrete. A solid ring of concrete was poured around each joint and as the steel bulkheads between sections were progessively cut through, the joints were welded from the inside and faced with an inner ring of concrete. Lying on the bottom in the trench the tube was covered with 5 feet of sand.

GEOTECHNICAL INVESTIGATION FOR THE DES PLAINES RIVER SYSTEM TUNNELS AND SHAFTS

By

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Building Construction Technology
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ACKNOWLEDGMENT

The Metropolitan Sanitary District of Greater Chicago has made available to the author reports and data from various consultants. Information from these reports form a partial basis for this paper.

ABSTRACT

As part of the Tunnel and Reservoir Plan for Greater Chicago an integrated geotechnical investigation was performed for a tunnel and drop shaft system located along the Des Plaines River in the Western Suburbs of Chicago, Illinois.

The proposed facility is composed of 17.8 miles of 12 to 33 feet diameter tunnels sited in hard rock up to 250 feet below the surface.

The main tunnel will be located in Silurian dolomites of the Alexandrian and Niagaran Series.

To evaluate the physical properities of the Silurian rock mass a geophysical downhole logging program was used in conjunction with core drilling, laboratory testing and stratigraphic studies. The Density, Caliper, Gamma Ray Neutron and 3-D Velocity Logs provided accurate rapid, reproducible physical and stratigraphic data about the rock mass quality and discontinuities.

INTRODUCTION

Removal of storm water runoff and domestic and industrial wastes from 360 square miles of the Greater Chicago Area poses a tremendous challenge. Presently, a combined sewer system built up over the years receives both storm water and raw sewerage. During periods of low flow the system functions adequately, however, during peak flow times the system becomes inadequate and overflows into basements, streets, and Lake Michigan posing a health hazard and becomes the major source of pollution to the streams and waterways of the area.

To find a means of relieving these conditions, the Metropolitan Sanitary District of Greater Chicago considered many solutions and settled on a composite program known as the Tunnel and Reservoir Plan (TARP). The Tunnel and Reservoir Plan will (1) separate storm and sanitary sewers, (2) provide temporary storage for the polluted water, and (3) through a system of tunnels and shafts convey the polluted water to treatment plants.

In general the Tunnel and Reservoir Plan is to capture all runoff which will occur from the combined sewers in a period of 25 years, except for peak discharge periods which will allow for spillage once in 5 years. Specifically, the Des Plaines system is designed to capture all spills from the outfalls presently discharging into the Des Plaines River from the combined sewers serving the project area.

The tunnel and reservoir facility will be set vertically 100 to 250 feet below the surface and consists of approximately 17.8 miles of mainline tunnel varying in size from 10 to 33 feet in diameter which will accommodate flows up to 22,000 cfs. derived from 50 drop shafts ranging in size from 4 to 15 feet. The system contains about 8.1 miles of lateral tunnels varying in size from 10 to 13.5 feet in diameter.

With the exceptions of small portions, the tunnel alignment follows the Des Plaines River and underlies property owned by the Cook County Forest Preserve District. The mainline tunnel begins in Lyons on the south and terminates in Des Plaines near Prairie Avenue on the north end. Three laterals feeding the mainline tunnel were investigated, these are along Roosevelt Road, Salt Creek in Brookfield and the Des Plaines River in Riverside. Only the mainline will be discussed in this report.

GEOTECHNICAL INVESTIGATION

The geotechnical program was conducted to secure data to evaluate the optimal horizontal and vertical alignment by establishing data pertaining to the following:

1. Relationship of faults and/or fractures to the design and construction of the facility,
2. Rock stratigraphy,
3. Physical properities of the rock mass in the proposed tunnel zone,
4. Grouting and rock bolting requirements,
5. Geohydrology and,
6. Soil parameters at the drop shafts.

To secure the desired data the geotechnical program was composed of the following elements:

1. Vertical core drilling and sampling of both the overburden and bedrock.
2. Angle core drilling at selected sites for fault, shear zone and fracture identification.
3. Geophysical downhole logging to secure a Gamma-Neutron Log, Caliper-Density Survey and a 3-Dimensional Velocity Log to establish the insitu physical characteristics of the rock mass.
4. Water pressure testing of selected zones in each boring to determine rates of infiltration/exfiltration and to anticipate grouting requirements.
5. Selective laboratory testing of rock samples to establish rock classification, shear strength, tensile strength, unit weight, moisture content, and other pertinent physical parameters.

GEOLOGY

The Des Plaines River System of Tunnels and Shafts is set in two physiographic divisions. The Chicago Lake plain which is characterized by exceptionally flat topography with the borders marked by sharp steps about 20 feet high. The lake stages are identifiable at the south end of the project area, while at the north end, the boundaries are less distinct where the Des Plaines River has cut through the surrounding Wisconsinan moraines. The most northern portion of the project terminates in the Woodfordian substage which exhibits the typical physiographic features developed by continental glaciers including kames, peat filled lakes and sharp relief of 50 to 100 feet.

Geologic Setting

The geologic setting of the Chicago area is divided into two parts, the bedrock units of Paleozoic rocks and Cenozoic glacial deposits. Glacial deposits mantle more than 95 percent of the area and consist of unconsolidated till, silt, clay, sand and gravel. In the project area glacial drift varying from eight to 193 feet thick was spread during the Wisconsinan Glacial Epoch largely, as part of the Lake Michigan Lobe. They are readily differentiated from the older consolidated bedrock formations of dolomite, shaley dolomite and shale. Within the project area exists a stratigraphic sequence of 4000 feet of sedimentary rocks ranging in age from the Cambrian to Silurian.

The Paleozoic bedrock formations gently dip down the east flank of the Kankakee Arch, which connects the Wisconsin Arch and the Cincinnati Arch and separates two broad depressions: the Illinois Basin to the southwest and the Michigan Basin to the northeast. Stream and glacial action has truncated the arch and produced a surface that has the broad aspects of a plain. As a result, the older rocks are exposed along the axis and the younger rocks are exposed in the bordering basins. Other than glacial tills, the older Paleozoic Rocks are found in quarries that in some instances have cut through the entire Silurian rock sequence.

In the Chicago area are a number of gentle east-west trending folds. Faults have been identified principally with the Sandwich Fault Zone near Joliet and in the Des Plaines Disturbance at the north end of the project.

Even though bedrock ranging from Cambrian to Silurian exists in the project area, only Ordovician, Silurian and Quaternary rocks were investigated and provided the data for this report.

A Geologic Profile is presented as Exhibit 3.

Stratigraphy

Ordovician System

Three Ordovician System rock groups were penetrated in the project area. The groups are Galena, Maquoketa and possibly Ancell. The Ancell Group, if present, is the St. Peter Sandstone, the Galena Group is composed of the Wise Lake-Dunleith Formation, and the Maquoketa Group contains the Scales Shale, Fort Atkinson Limestone, Brainard Shale, and Neda Formation. Most boreholes were terminated in the upper portion of the Maquoketa Group.

Silurian System, Alexandrian Series

This series consists of two formations through which a portion of the tunnel will be driven near the south terminus of the tunnel. Both the Edgewood and Kankakee Formations consist of light grey, fine to medium grained dolomite. The Edgewood contains chert and is argillaceous, while the Kankakee exhibits wavey bedding with the dolomite bands separated by thin seams of green shale.

The Edgewood Formation has a wide variation in thickness from nothing to over 81 feet. The variation resulted when shaley dolomite was deposited on the eroded surface of the Brainard Shale. The upper contact is nearly flat, so that the thicker portions occupy channels which were cut in the Brainard Shale. The Kankakee dolomite is persistent throughout the project area and varies between 38 to 60 feet thick.

The engineering properties of both formations are very similar as shown by the data summary on Table 1. For both formations, the compressive strength ranges between 13,352 psi and 14,680 psi, with a tensile strength between 1202 psi and 1557 psi.

Silurian System, Niagaran Series

The Niagaran Series contains five formations all composed of dolomite. The majority of the tunnel will be driven through these units.

Brandon Bridge Member

The Brandon Bridge Member is described as a red to pink, fine to medium grained, argillaceous dolomite with a greenish tint in various areas, separated by numerous red and green shale partings. This member is not present in all areas of Chicago, but has been identified in the Des Plaines area where it averages about 14 feet thick.

Unconfined compressive strengths ranged from 4620 psi to 20,990 psi, while a single tensile strength test yielded 1647 psi.

Markgraf Member

This formation is a widespread, bluish grey dolomite. Thickness measurements range from nine to 47 feet with an average of 32 feet.

Unconfined compressive strengths ranged from 4510 psi to 25,580 psi and a tensile strength of 1335 psi.

Romeo Member

This unit is a persistent, thin, very dense, light grey, fine grained dolomite and varies from seven to 19 feet thick.

Unconfined compressive strengths ranged from 5770 psi to 40,120 psi. Tensile strengths tests varied from 1940 psi to 3420 psi and averaged 1492 psi.

Racine Dolomite Formation

The Racine Dolomite is the rock unit most often encountered in the rock quarries in the Chicago area. The Racine Dolomite is light grey with interbedded chert and shale. A thickness of twenty feet was measured near O'Hare Airport, while 250 feet was logged at Roosevelt Road. This unit was subjected to erosion prior to deposition of the overlying glacial tills, which accounts for the variable thickness and extremely uneven top surface. Throughout the Chicago area, reef facies (i.e., core, flank and interreef) have been identified in the Racine Dolomite and are exhibited in many road cuts and quarries.

Unconfined compressive strength values of 1700 psi to 41,970 psi were measured while the tensile strength averaged 1144 psi.

Quaternary System

The Quaternary System consists of all rock younger than the Tertiary, including those accumulating at the present time. As the system contains only one series, the Pleistocene, the terms Quaternary and Pleistocene refer to the same rocks in the project area.

The Pleistocene series included all unconsolidated rock formations in the project area that overlie the Paleozoic bedrock. The unconsolidated rock deposits are related to the repeated advances and retreats of the glaciers that covered the area. Also included are post glacial deposits made by streams, rivers, and sediments in lakes and ponds.

The last glacial epoch, Wisconsinan consisted of several advances and retreats and is responsible for the development of the modern drainage system and topography. In the project area the unconsolidated overburden is all part of the Wedron Formation.

Wedron Formation

The Wedron Formation is largely till but also contains several outwash layers of gravel, sand and silt.

The Wedron Formation in the project area is divided into the Malden Till, composed of clay, sand and gravel, Yorkville Till, a silty clay and Wadsworth Till also a silty clay.

Historical Geology

The Ordovician Maquoketa Shales were deposited on the existing Galena dolomite in a shallow sea that covered much of the interior portion of the continent. At the close of Ordovician time the sea regressed exposing the shale to weathering in a humid climate, developing the Neda Formation. During the hiatus between the ending of Ordovician deposition and the beginning of Silurian deposition erosion developed channels and a small amount of uplift occurred creating a high in the area now occupied by O'Hare Airport.

When the seas returned during Silurian time the Edgewood Formation was deposited in topographic low areas. On the south end of the project 81 feet of Edgewood was laid down while on the north end or the topographic high portion of the alignment no Edgewood was deposited.

Beginning with the Kankakee Formation and continuing through the deposition of the Racine Group a normal sequence of warm shallow sea deposition is recorded. A minor break in deposition (or unconformity) exists in the dolomites where a series of thin shales (red and green) are found between the Kankakee and Joliet Formations.

No evidence of other Paleozoic or Mesozoic rocks was noted in the project area; however, Willman, Frye and others have stated these rocks were deposited in the Chicago area.

At the end of Cretaceous time, world wide tectonic disturbances which built most existing mountain chains caused faulting up to 220 feet in the Des Plaines Disturbance Area bringing the Galena Dolomite and Brainard Shale into contact and again up-lifting the north end of the project area. Evidence for this is suggested by the thinning of the Racine from South to North and the exposure of Ordovician rocks (Galena and Brainard) within the Des Plaines Disturbance Area.

Erosion removed the Silurian and other Paleozoic rocks in Des Plaines Disturbance Area during Tertiary and early Quaternary time. During or just prior to the early Wisconsinan Glacial epoch, deep erosional channels were cut into the Brainard Shale, up to 200 feet deep, by the ancestral Des Plaines River.

Examination of cores and Gamma-Neutron Logs established the existence of three distinct and perhaps four glacial advances and retreats across the alignment area. These took place as part of the Wisconsinan glacial period.

STRUCTURAL GEOLOGY

The alignment area contains, for the Chicago Area, a variety of structural features including minor folding, faulting and erosion channels.

The project area exhibits gently folded beds with regional dips that are less than five degrees even in the presence of the Kankakee Arch. No major folds related to this feature were identified along the Des Plaines River.

Joints or fractures were found in varying numbers in the Silurian rocks. The joints or fractures are vertical or nearly vertical and are principally in the Racine Formation and at the Romeo-Markgraf contact. Most contain a green clay filling that has been identified as Devonian Shale elsewhere, however, some joints or fractures are filled with black clay or are open. The open joints took grout when the borings were backfilled and therefore, can be expected to contain water.

In the area outside the Des Plaines Disturbance only one fault was interpreted. This fault is located south of Roosevelt Road and has an apparent displacement of 45 feet. Data from borings in the area indicate a displacement of 45 feet vertically in 500 feet horizontally. An alternate explanation for this phenomenon would be a displacement on the flank of a reef which would appear as monocline or as a sheared and fractured zone when the growing reef settled into the unconsolidated underlying rocks.

In the Des Plaines Disturbance at least one fault was identified. Borings on a short lateral indicated the Galena dolomite was displaced 220 feet and into contact with the Brainard Shale. On the mainline, a repeat section of Maquoketa Shale was drilled with an indicated displacement of 220 feet. Tentatively, these two displacements are being called one fault.

The highly eroded bedrock surface is covered with glacial tills throughout the project area. The bedrock varies from eight feet below the surface to 193 feet in the Des Plaines Disturbance Area. The difference in bedrock relief is attributed to faulting in the Des Plaines Disturbance which fractured the rock allowing easy erosional removal. The remaining surface is the result of differential weathering between hard reef core dolomites and more porous, shaley and less erosion resistance dolomite and to a lesser extent fractures and faulting.

GEOPHYSICAL LOGGING

Geophysical logging produces an insitu record of the response from the rock strata to various measuring devices lowered into the borehole. Logging devices must be calibrated to produce uniform results from one borehole to another. Variations in borehole conditions caused by a change in diameter, presence or absence of fluid or background radiation will change the response of the device and cause the results to deviate from acceptable values. The geophysical downhole logging program was conducted by the Birdwell Division of Seismograph Service Corporation and included Gamma Ray, Neutron, Formation Density, 3-D Velocity and Caliper logs.

The Gamma Ray Log records the natural radioactivity of the rock mass under a variety of borehole conditions. For example, shales normally contain higher percentages of radioactive elements than other sedimentary rocks.

The Neutron Log records the hydrogen content of the rock mass in fluid-filled or dry, open or cased borings. The logging tool emits neutrons that collide elastically with heavy atoms without much energy loss. However, a collison with a hydrogen atom reduces the energy of the neutron and capture eventually takes place. Capture is accompanied by emissions of gamma rays which are recorded. A measure of hydrogen present in the formation indicates the amount of fluid present and is directly related to the porosity of the formation. For the study reported here, the Gamma Ray - Neutron log was used primarily as a stratigraphic correlation tool with the geologic field log.

The Caliper - Density Log is recorded on one log sheet and contains two parts: The Caliper Log on the left of the log is a mechanical measure of the borehole diameter. At the right is the Formation Density Log which records the intensity of the reflected gamma rays that are dependent on the electron density in the rock mass. The electron density is directly proportional to the bulk density of the rock mass.

The 3-D Velocity Log is an accoustic signal record. The velocity of the shear and pressure waves in a fluid-filled uncased borehole are recorded as a variable intensity display. The 3-D Velocity Log and the Density data are combined and used to compute Poisson's ratio, Young's modulus and provide a measure of in situ density and porosity.

Formulas and mathemetical computations have been presented by Birdwell invarious publications and need not be repeated here.

GEOPHYSICAL DATA

Examples of geophysical data and typical presentation are shown on Exhibits 5 and 6. The three individual log sheets have been adjusted to the same datum allowing correlation of individual log records. The Rock Properties sheet presents the geophysically computed bulk density, Young's modulus, porosity, density and Poisson's ratio. These data are computed from the 3-D Velocity Log and the log can be recorded only when the borehole is full of fluid.

Exhibit 5, presents the geophysical data taken for boring D1-AS-104. All four log sheets are shown adjusted to the same datum, in this case the ground surface. The caliper log indicates the casing was set at a depth of 38 feet. The Neutron Log and the Density Log, marked with an "A", indicate a layer of clay till about 15 feet thick, a sand approximately 8 feet thick and a dense 12 foot layer of sandy gravel probably containing boulders.

Examination of the 3-D Velocity Log at "B" shows a 6 foot fracture and broken zone. This zone and the gravel in the overburden combine to form one of the most utilized aquifers in the Chicago area.

The Racine Formation at "C" contains many layers of deleterious chert. The Density, Neutron and 3-D Velocity Logs as well as the Rock Properties Log indicate this is a porous, low density poor rock zone which will cause construction problems.

Just below the Romeo, at point "D", near the top of the Markgraf Formation, a discontinuity is suggested by the variation in the 3-D events record and a deviation to the left on the Neutron Log. Sometimes the rock cores contained vertical fractures, but not always.

Exhibit 6, boring D2-AS-109, has casing set to 64.0 feet or through the overburden and into the bedrock. The section labeled "A" is composed of three glacial till layers. At the top is a layer of clay or silty clay about 12 feet thick with a relatively low density, in the middle is a sand or silty zone 22 feet thick of a higher density than the clay layer, and at the bottom is a sandy gravel with boulders showing an increased density.

At "B" is a 6 foot sound layer of dolomite capping "C" which are cherty zones in the Racine Formation. Point "D" marks a fracture or a section where the chert caved into the boring. At "E" are the twin red shales marking the top of the Kankakee Formation.

RESULTS

Every other borehole was geophysically logged and when coupled with selective rock testing reduced the total time and monies spent on the Geotechnical Investigation. Originally the geophysical phase was expected to provide data to verify the stratigraphy and laboratory test results. But two additional benefits accrued from the Gamma Ray-Neutron and 3-D Velocity Logs as familiarity with the stratigraphy increased, fractures were identified regularly on the Gamma Ray-Neutron Log and zones of weathered, fractured and low density rock requiring grouting or special treatment showed as blank areas on the 3-D Velocity Log.

Table 1, presents a comparison of the physical properties determined by geophysical methods and laboratory testing of selected specimens. As general information, the unconfined compressive (Qu) and the tensile (STS) strengths are presented. Comparison of the two techniques is made by using Young's Modulus (E) and Poisson's Ratio.

The observed differences in the two elastic moduli in the Brainard Shale is due to weathering, fracturing and recementing in the tunnel zone.

The Edgewood, Kankakee and Markgraf units contain thin seams of shale and when tested in the laboratory failure takes place sooner than when the same sample zone is examined geophysically insitu. Under these conditions the geophysical data is probably more reliable than the laboratory data.

When the physical properties of sound, dense rock are examined by the two techniques, very little difference in Young's Modulus and Poission's Ratio is observed.

CONCLUSION

The successful design and construction of a tunnel facility is dependent upon the evaluation of the geologic framework. Methods of exploration must be selected to optimize data gathering and be readily interpreted. The conceptual geotechnical investigation for the Des Plaines River System of the Tunnel and Reservoir Plan consisted of core borings, laboratory testing, stratigraphic studies and an independent technique utilized to evaluate and in instances extend the available data.

A geotechnical investigation must establish accurate data on the average quality, extent of discontinuities, and physical properties of the rock mass. Using geophysical downhole logging devices provided a rapid independent source of information. The Gamma Ray-Neutron Log correlated with stratigraphy from cores, and the physical properties computed from the geophysical logs were, in most cases, more reliable than laboratory test data because a larger insitu rock sample was analyzed. In addition the 3-D Velocity Log provided information to establish fractured, low density, porous zones requiring grouting or special treatment.

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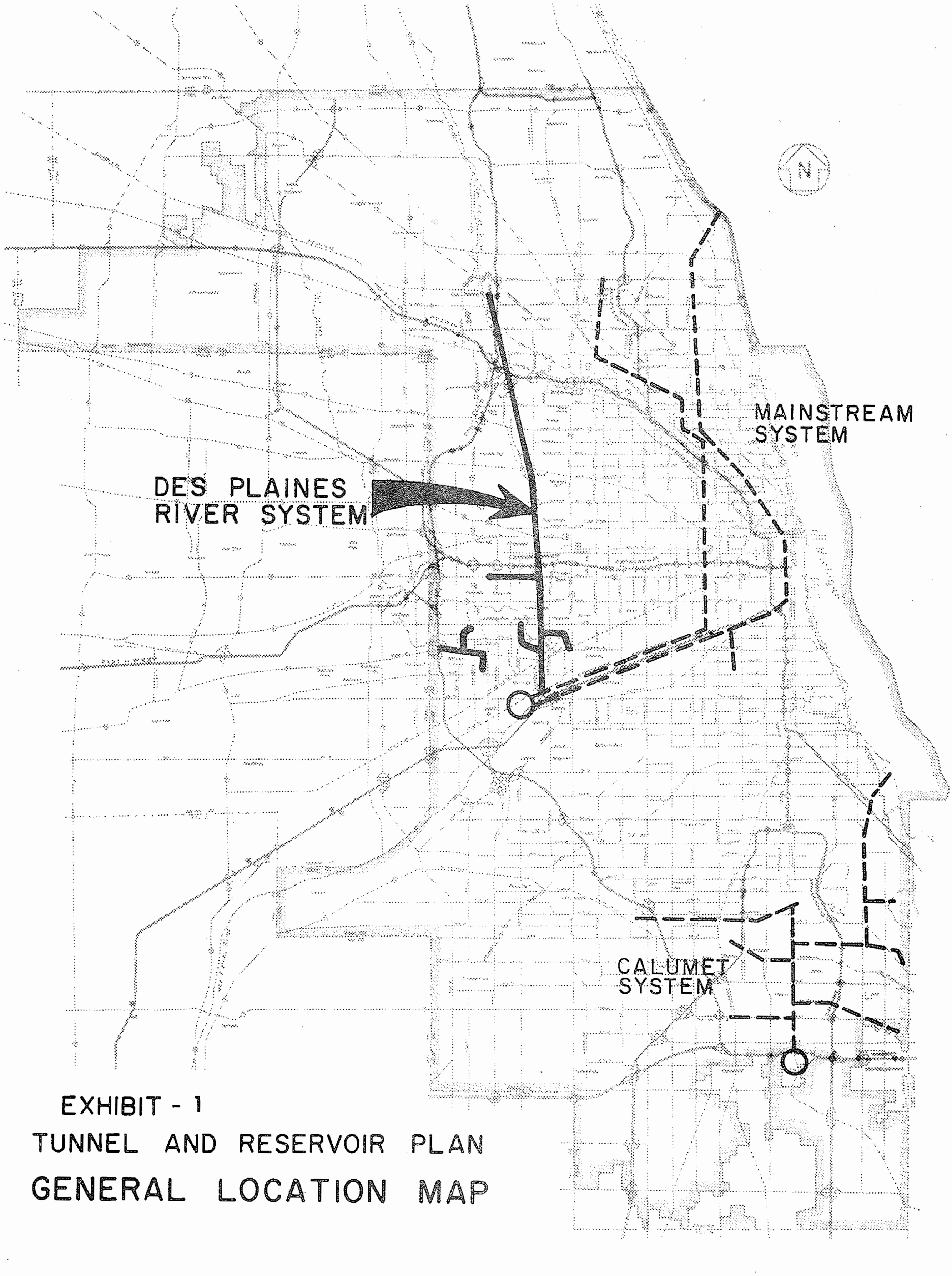
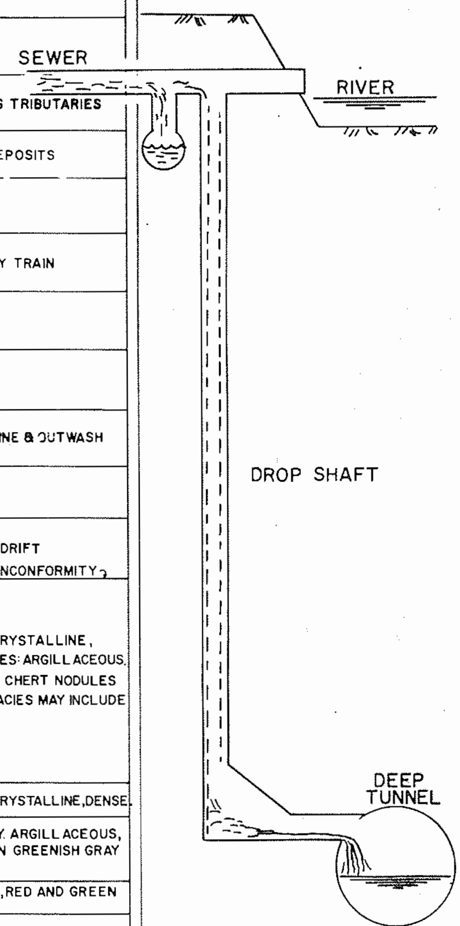


EXHIBIT - 1

TUNNEL AND RESERVOIR PLAN

GENERAL LOCATION MAP

SYSTEM	SERIES	STAGE	GROUP	FORMATION	MEMBER	GRAPHIC LOG	THICKNESS (FEET) MAX.	MIN.	AVG.	DESCRIPTION
QUATERNARY	PLEISTOCENE	HOLOCENE		MAN-MADE DEPOSITS			193	8	57	MISCELLANEOUS FILL
				CAHOKIA ALLUVIUM						ALLUVIUM IN DES PLAINES RIVER & ITS TRIBUTARIES
		WISCONSINAN		EQUALITY FM	DOLTON MEMBER (EQUALITY FM)					LAKE CHICAGO BEACH & SHORELINE DEPOSITS
					CARMI MEMBER (EQUALITY FM)					LAKE CHICAGO OFFSHORE DEPOSITS
				HENRY FM	MACKINAW MEMBER (HENRY FM)					OUTWASH-DES PLAINES RIVER VALLEY TRAIN
				WEDRON FORMATION	WADSWORTH TILL MEMBER					SILTY CLAY / CLAYEY SILT TILL
					YORKVILLE TILL MEMBER					CLAYEY SILT / SILTY CLAY TILL
										INTERGLACIAL / PROGLACIAL LACUSTRINE & OUTWASH
					MALDEN TILL MEMBER					SANDY SILT TILL
										INTERGLACIAL / PROGLACIAL LEMONT DRIFT UNCONFORMITY
SILURIAN	NIAGARAN			RACINE			250	0	113	DOLOMITE, LIGHT GRAY. REEF FACIES: CRYSTALLINE, MASSIVE AND POROUS. INTERREEF FACIES: ARGILLACEOUS, THIN BEDDED WITH SCATTERED POROUS CHERT NODULES AND THIN CHERT BEDS. REEF-FLANK FACIES MAY INCLUDE BEDS WITH DIPS TO 30°
				JOLIET	ROMEO		15	7	10	DOLOMITE, LIGHT GRAY, FINE GRAINED, CRYSTALLINE, DENSE
					MARKGRAF		47	9	32	DOLOMITE, LIGHT GRAY TO BLUISH GRAY. ARGILLACEOUS, FINE GRAINED, SOME CHERT, SOME THIN GREENISH GRAY SHALE PARTINGS.
					BRANDON BRIDGE		35	5	14	DOLOMITE, PINKISH GRAY, FINE GRAINED, RED AND GREEN SHALE PARTINGS.
	ALEXANDRIAN			KANKAKEE			60	38	49	DOLOMITE, GRAY TO PINK, MEDIUM GRAINED, SCATTERED CHERT NODULES, PROMINENT GREEN SHALE PARTINGS TO 0.5 INCH THICK.
				EDGEWOOD			81	0	37	DOLOMITE, CHERTY, LIGHT GRAY TO GRAY, FINE TO MEDIUM GRAINED. BECOMES DARKER IN COLOR AND MORE SILTY AND SHALY WITH DEPTH, ESPECIALLY WHERE THICK. UNCONFORMITY
ORDOVICIAN	CINCINNATIAN	RICHMONDIAN	MAQUOKETA	NEDA			24	0	8	SHALE, RED, HEMATITIC & OOLITIC.
				BRAINARD			135	10	89	SHALE, GREENISH GRAY, MEDIUM SOFT. INTERBEDS OF LIGHT GRAY DOLOMITE.
				FORT ATKINSON			23	10	15	DOLOMITE, GRAY, MEDIUM GRAINED. SOME INTERBEDS OF BROWN-BLACK SHALE.
				SCALES			84	33	58	SHALE, DARK GRAY TO BROWNISH BLACK, MEDIUM SOFT TO SOFT, WITH INTERBEDS OF GREEN, GRAY SHALES & DOLOMITE.
	CHAMPLAINIAN	TRENTONIAN	GALENA	WISE LAKE					198	DOLOMITE, GRAY, PURE, VUGGY.
				DUNLEITH						DOLOMITE, GRAY, ALTERNATING PURE & ARGILLACEOUS UNITS. CONTAINS CHERT.
		BLACK RIVERAN	ANCELL	GLENWOOD					197	SANDSTONE, WHITE, POORLY SORTED, INTERBEDS OF IMPURE DOLOMITE & SHALE, BIMODAL.
				ST. PETER						SANDSTONE, WHITE, FINE TO MEDIUM GRAINED, WELL SORTED, WELL ROUNDED, FROSTED, QUARTZ GRAINS.



STRATIGRAPHIC COLUMN AND TUNNEL SITING

EXHIBIT - 2

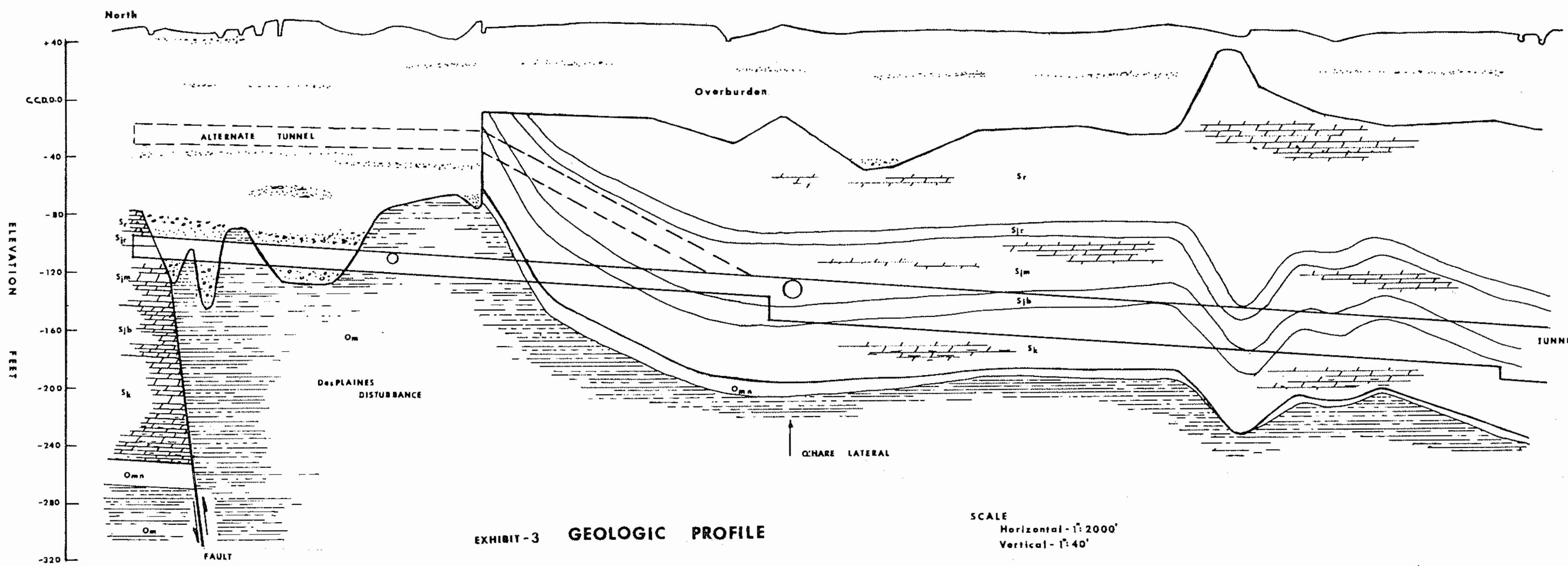
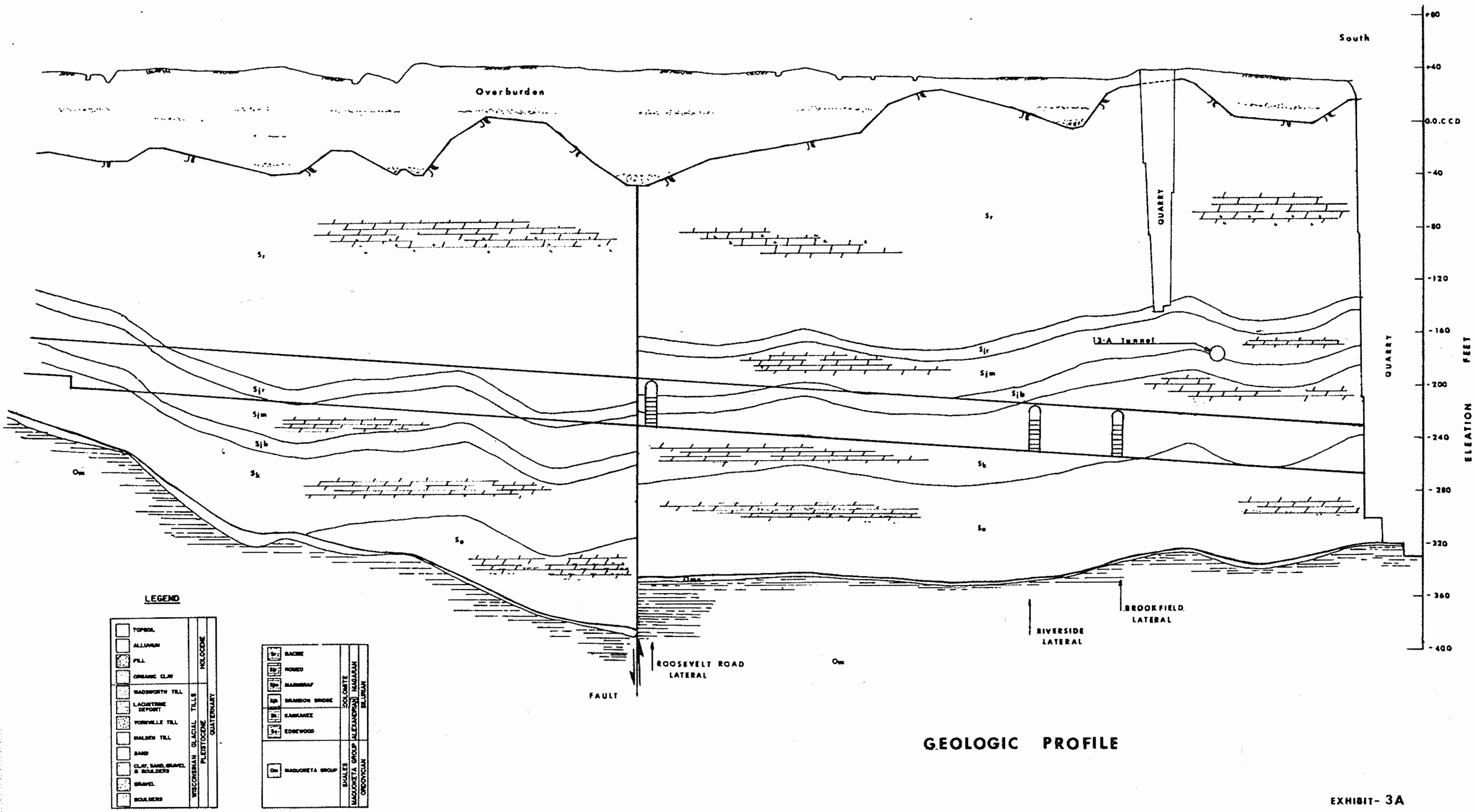


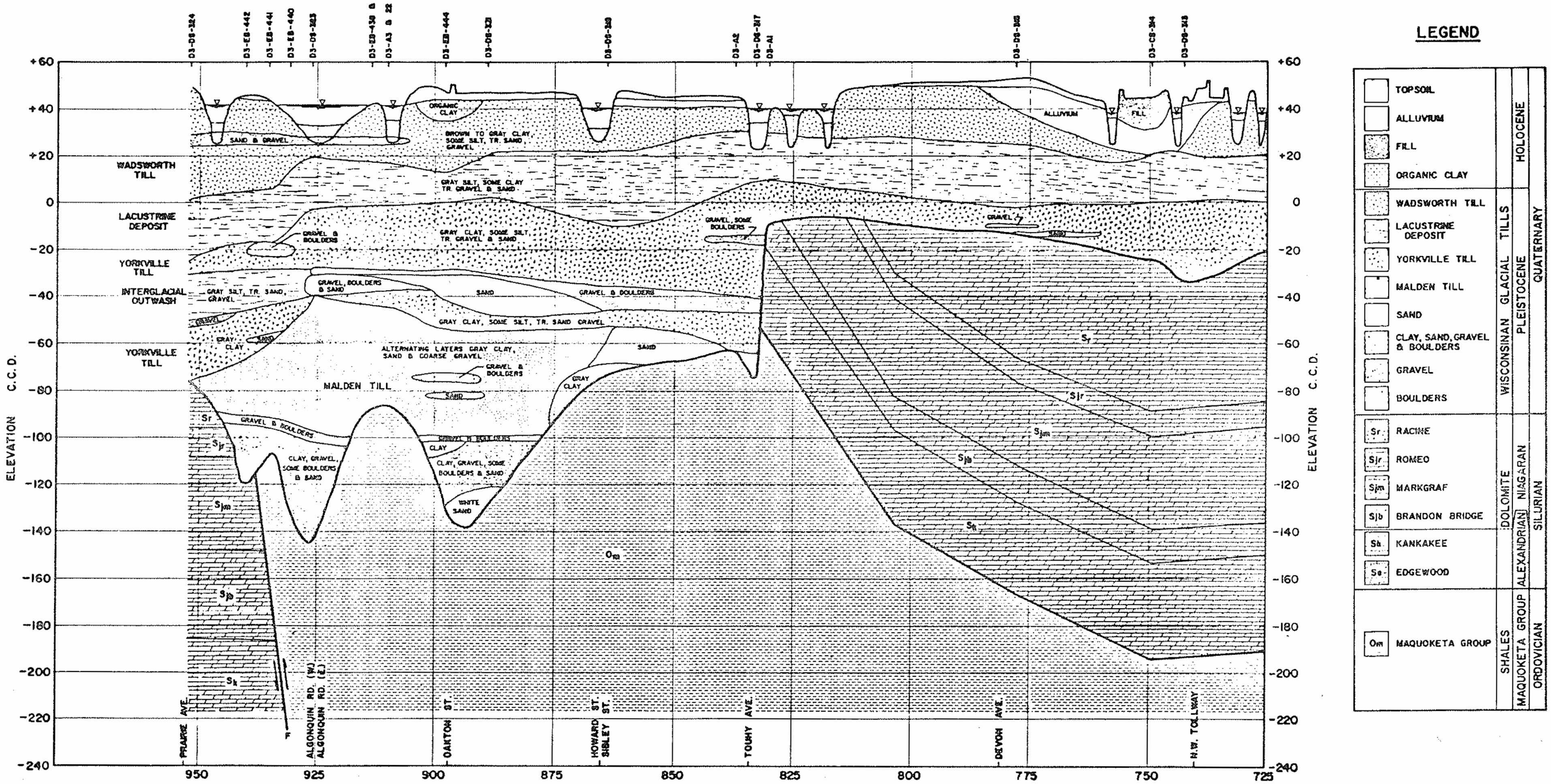
EXHIBIT - 3 GEOLOGIC PROFILE

SCALE
Horizontal - 1": 2000'
Vertical - 1": 40'



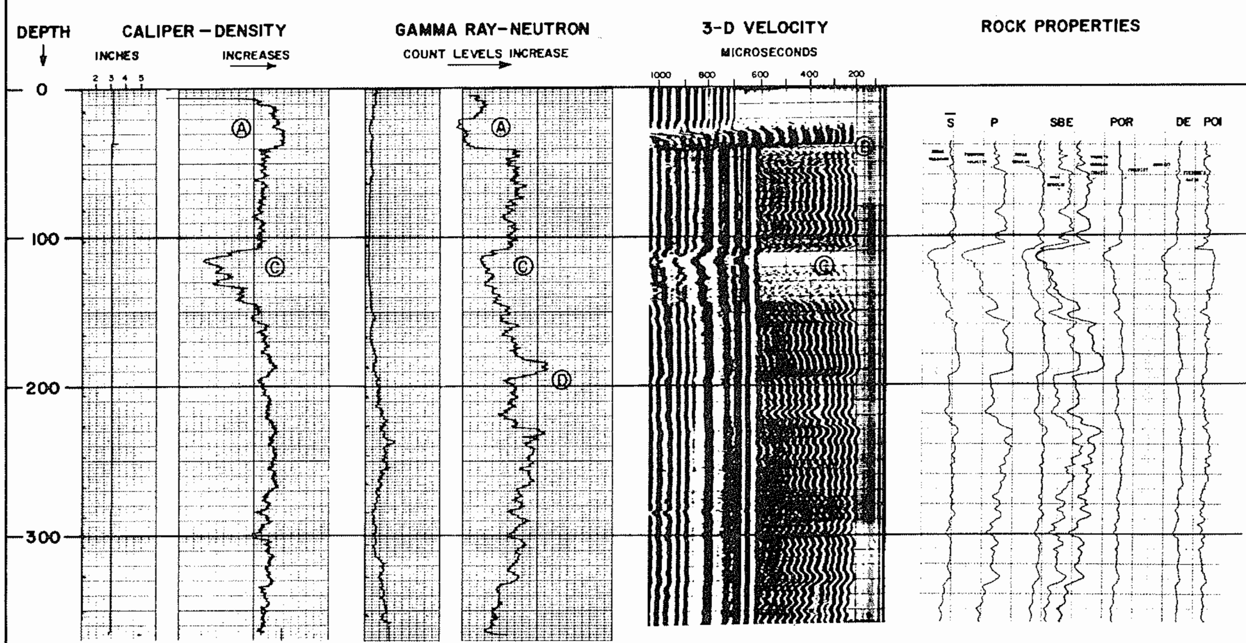
GEOLOGIC PROFILE

EXHIBIT- 3A



GEOLOGIC PROFILE IN DES PLAINES DISTURBANCE AREA

EXHIBIT - 4



GENERALIZED LITHOLOGY

SYSTEM	GROUP	FORMATION	MEMBER	DESCRIPTION
QUATERNARY				ALLUVIUM/TILL
SILURIAN		RACINE		DOLOMITE, light gray, reef facies: crystalline, massive and porous, interreef facies: Argillaceous, thin bedded with scattered porous chert nodules and thin chert beds. Reef-flank facies may include beds with dips to 30°.
		JOLIET	ROMEO	DOLOMITE, light gray, fine grained, crystalline, dense.
			MARKGRAF	DOLOMITE, light gray to bluish gray, argillaceous, fine grained, some chert, some thin greenish gray shale partings.
			BRANDON BRIDGE	DOLOMITE, pinkish gray, fine grained, red and green shale partings.
		KANKAKEE		DOLOMITE, gray to pink, medium grained, scattered chert nodules, prominent green shale partings to 0.5 inch thick.
		EDGEWOOD		DOLOMITE, cherty, light gray to gray, fine to medium grained. Becomes darker in color and more silty and shaly with depth, especially where thick.

ROCK PROPERTIES

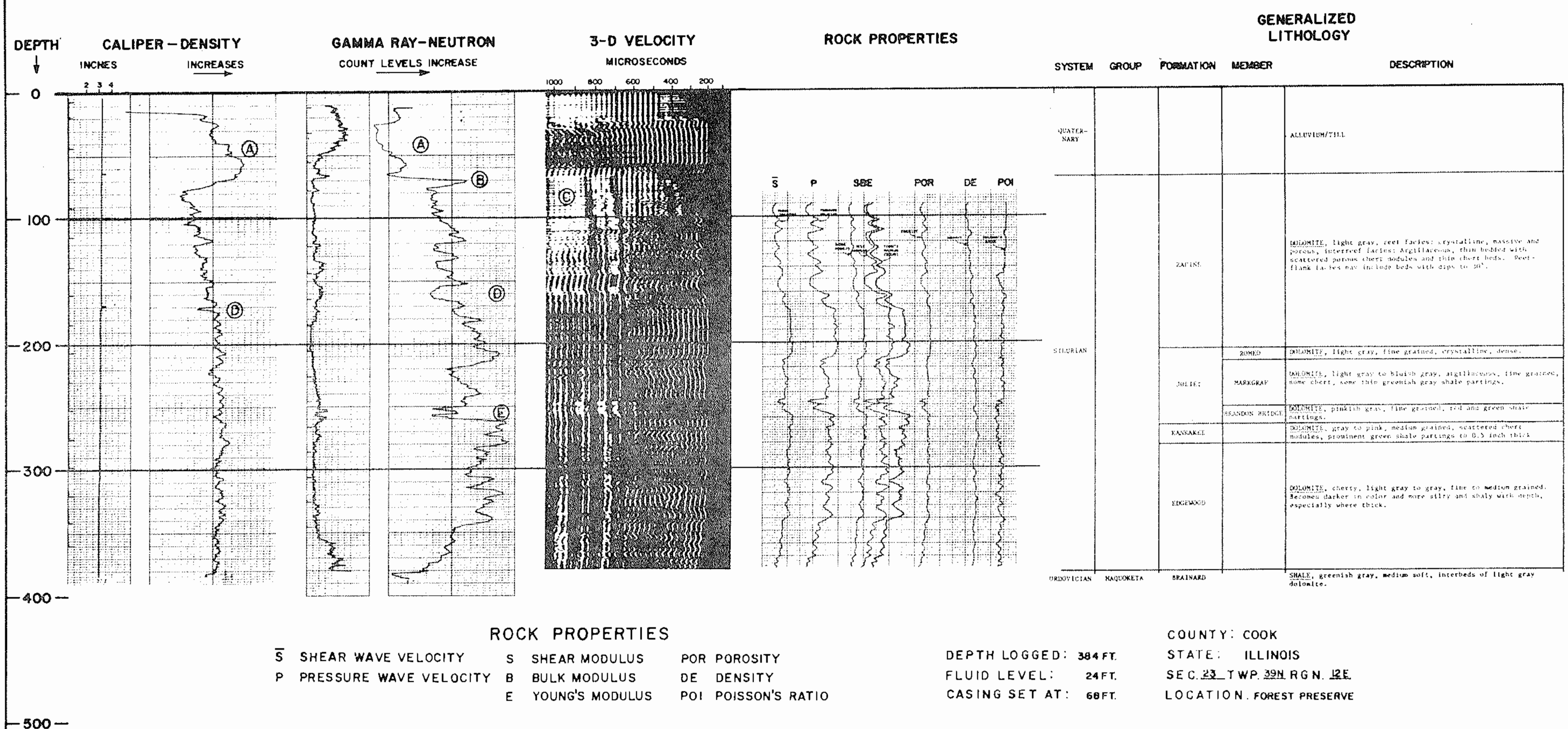
$\bar{S}$	SHEAR WAVE VELOCITY	S	SHEAR MODULUS	POR	POROSITY
P	PRESSURE WAVE VELOCITY	B	BULK MODULUS	DE	DENSITY
		E	YOUNG'S MODULUS	POI	POISSON'S RATIO

DEPTH LOGGED: 365 FT.
FLUID LEVEL: 22 FT.
CASING SET AT: 37.7 FT.

COUNTY: COOK
STATE: ILLINOIS
SEC. 35 TWP. 39N RGN. 12E
LOCATION. SOUTHVIEW AVE. AND DES PLAINES RIVER

GEOPHYSICAL DATA
(BY BIRDWELL DIVISION)
BORING DI-AS-104

EXHIBIT – 5



ROCK PROPERTIES

$\bar{S}$ SHEAR WAVE VELOCITY	S SHEAR MODULUS	POR POROSITY
P PRESSURE WAVE VELOCITY	B BULK MODULUS	DE DENSITY
	E YOUNG'S MODULUS	POI POISSON'S RATIO

DEPTH LOGGED: 384 FT.
FLUID LEVEL: 24 FT.
CASING SET AT: 68 FT.

COUNTY: COOK
STATE: ILLINOIS
SEC. 23 TWP. 39N RGN. 12E
LOCATION. FOREST PRESERVE

GEOPHYSICAL DATA
(BY BIRDWELL DIVISION)
BORING D2-AS-109

EXHIBIT - 6

A STUDY OF THE FOREST CITY CREEP SLIDE ON THE OAHE RESERVOIR

By

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North Carolina Department of Natural Resources & Community Development
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and

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I. INTRODUCTION

Throughout most of its length through South Dakota the Missouri River is bounded on the east by relatively high bluffs marking the western extent of the Pleistocene glaciation. The bluffs expose a large thickness of the Pierre Shale formation capped by glacial till. Engineering experience with the Pierre Shale and similar Cretaceous shales in the Northern Great Plains has shown them to be unstable particularly in cut slopes.

Construction of the Oahe Dam project in the late 1950's required relocation of the only two river crossings in the north half of South Dakota. At Forest City, US 212 was relocated about 2 miles south of its original crossing and a ¾ mile long bridge 160 feet above the original river elevation was constructed. An approach fill up to 40 feet thick was constructed to allow a gentle descent of the road from the plateau down the bluff to the bridge. The road relocation and bridge construction were completed in 1959.

Indications of problems were first observed during a bridge inspection in the winter of 1965, about the time the Oahe Reservoir reached normal pool. In 1968, pavement cracking began and has continued in the approach fill. Periodic site studies were begun in 1968 by the South Dakota Department of Transportation, Foundations and Geology Section. Some instrumentation was installed which revealed subsurface movements. But, because movements were slow and no sudden mass failure occurred, some questions arose as to the exact nature of the problem. There was even some questions whether the observed movement should be called a "landslide".

In order to gain a fresh perspective of the problem, the South Dakota Department of Transportation retained Law Engineering Testing Company in 1975 to make a complete review of all available data and provide an independent interpretation of the movements as well as any possible recommendations for remedial measures and future work needed. This paper is drawn from that study.

II. REGIONAL AND AREA GEOLOGY

The site is located on the "Breaks" or bluffs of the Missouri River at the west edge of the Glaciated Missouri Plateau (Coteau du Missouri) section of the Great Plains Geologic Province. The region is characterized as a nearly flat plain developed on glacial soils, with the only significant relief being along a few deeply entrenched major streams, which are bordered by relatively steep valley walls or bluffs. Prior to the Illinoian glacial advance, the area was on the Southeast bank of the Cheyenne River, which drained northeastward toward Hudson Bay (Flint, 1955). Glaciation blocked the drainage of the Cheyenne and other northeastward flowing rivers forcing the Missouri River to assume its present channel with its drainage redirected southward (Hunt, 1967).

The glacial soils in the region typically are 20 to 30 feet thick and consist of compact slightly sandy silty clay till with erratic gravel and boulder concentrations. Some end moraines and glacially filled pre-glacial river channels may contain till thicknesses of over 100 feet.

In addition to till, an 8 to 10 foot thick widespread blanket of loess and alluvial soils many tens of feet thick in the major valleys form surficial deposits.

Beneath the surficial loess, alluvial, and till deposits the region is underlain by the Pierre Shale formation, which consists primarily of bentonitic marine shales of Cretaceous age. A stratigraphic chart (Table 1) and a regional correlation chart (Table 2), both from Bruno Petsch (1946), show the regional stratigraphic relationships. Note that the upper Pierre formation correlates in age with the lithologically similar Bearpaw formation of Montana and Alberta. In the region of the site, the Pierre formation is several hundred feet thick and is nearly flat lying.

The region is relatively inactive seismically, and is classified as a "low risk" area on the NOAA seismic risk map.

REGIONAL ENGINEERING GEOLOGY OF THE PIERRE SHALE

Numerous references are found in the geologic literature concerning the slumping tendency of slopes in the Pierre Shale. Of special note is the work performed by Crandell (1952, 1958), which included terminology discussions and classifications of different types of landslides. In his 1952 paper, Crandell makes the following salient points:

1. **The Missouri River Trench is almost continuously lined with old landslide scars;**

2. The Missouri Trench was cut in Sangamon Interglacial time, and some of the landslides probably occurred at that time. Alluvial fill in the river channel is usually about 90 to 120 feet thick; the channel cut and most of the backfill is probably of Sangamon age.

3. Most of the landslides are late Wisconsin in age (post-Tazewell).

4. **"No new slumps in which there has been unit block movement are known to have occurred within historic time."** (This statement was in 1952, prior to the Missouri River impoundments), (emphasis ours).

The Pierre shale is closely jointed, with joint frequencies on the order of inches. Though the bentonitic layers are highly impermeable when nondessicated, jointing between bentontic layers provides avenues for groundwater migration (Scully, 1973). Joint measurements by Scully indicate that jointing orientation is controlled primarily by local structures and topography rather than by regional tectonic history.

The Pierre shale is reported to be extensively faulted, and landslide movement planes, marked by gouge and slickensides, are often correlated with previous fault planes. However, many of the features which have been identified as faults may be non-tectonic slump surfaces in old landslides.

The instability of the Pierre shale has also been noted in engineering work. Studies of the strength and the behavior of the Pierre at the Oahe Dam have been published by Underwood (1961), Knight (1963), Fleming, Spencer, and Banks (1970). Scully (1973) published an excellent study of landslides in the Pierre along highways in central and southern South Dakota. Also, Hamel (1973) published an excellent study of a Pierre stratigraphic equivalent at the Ft. Peck Dam in Montana.

III. SITE GEOLOGY

Topographically, the plateau in the site area is at about elevation 1900 and the Missouri River natural level is at about 1500, a relief of 400 feet over a horizontal span of about 4000 feet. The slope which Highway 212 follows from the plateau down to the valley level is step-like or hummocky, reflecting ancient (now active) landslide activity (Fig. 2, 3, 4).

A geologic map of the site area is shown on Fig. 5. Recent deposits consist of 5 to 10 foot thick layer of loess which covers much of the landslide site, and up to 120 feet of sandy silt and clay alluvium in the Missouri River flood plain. Materials of Pleistocene age are end and ground moraine of glacial till. Underlying the till is the Cretaceous Pierre shale.

Core borings at the site made by the Corps of Engineers in the 1950's and auger borings made by the South Dakota Department of Transportation since 1968 show the following sequence of materials in the landslide area:

Man-made fill (in approach area) - soft to firm (?) gray and brown sandy silty clay with some gravel and boulders (probably borrowed from cuts in till). Penetration tests difficult to interpret due to gravel and boulders, but matrix apparently soft.

Till - very stiff to hard brown sandy, silty, or gravelly clay.

Weathered shale - soft to hard brown to gray silty clay and relict shaly laminations.

Shale - hard (standard penetration resistance greater than 100 blows per foot) light to dark gray siliceous shale, bentonitic shale, claystone, and thin bentonitic seams. Occasional gray cobble to boulder-sized tabular carbonate concretions.

In general, the fill thickness is in the range of 20 to 40 feet. The till thickens southward, probably a reflection of erosional removal of till from older to lower slump blocks nearer the river; in the fill area the till is 10 to 30 feet thick on the plateau it is about 150 feet thick. In most areas the till is 40 to 80 feet thick. The weathered shale is typically 5 to 10 feet thick according to the test borings. The main strata encountered in the borings are shown on Figure 6. Previous geologic mapping in the Forest City area by the SD Geological Survey indicates the following general Pierre Shale stratigraphy at the landslide site: (see top of next page)

An example of the thin bentonitic layering in the site area is taken from a detailed section measurement of the Oacoma unit at nearby Whitlock's Crossing (Gries, 1939):

Elevation*	Member or Zone	Description
Above 1730	Virgin Creek Member	Bentonitic shale, prominent pair of bentonite layers about 20 feet above base.
1732-1628	Sully Member Verendrye Zone	Bentonitic shale, claystone, bentonite disseminated, forms "Gumbo" soil when weathered.
1628-1620	DeGrey (Oacoma) Zone	Interlayered shale and abundant thin (mostly 1/8 inch to 1/4 inch) bentonite seams.
1620-(Base probably below elev. 1470)	DeGrey (Agency) Zone	Siliceous shale with some bentonite seams; abundant bentonite seams at about elevation 1550.

*These contact elevations are projected from measured sections at Whitlock's Crossing about two miles north of the site and are probably accurate only within ±10 feet. Stratigraphic interpretations based on data by Russell (1930), Searight (1937), Gries (1939, 1942), Petsch (1946), Mickelson and Klein (1952), Crandell (1950), Montgomery (1962), and Agnew and Tychsen (1965).

Layer No.	Thickness Ft.	In.	Material
19		¼	Bentonite
18	1	9¼	Shale
17		½	Bentonite
16	1		Shale
15		1	Bentonite
14		8	Shale
13		1½	Bentonite
12		7½	Shale
11		½	Bentonite
10		5½	Shale
9		½	Bentonite
8		8	Shale
7		½	Bentonite
6		4	Shale
5		¼	Bentonite
4		8	Shale
3		¾	Bentonite
2		5	Shale
1		2	Bentonite

Gamma logs were recorded by the SD Geological Survey on Borings M-8, M-9, M-10 and M-11. Signatures on shale in Boring M-8 match well with signatures about 15 higher in elevation in M-11, indicating 15 feet more vertical slumping at M-8 than at M-11, assuming a horizontal and nonfaulted pre-landslide condition. Tentative correlation of radioactive peaks at elevation 1644 in M-8 and 1658 in M-11 with reported data from Virgin Creek bentonites at about elevation 1745 (± 10') forms a pattern consistent with the ground elevation difference at the back scarp. These data indicate that the shale at the location of M-8 and M-11 has slumped vertically about 100-115 feet (± 10') from its original position. At M-9 and M-10 the materials cannot be correlated by this method, probably because of the additional slumping and disturbance.

No data are available on groundwater levels prior to construction. Our interpretation of the location of the present water table is presented on Fig. 6. In the approach fill area the groundwater table probably closely follows the adjacent reservoir level, but insufficient groundwater data are available for close comparison. In most places, the water table is about 20 to 40 feet below the ground surface.

A groundwater seepage area exists about 100 feet east of Station 151 + 00 and is indicated on Fig. 2. This seepage area correlates with the location of the gulch which was filled by failed material during approach fill construction. This is the only groundwater seepage area known at this time in the landslide area.

A flowing aretesian well (several hundred feet deep and tapping the Dakota Formation or deeper units) is located at the top of the scarp (south of M-12). Though the artesian groundwater head has no direct significance to the landslide, we understand that the overflow is piped down the slope to the service station within the slide area, and that the overflow from that point is dumped directly on the ground surface; therefore, its contribution to the shallow water table could be significant in a localized area.

The permeability of the till at the site is primarily governed by the location and abundance of landslide cracks. Some granular seams may exist, and a buried (Iowan) loess is possible, but the cracks probably serve as the most important avenues for water movement, and certainly for entry of rainfall and meltwater. Outside of pavement areas, none of the surface cracks have been sealed.

The permeability of the underlying shale is controlled by landslide cracks and by closely spaced jointing.

In addition to the information presented above, the following data were obtained by the South Dakota Department of Transportation during their studies.

1. Precipitation records from a U.S. Weather Bureau Station in Gettysburg, South Dakota. About 20 inches of precipitation are received a year, mostly in the summer.
2. Reservoir level records.
3. Soil test data, limited to classification type tests.
4. Seismic refraction survey data.
5. Electrical resistivity survey data.
6. Inclinometer data.
7. Ground surface surveys of boring locations and road points.
8. Bridge movement surveys of abutment and pier locations.
9. Tiltmeter surveys on bridge piers.
10. Aerial photographs at various times and scales dating to 1938.

Significant features of the above data will be discussed but due to space limitations, little of the data will be presented.

IV. CONSTRUCTION AND PERFORMANCE HISTORY

Planning for the US 212 relocation began in the early 1950's. The Corps of Engineers performed subsurface borings and provided the data to the DOT for their use in design. One of the Corps' drawings contains indirect references to slumping in the vicinity of the south approach by noting an "assumed top of slumped shale" and an "assumed top of structurally firm shale." The Corps logs also noted some thin bentonite seams at various levels. However, the Corps attributed the hummocky ground in the south abutment to glaciation rather than landslides.

The design of the bridge and its foundation was done by the DOT. The bridge consists of a two-lane steel structure combining suspended through trusses, cantilevel through trusses, and continuous plate girder units for a total length of about 4600 feet. The deck is about 160 feet above the original river level (about 40 feet above the high reservoir level). Concrete piers with steel H pile foundations support the structure. Spacing of the piers is about 375 feet for truss sections and varies from 156 to 204 feet for the plate girder units. The steel H piles are reported to have been driven through the alluvium and fill into the shale to maximum depths of 140 feet (Engineering News Record, 1959). At the ends of the bridge the abutment sills are supported on timber piles reported by Sverdrup and Parcel (1973) to be 30 feet long. Two H piles were also driven to reported depths of 60 feet under each girder support point at the sills.

Construction began in 1956 and was completed in 1958 at a cost of 4.8 million dollars. No major difficulties were experienced with the bridge construction itself, but some problems were encountered during placement of the south approach fill. Construction of the approach fill necessitated filling a gully at about Station 153 + 50. Failure of the fill between this location and the end of the bridge (Station 143 + 80) occurred with the fill and part of its foundation reported to have moved eastward during fill placement. The failed material was pushed further eastward, forming a berm, which was braced against the opposite side of the gully. Filling was then completed without further difficulty.

Filling of the Oahe Reservoir began in 1958 and reservoir reached its normal pool of elevation 1600 to 1620 at the bridge in 1968. In 1959 or 1960 the site was studied as part of an area geologic study. The report of this study noted "extensive slumping" in the bluffs but specifically noted "No highway repairs occur along this new highway strip," (Montgomery, 1962).

No signs of bridge distress or pavement problems were noted until December 1965. Significant events since 1965 are listed below.

1965 Bridge expansion joint at south abutment found completely closed during December inspection by Sverdrup and Parcel Engineers.

1968 Pavement cracking noted in approach fill about 500 feet from south end of bridge, same general area where construction fill slide occurred. South Dakota Highway Department Foundations and Geology Section performed five auger borings with some penetration testing, seismic surveys, and electrical resistivity surveys in the approach fill area.

1969 Pavement cracking continues. Maintenance overlays required for 1300 foot section nearest bridge and for a 100 foot section about 4300 feet south of the end of the bridge. Annual to semi-annual work needed until early 1970's. Frequence of repairs increased to semi-annual to quarterly in mid 1970's.

1970 Annual bridge inspections begun by the DOT Bridge Maintenance Engineer.

1972 Intermittent second order surveys of coordinates on bridge using electronic distance measuring equipment begun by DOT.

Foundations and Geology Section installs three inclinometer casings and makes roadway crack map.

1973 First inclinometer casing installed found sheared in January. Subsurface movement indentified in a second inclinometer. Depths of movement suggest large, deep-seated slide not confined to approach fill. Tiltmeter survey of bridge performed. Three additional inclinometers installed in spring; all three shear by end of year. Sverdrup and Parcel issue report on bridge distress recommending structural extension of south end of bridge.

1974 Four more inclinometers installed in spring; one shears by end of year.

1975 New inclinometers installed at locations of 1973 holes. Piezometer installed in deep boring on plateau.

Consultants retained by DOT to review data and recommend course of action.

1976 South end of bridge extended 30 feet to south and special expansion joints installed. Road resurvey found 5'8" displacement of pavement from original location at start of approach fill and 5" lateral displacement of bridge at abutment. When bridge beams cut free of abutment they moved 3" back toward original position when warmed by sun.

In reviewing the above information, three points should be noted. First, a geologist recognized at the end of construction but prior to the onset of serious slides that the site is on an ancient landslide. Second, no distress was observed until several years after completion of the project. The first signs of distress were noted as the reservoir pool approached its normal operating level. Third, the magnitude of the problem could not be grasped until inclinometers were installed and the depths of movement identified.

V. ANALYSIS AND EVALUATION OF DATA

Analysis of the available data tried to resolve the following questions:

1. Is the present ground cracking pattern in the area related to any historical slide scar pattern existing prior to construction?
2. What mass is actually moving?
3. What is the subsurface stratigraphy?
4. Are the movements related to precipitation or fluctuations in the reservoir?
5. What is the relationship of the reservoir to the movements?

If the above questions could be reasonably answered, it was hoped that the answers would indicate if remedial action should be considered, the nature of possible remedial measures, and the future behavior of the mass.

Inspection of preconstruction aerial photographs of the site area dated 1938, 1939, and 1950 reveals numerous slide scars on the south bank of the river; the major scars are shown on Figure 2. A half-mile long bulge in the river bank at the site area indicates the toe of the largest slides may have been at or below river level (about elevation 1495). Detailed study of these and later photographs indicates several generations of slide development, based on geometry and degree of post-movement erosion of slide blocks. The hummocky ground in the breaks of the river along the US 212 Route and in the vicinity of the bridge is very strongly indicative of landslide topography; however, in the preliminary site studies the US Corps of Engineers thought the hummocky ground was an end moraine feature, as indicated in their report to the DOT (US Corps of Engineers, 1952). The aerial photographs indicate the slide is over a mile long and over ½ mile wide.

Of special significance on the pre-construction aerial photographs are relatively young (un-eroded) arcurate slide scars bordering the topographic knob which was selected to be the end point of the bridge. Fig. 7 illustrates the original ground topography in the approach fill area and the crack patterns existing before and after construction. The locations of the young scars correlate closely with the major pavement cracking areas in the approach fill. Also, the easternmost scar correlates with the sliding experienced during fill placement.

The crack patterns before and after construction suggested large masses were in motion along relatively deep planar surfaces. The present position of the rear-most slide boundary was probably developed by progressive failures of smaller blocks therefore creating the existing "horst and graben" appearances near the reservoir.

SUBSURFACE STRATIGRAPHY AND MOVEMENT

The subsurface stratigraphy has previously been discussed. Based on the widely spaced borings, the profile in Figure 6 was developed. Unfortunately, none of the borings incorporated rock coring so the location of bentonite seams, weathered zones or sheared zones in the shale cannot be determined. Figure 6 includes for comparison an estimated geologic column based on projections from previous geologic mapping in the site area.

Shown on Figure 2 are the locations of

inclinometers and direction of movement. There is no apparent pattern to the movement depths. Several factors make interpretation of inclinometer movements in this situation difficult. First, based on the hypothesis developed from the aerial photostudy, a deep planar movement may extend from near the original river level back and up toward the plateau with movements of individual blocks occurring above the deep surface. If the movements of the shallow blocks are rapid compared to the movement of the entire mass, an inclinometer may shear off due to shallow movement before an indication of deep movement can be seen. Secondly, the inclinometers may not have been installed deeply enough to reach the deep planes. Finally, the actual failure surfaces may be stepped or undulating rather than planar.

The crack maps and the subsurface movement data, indicate that the present slides are developed on planar surfaces (probably many glide-planes) which are at around elevation 1560 in the vicinity of the bridge and which overlap upward to about elevation 1590 below the back scarp at the edge of the plateau. Based upon the location of the back scarp and the shearing depth in inclinometer M-11, a planar surface can be reasonably extended to approximately the old river, elevation of 1490 roughly as indicated on Figure 8. Such a planar surface has an average dip of about 2° toward the river which corresponds well with failure surface inclinations documented for slides in the Pierre and equivalent formations.

This planar surface or set of individual planes probably represents the deepest and oldest landslide movement at the site. More recent movements which are occurring in the vicinity of the bridge indicated by slope inclinometers at between elevation 1560 and 1570, may form a higher level block. Numerous near vertical (perhaps 60° dipping) branches of the deeper shear zones intersect the ground surface representing the boundaries of "horst and graben" sliding blocks, some of which have been reactivated as individual blocks creating new ground cracks, but all of which are moving at a slow rate toward the reservoir.

In the approach fill area, sliding appears to be occurring both along planes coincident with the shear surface on which the whole mass is moving (elevation range 1560 - 1580) and along higher level scars from pre-construction slides on the sides of the topographic knob upon which the fill is located. A very good correlation was found between the slide scar on the east side of the knob and the indicated subsurface and surface movements (see Fig. 7). The possible slide scar on the west of the knob appears to be near the toe of the present fill and does not correlate well with the observed cracks and movements; however, progressive development of the now buried slide was inferred from the presence of longitudinal cracks in the fill.

STABILITY ANALYSES

A limited amount of slope stability analysis was performed to examine qualitatively the effects of changes in the site caused by construction of the road and formation of the reservoir. Due to a lack of specific site data on soil strengths and preconstruction groundwater levels, the results were not considered quantitatively useful or reliable. Separate analyses were conducted for the large block movements shown on Fig. 8 and for a cross-section through the approach fill represented by the typical section on Fig. 9. The ICES version of the Morganstern-Price analysis method was used. This method allows the shape of the failure surface to be other than circular; thus, the planar type surfaces postulated for the sliding could be used directly in the analysis. Because of the slow nature of the movements, drained conditions were assumed.

No specific site data on soil strengths were available and assumptions were necessary. Properties of the fill and till were assumed based on experience and tests of similar soils in South Dakota. Properties of the weathered shale were based on published test data for the Pierre Shale. For shale along presumed failure surfaces it was assumed that the historical nature of the movements would have reduced the strength to residual values. Several preliminary analysis were made using the largest block movement and varying the assumed strength of the shale along the failure plane until a computed factor of safety of 1.0 was obtained. The friction angle thus obtained (7°) was in good agreement with values reported by Hamel (1973) from back calculations on other landslides in overconsolidated clay shales, and with test results and analyses of Pierre shale at the Oahe Dam site reported by Knight (1963). Table 3 summarizes the soil properties used in the analyses.

Analyses were conducted 1) for slopes existing prior to construction, 2) after fill placement but before raising the reservoir, and 3) with full reservoir pool present. The numerical values of calculated factors of safety were not considered particularly significant because of the uncertainties as to soil assumptions; rather, the relative change in factor of safety for the various conditions was studied.

Figure 8 illustrates the general configuration analyzed for the large block movement. With respect to the entire mass involved in sliding, the fill addition is very small both in terms of area of the failure plane affected and increased stresses at the level of the failure plane. With ground water levels at estimated prereservoir levels, adding the fill slightly increases the factor of safety of the profile shown in Figure 8.

Raising the water level to normal pool level increases the weight of soil previously not submerged, but reduces the effective normal stresses acting on the failure plane by 15 to 20 percent. For profiles outside the area of fill, an increase in water level reduces the factor of safety by 10 to 15 percent. Within the fill area, the reduction in effective stresses caused by raising the water level is offset to some extent by the stress increase due to added fill. Analyses indicate the combined effect of the modifications produces a slight decrease in the overall factor of safety. Although the decrease is slight, ancient slides which had reached tenuous natural equilibrium were probably reactivated and those already creeping accelerated. As previously discussed, post-reservoir construction slide activity is noted by others to be present at many locations along the shore of the reservoir; the phenomenon observed at this site is not unusual.

Analyses for cross-sections near the bridge were done primarily to evaluate the possible location of failure surfaces that fit the observed movements so that remedial measures could be evaluated. One of the Corps of Engineers 1954 borings nearest the approach fill slide area notes a bentonite seam at elevation 1565 within the zone of movement shown by inclinometers. This fact was used to help identify a possible failure surface. A reasonable agreement between a geometrically satisfactory failure surface and the indicated movements could be obtained on the east side of the roadway, but not on the west side. The postulated location of the failure surface on the east side is indicated on Figure 9. A similar configuration probably exists on the west side.

The failure surface shown on Figure 9 for the east slide indicates movement at right angles to the road. If the overall movement pattern postulated in this report is correct, movement is also occurring simultaneously along the flat portion of the failure plane in a direction toward the bridge. The inclinometer movement directions tend to imply the relative movement is greater perpendicular to the road than toward the river, however.

Because the fill and till soils contribute a large proportion of the driving and resisting forces for failures in the approach fill, errors in assumptions for their strengths would have a much greater affect on the analysis of the approach fill than for the large block slides. For the strength properties assumed, fluctuations in the reservoir level would have a relatively minor effect. However, as for the block slides, the net effect on the old slide scars existing prior to construction would be a slight decrease in factor of safety after the reservoir reached normal pool.

About 4300 feet south of the bridge, a small isolated section of fill is experiencing pavement cracking and an apparent bulge in the east side of the fill. This location corresponds to the area where the road route crosses slide scars representing boundaries of the major movement blocks (see Fig. 2). The problems at this point are probably due to stretching of the fill by downhill movement of the large block.

MOVEMENT RATES AND RELATIONSHIPS:

There were relatively little long-term data available on the rates of movement for the various blocks involved in sliding. Survey data did not contain enough individual observations to establish any trend lines. The slope inclinometer casings generally remained functional for periods of 9 months to a year and usually showed 2 or 3 different rates during their life. Most of the inclinometer data were characterized by an initial rapid movement, a central slower rate of movement, and, usually, an acceleration in rate just prior to inclinometer shear. This pattern can be attributed to initial take-up of slack in the inclinometer hole, then long-term straining until the grout column has been sufficiently cracked by movement to lose its rigidity. Because of this combination of dissimilar materials in the hole, it is difficult to use the rates of movement indicated by the slope inclinometers to indicate the true rate of slide movement.

Some indication of the relative rates of movement of various parts of the slide area can be obtained by considering the life span of each inclinometer prior to shear. These data showed that the east section of approach fill is moving the most rapidly. The west section of fill appears to be moving much slower - at the time of this writing inclinometer M-10 had been active for better than 18 months and was still not sheared. Movements of the larger failure blocks as indicated by M-7, M-11, and M-8 appear to be at a relatively slow rate as these inclinometers required approximately 1 year before shearing.

Considering all of the ground survey and inclinometer movement available, it appears that the ground surface of the bluff slope is moving about 1/3 to 1/2 foot per year and that the bridge abutment is moving a few tenths of an inch per year.

None of the available data suggested a relationship between slide movement and either precipitation or reservoir fluctuation.

VII. POSSIBLE FUTURE MOVEMENTS

The available evidence at the time of our study suggested that all of the slide movements were slow and could be classified as creep landslide distortions as opposed to rapid landslide activity. The most rapid movement was occurring in the east approach fill area, particularly in the vicinity of the inclinometer M-5.

Nelson and Thompson (1974), have provided a recent summary of hypotheses on creep failure of slopes in clay and clay shale. Their discussion indicates that many natural slopes may undergo creep for many years without experiencing acceleration of the rate of movement or ultimate failure in terms of rapid sudden displacements. However, cases are reported where slow creep movements have gradually accelerated into a sudden failure. Whether or not the end result of creep is a sudden failure appears to depend upon the stress level acting upon the movement plane in relation to the maximum shear strength.

The raising of the reservoir combined with the placement of the approach fill has acted to slightly decrease the effective normal stresses acting upon the failure planes and thus to slightly increase the shear stress level in the soil. The amount of change by these two effects is actually relatively small in the area of the approach fill, being only about a 5% to 10% decrease in effective stress at the failure plane. Thus, it would seem likely that the effects of the higher water level and the addition of the weight of the fill have not greatly increased the shear stresses at the failure planes. Based upon this reasoning, a rapidly moving landslide would not be expected to develop from the present creep movements. This conclusion is supported by observation of other reactivated landslides on the reservoir margins; they have remained typically creeping landslides.

In the vicinity of inclinometer M-5 in the approach fill, shallow movements were found to be increasing. The original M-5 inclinometer required 2 months to shear while the recent installation to replace the original M-5 sheared in about 1 month. Even better evidence for accelerating (though still very slow) movement rates was the increasing frequency of pavement repairs. An ultimate sudden failure may be possible in the approach fill.

VIII. REMEDIAL CONSIDERATIONS

The movements occurring at this site consists of slow creep movements involving a very large mass of material, and more rapid movements of the approach fill. The problems consist of structural distress of the bridge and continued pavement cracking. The pavement cracks have been kept repaired by continued maintenance by DOT district personnel since problems were first noted in 1968. As discussed earlier, the frequency of overlay and patching has had to be increased during the last several years. Due to the anticipated high costs of repair to stop movement, it may be economically justifiable to not stop movement, but only maintain the pavement. If the bridge distress can be treated before electing this alternate, however, additional stability studies would be required to assess the danger of a sudden landslide developing as a result of continued creep. The only area which appears to have potential for a more rapid movement is the east side of the approach fill; this area should be stabilized even if the remainder of the road is only maintained.

For the approach fill remedial measures considered included structural support through the problem area, internal drainage of the fill, structural retention for the fill, and berms at the toe of the fill.

Any structure used to replace the approach fill would be subject to movements toward the reservoir by creep of the large mass. Due to numerous boulders in the till, construction of a structure would be difficult and costly. Also, the road would have to be closed to traffic during the construction. A structure was thus not considered practical.

Water levels in the approach fill near the bridge are not much higher than the reservoir level, and measurements of water levels in boreholes indicate good communication with the reservoir. Thus, internal drainage would not be practical; analyses also indicate the slight reduction in water level would not offer significant improvement.

Structural restraint of the approach fill by drilled-in piles or caissons, or by using the Fondedile reticulated pile system, appears promising, particularly for the west side of the approach fill. Such a system would extend through the movement planes between elevation 1560 and 1570 and would be designed to hold the road in position. The structure location would be near the edge of the pavement. Before such a structure could be designed, additional data on movement within the fill would be required. The cost of a structural system was estimated to be in the range of $1500 to $2000 per plan foot.

The use of berms to provide restraint is feasible where the toe of a moving mass is identifiable and is readily accessible. For the slide in the east side of the approach fill, the geometry of movement appears relatively well-defined by pavement cracks, the old slide scar, and the inclinometer data. Fig. 10 shows the berm plan location and Fig. 11 illustrates a possible configuration. Such a berm would stop the easterly movements perpendicular to the road in the area between Stations 144 and 150; however, the berm might itself cause some additional movement parallel to the road at the toe of the fill. The design of a berm would have to consider this possibility.

For stabilizing the entire mass carrying the approach fill and south end of the bridge to the north, which is the cause of the bridge distress, extremely costly measures would be required. Unloading, groundwater drainage, and road relocation have been considered as remedies in this study. All on-site remedial measures considered acknowledged that the existing bridge abutment would be replaced and additional movement capability built into the new abutment.

One common means for stabilizing moving earth masses is to remove the driving force by excavation at the head of the slide. Work done by other investigators in the clay shales of Montana and Alberta suggests, however, that unloading often increases the rate of movement rather than slowing it. This effect is due to the very high overconsolidation of the clay shale which creates an expansion tendency upon unloading. By expansion, the clay shales absorb water and lose strength. A second major consideration in evaluating unloading would be the potential for initiating further movement in the plateau area behind the present rear scarp. Finally, unloading would require purchase of large additional right of way or easements, and disposal of the removed material would present problems.

Structural restraint using a drilled-in system of piles, possibly of the Fondedile reticulated type, may be feasible. More data on the movement pattern and material properties and detailed stability analyses would be necessary to evaluate the feasibility. Costs for any such system of this type would likely exceed 4 million dollars.

The use of chemical grouts injected along the movement plane has been used in some cases as a means of stabilization by increasing the strength. Here, the entire mass would require grouting to avoid shifting the movement to another adjacent weak layer. The ability to achieve adequate grout penetration is questionable as it depends upon the degree of fracturing and jointing in the shales. Grouting was not considered feasible.

Internal drainage to lower the water levels within the moving mass and thus increase effective stresses on the failure plane has been successful for numerous slides. In this case, however, horizontal drains would be difficult to install due to the boulders in the glacial till, and lengths on the order of 900 to 1000 feet would be required to achieve even a 10 to 20 foot lowering of the water level. In the area near the reservoir where the greatest increase in pre-construction water levels has occurred, essentially no change can be effected by horizontal drains due to the reservoir. The best use for horizontal drains would be at areas where local surface seepage is present.

The only way to achieve any significant increase in factor of safety by lowering water levels would be to install numerous vertical wells with pumps. The area of influence for such wells cannot be estimated based on the present data and would be extremely erratic in any event, since the degree of fracturing and jointing in the shale would control the behavior. The area which could be most improved by vertical well pumping would be the south portions of the large blocks where the reservoir has not caused much increase in pre-construction water levels. The overall effect on the moving masses would have to be evaluated by further studies to determine if movement of other blocks nearer the reservoir would be affected.

The use of drainage to route surface runoff away from the approach fill area would be of some benefit. Also, filling the existing open cracks would reduce the amount of rainfall entering the slide mass. Locations of suggested drainage features are shown on Fig. 10.

Relocation of the entire road and bridge to a stable location may not be an unreasonable alternate. A brief study of aerial photographs covering a distance of about 1 mile in each direction from the present location indicates that a favorable site exists about 1 mile west of the present site. The road route could follow the old Forest City Railroad grade, a relatively gentle natural grade to water level with no apparent landslide scars visible on aerial photographs. The bridge at that location would, however, be considerably longer than at present.

IX. FURTHER STUDY

While a good program of data gathering, particularly with respect to subsurface movements, has been conducted to date, there are several areas in which additional data are needed: surface movement patterns, groundwater information, subsurface stratigraphy, and shale strength properties.

Several additional borings incorporating rock coring, Denison or Pitcher sampling of the shale, electric logging, and installation of piezometers and inclinometers would be beneficial.

A survey grid with points 400 to 500 feet apart generally, and 200 feet apart in areas of fill distress, would provide data on surface movement patterns and rates. Both horizontal and vertical deviations should be measured.

Laboratory testing of shale samples should concentrate on determining stress-strain properties and time-related loss in strength as well as conventional strength properties.

SUMMARY AND CONCLUSIONS

Based on our study of the existing data we believe the following conclusions about the problems at Forest City can be drawn:

1. The approach route of US 212 in its descent from the Plateau to the bridge crosses a complex of ancient Pleistocene and later

landslides.

2. The geologic setting of the site is in an area where numerous ancient landslides were recognized prior to construction although there was apparently no record of actual slide movement in the written historic record prior to construction of the highway.

3. Creation of the Oahe Reservoir raised groundwater levels sufficiently to initiate slow creep movements along the shear surfaces of the ancient landslides.

4. The creep movements involve a mass of approximately 75 million cubic yards. The moving mass extends to the original location of the Missouri River and involves the foundation for the south abutment and several piers.

5. Only a very slight increase in shear stress is believed to have occurred due to the added fill and raised water. The creep movements should continue at about the same rate indefinitely.

6. In addition to creep movements of a large mass, more rapid secondary shallower sliding is occurring in the approach fill. These slides are moving at an increasing rate and are related to pre-existing topography and pre-existing localized sliding. Surface water infiltration may also be contributing to these slides.

7. Further geotechnical studies are needed to better determine the properties of the shales involved in sliding, the groundwater region, and other information to allow quantitative analyses of the creep movements.

The problems at Forest City illustrates the need for good geologic input in the siting process. Fortunately in the twenty years that have elasped since the construction of the Forest City Bridge, most highway departments have recognized this need and now includes soils engineers and engineering geologists in the siting process.

The Forest City slide activity also illustrates a very narrow balance of equilibrium in nature. Often we observe natural slopes, and seeing them apparently stable, assume they have adequate engineering stability with safety factors significantly above 1.0. Many natural slopes, particularly where very rapid erosional processes have governed the formation, actually exist at an overall factor of safety of about 1.0 and are marginally stable, often exhibiting creep. Using the natural slopes as a guide to selecting design cut slopes cut lead to landslide problems as only very slight alterations in existing natural stress patterns can be enough to start significant movement.

ACKNOWLEDGEMENTS

We wish to thank Vernon Bump and his staff in the South Dakota Department of Transportation Foundation and Geology Section for their assistance and cooperation during our studies and for their permission to publish this paper.

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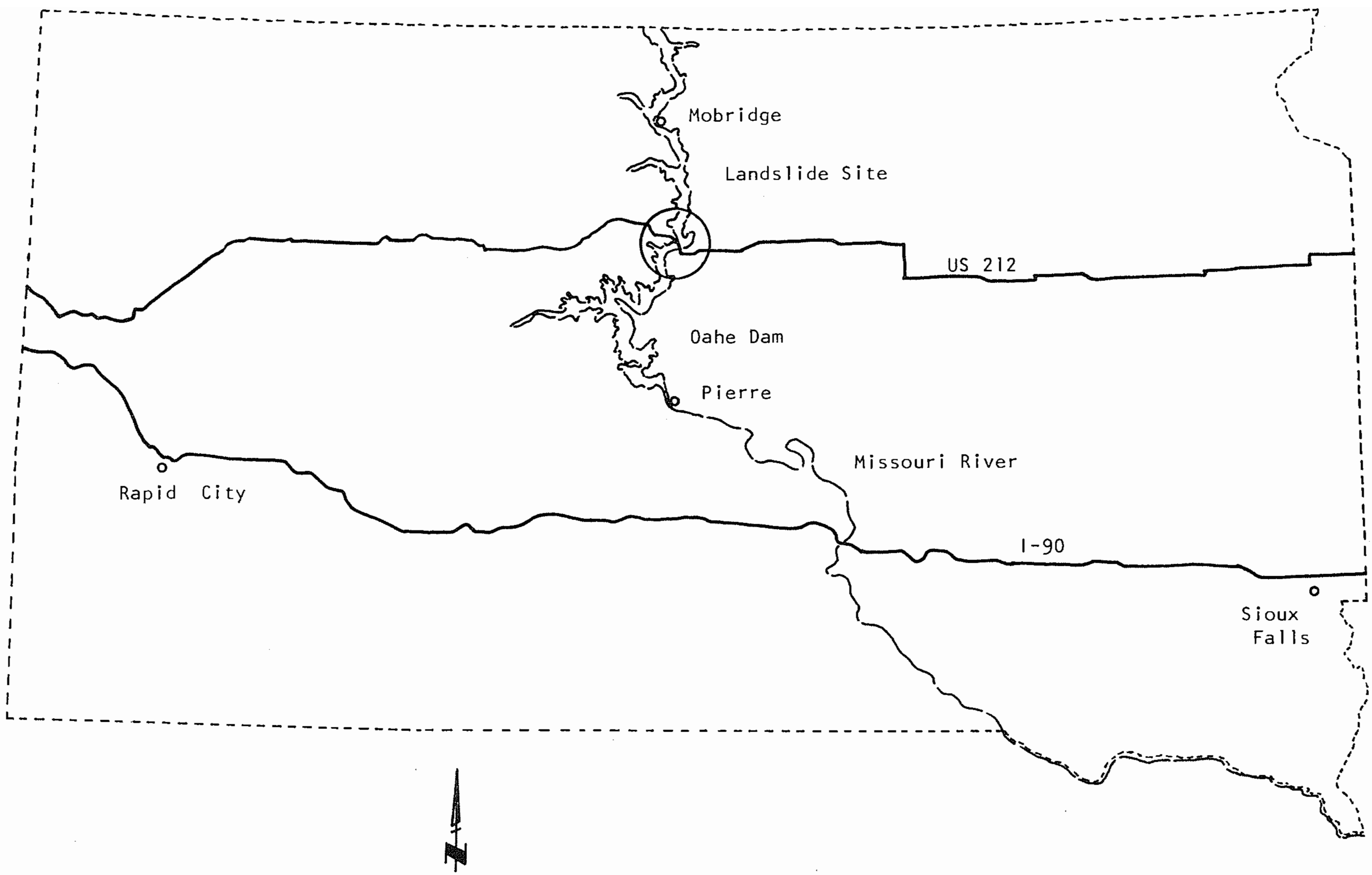

Fig. 1: Forest City, South Dakota Landslide Site Location Map

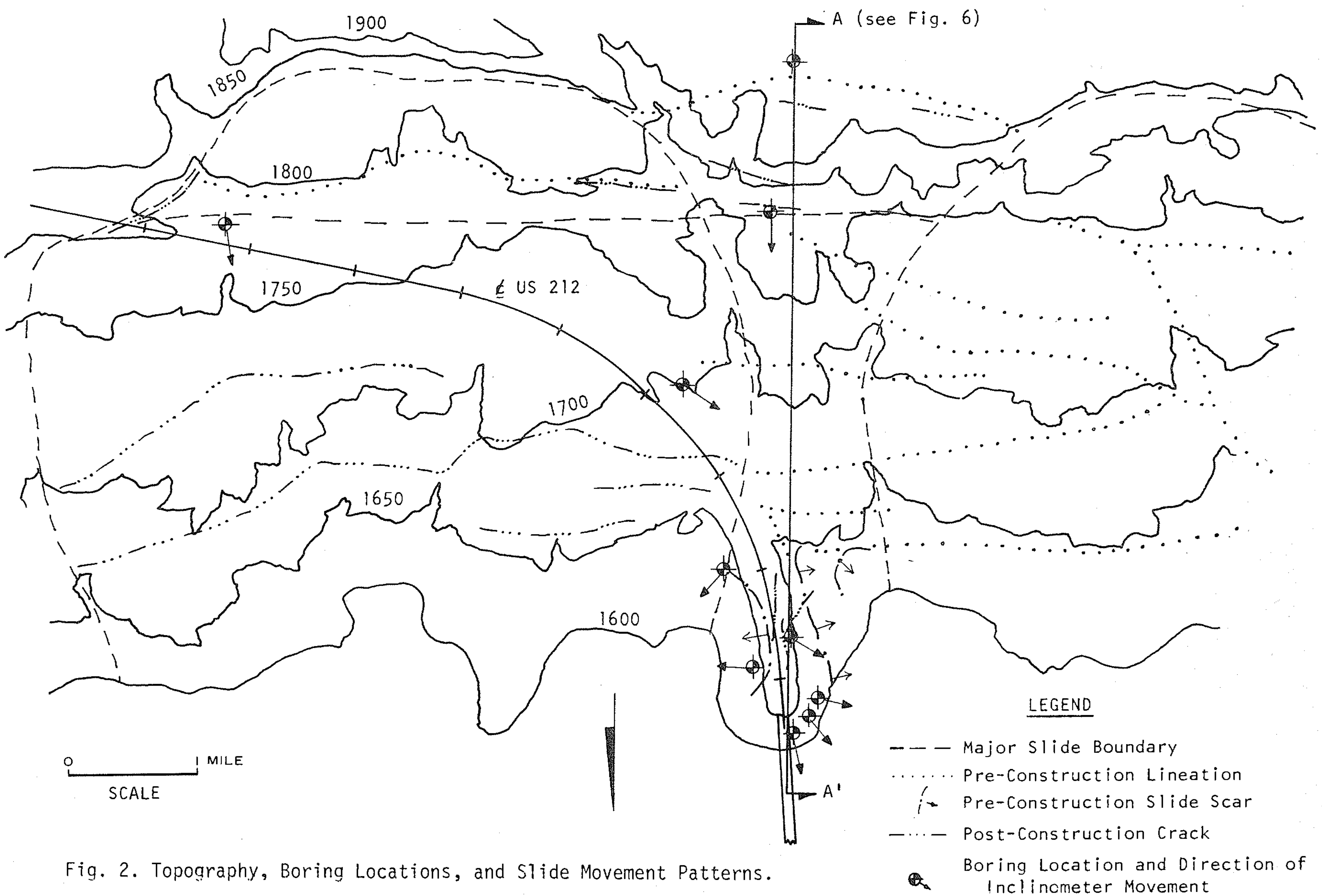

Fig. 2. Topography, Boring Locations, and Slide Movement Patterns.

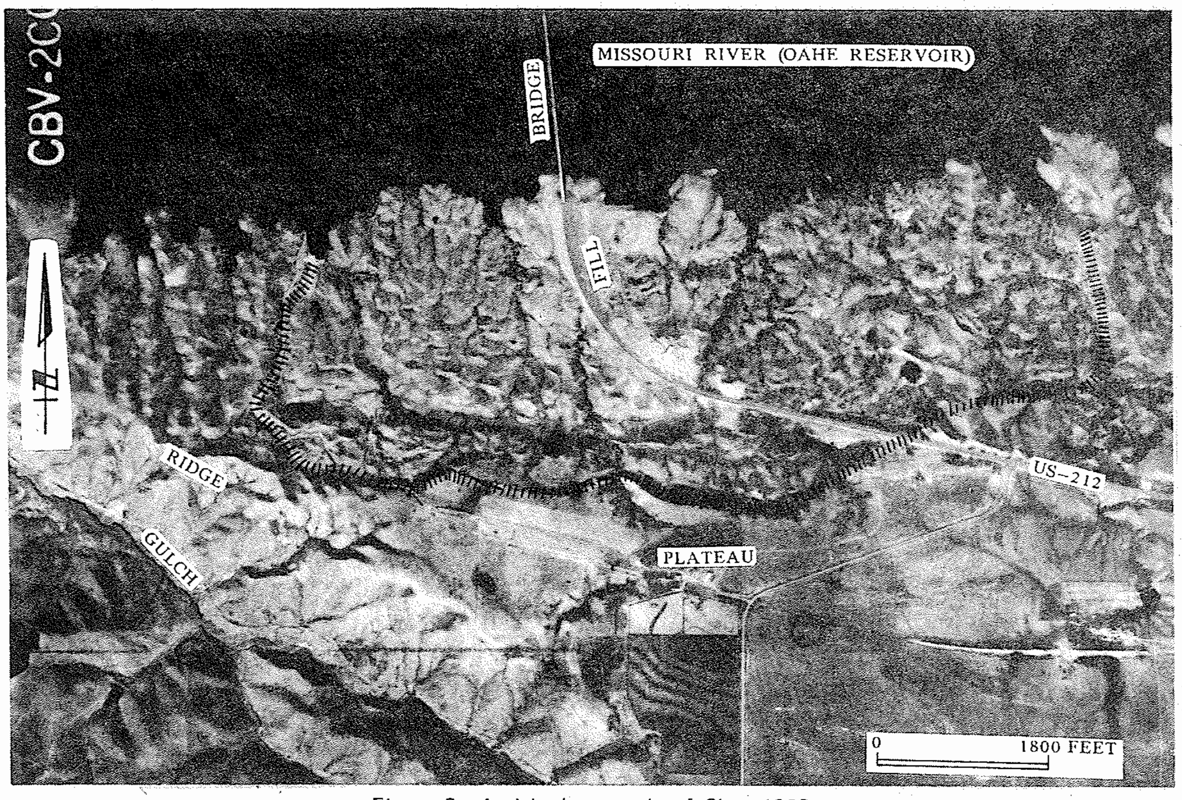

Figure 3. Aerial photograph of Site, 1962.

Figure 4. Oblique photograph of back scarp area, looking south, 1962.

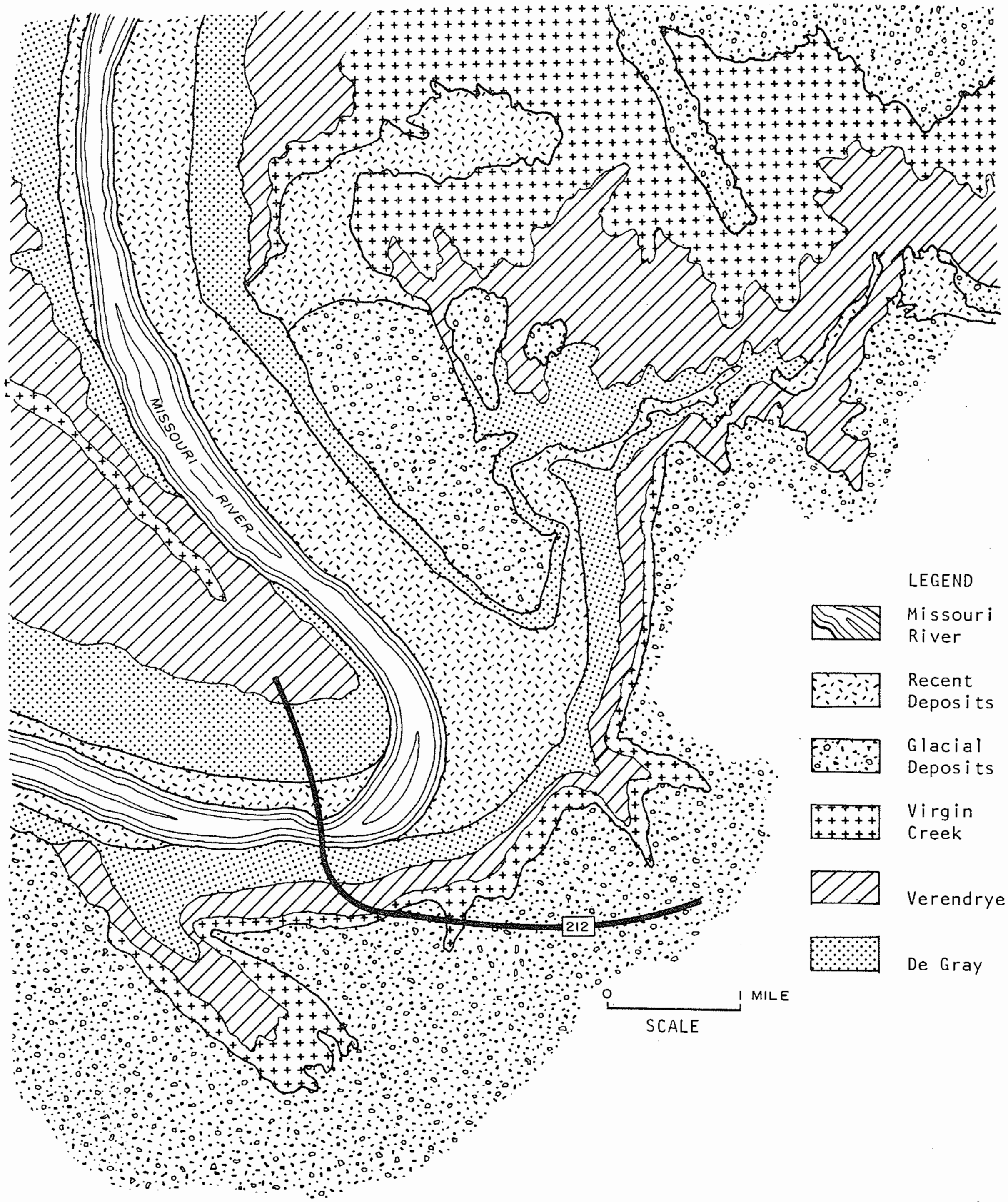

Fig. 5. Generalized Geologic Map

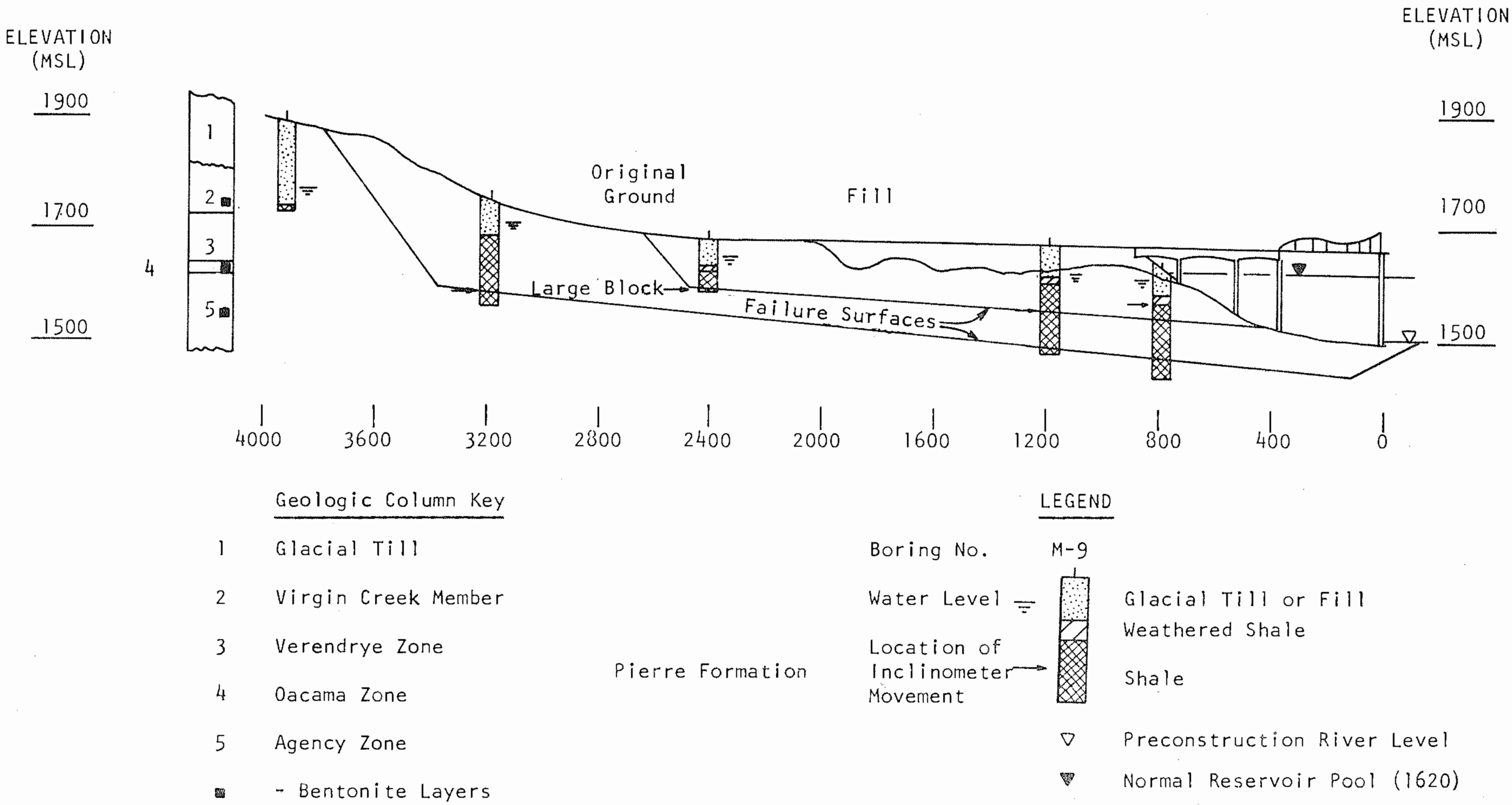

Fig. 6. Generalized Subsurface Profile and Analysis Section.

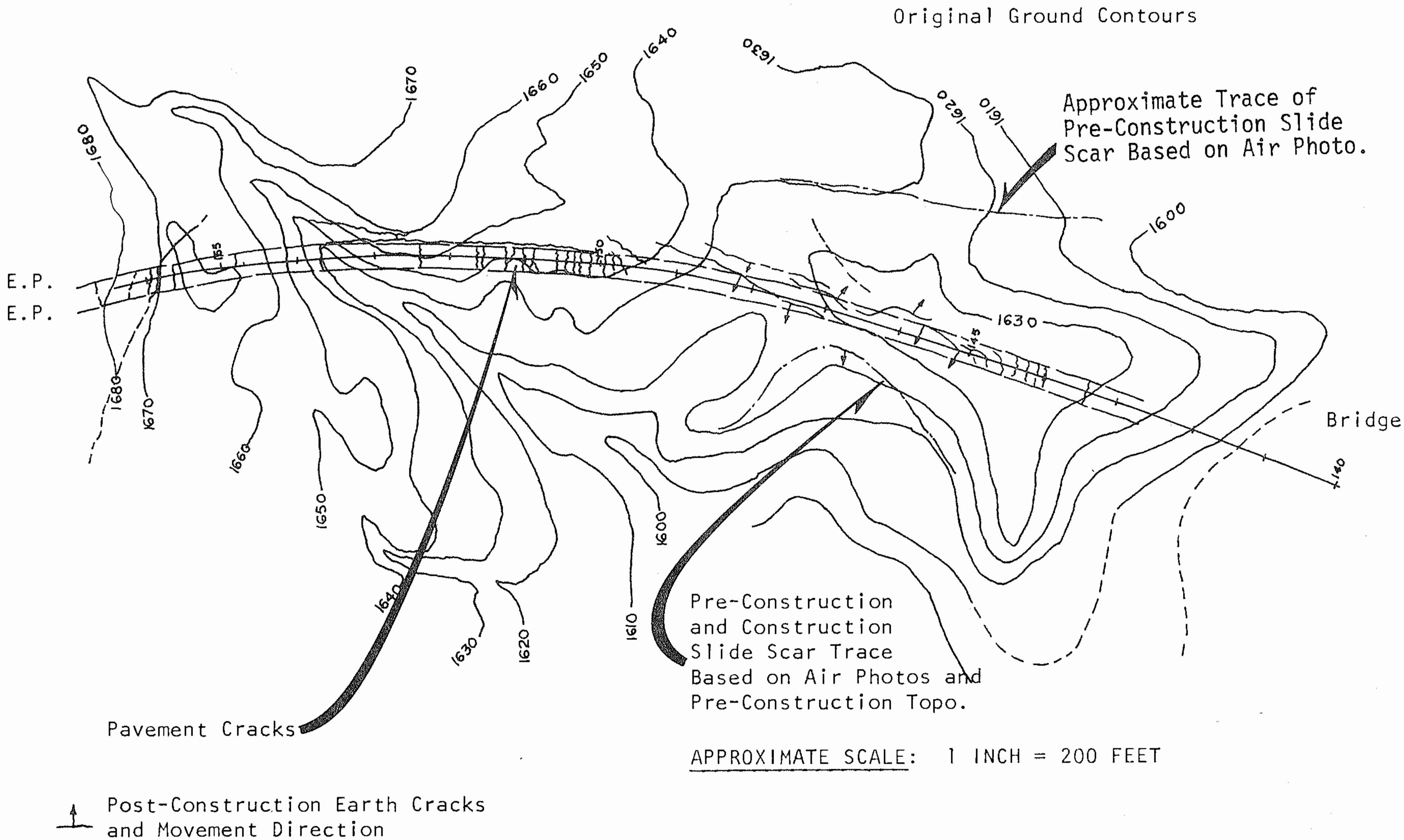

Fig. 7. Approach Fill Original Topography and Crack Patterns.

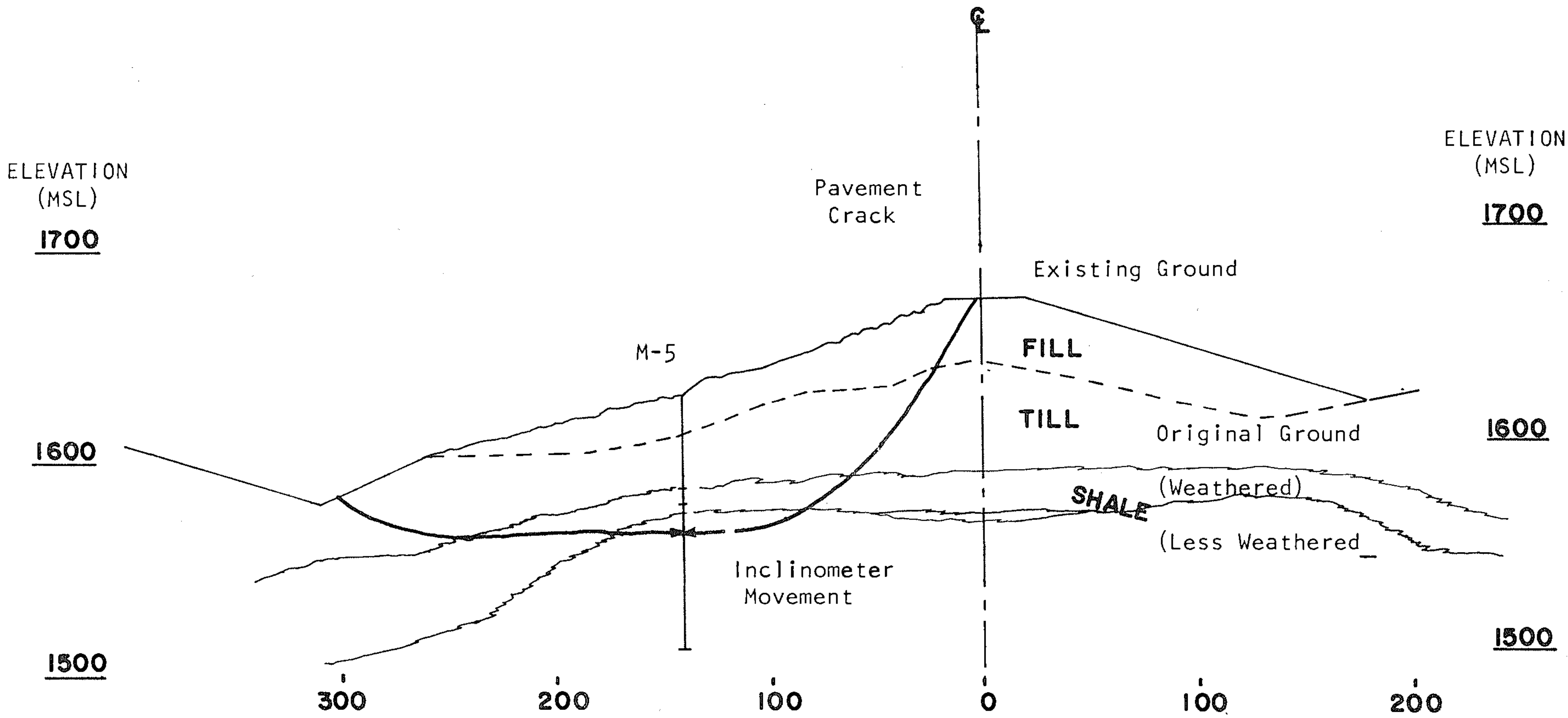

Fig. 8. Section Through Approach Fill Showing General Stratification and Assumed Failure Plane.

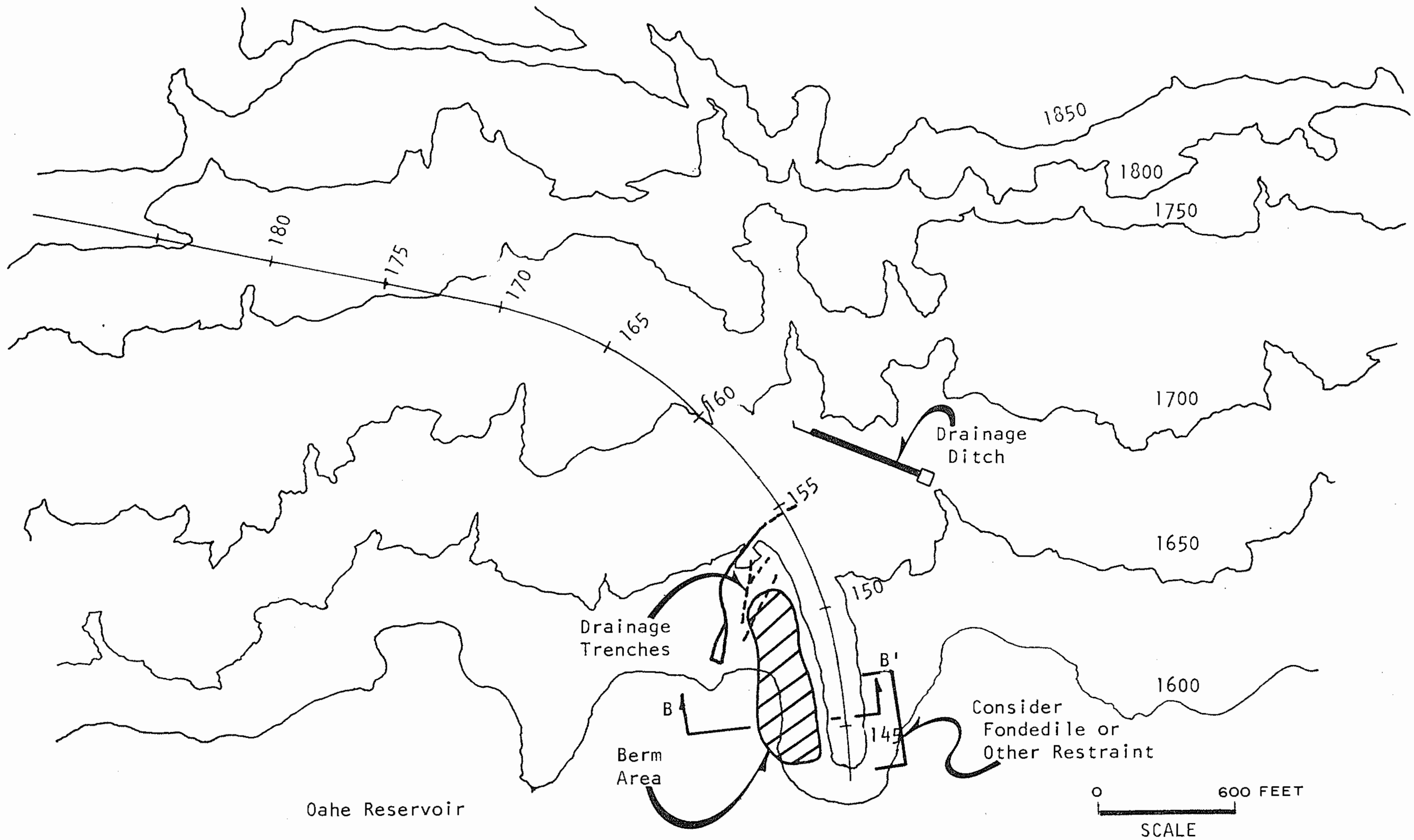

Fig. 9. Plan of Possible Remedial Measures

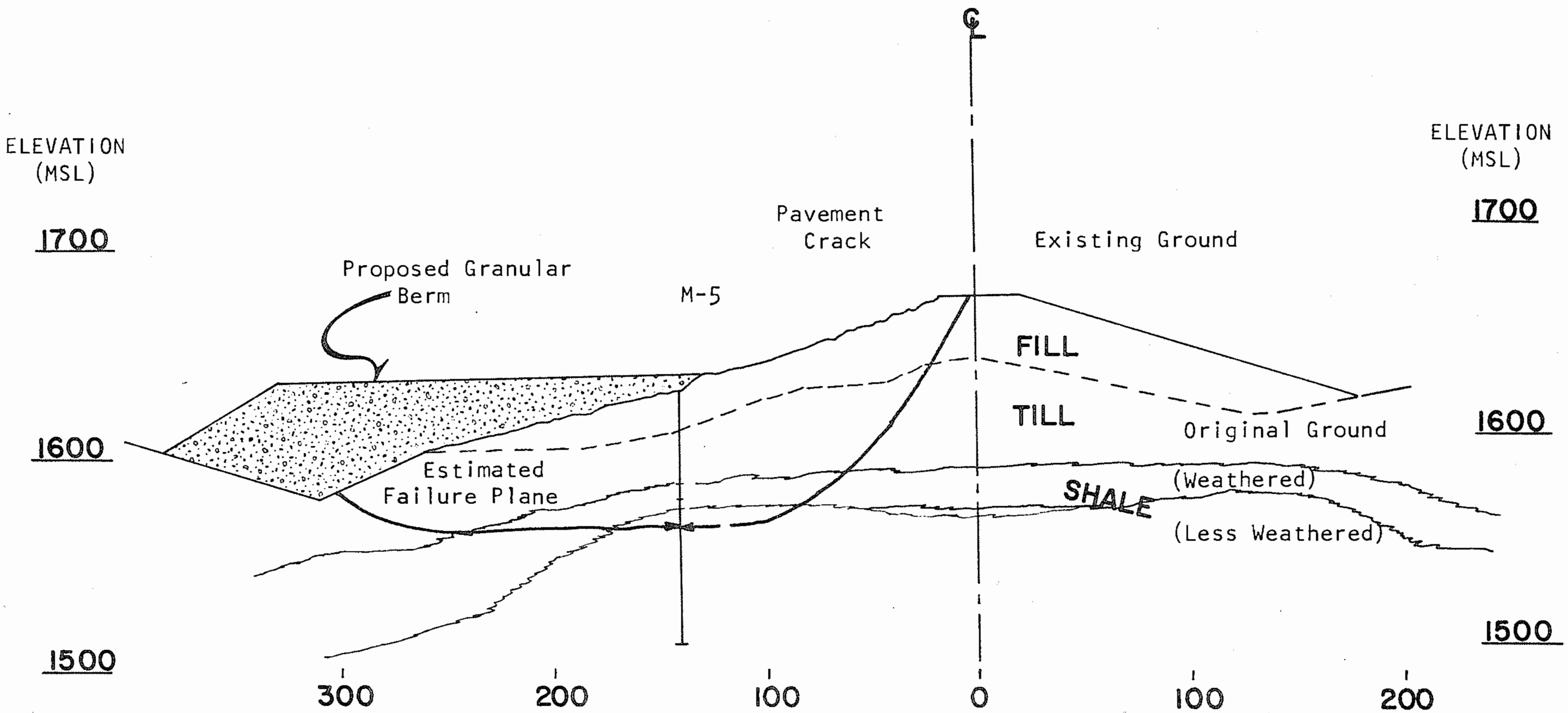

Fig. 10. Section Through Approach Fill Showing Proposed Berm.

TABLE 1
FORMATIONS EXPOSED IN THE
MISSOURI VALLEY, SOUTH DAKOTA
(PETSCH, 1946)

Quaternary			Pleistocene formations			Alluvium
						Loess
						Glacial Drift
Tertiary			Arikaree formation			
			White River formation			
CRETACEOUS	MONTANA GROUP		Fox Hills formation	Timber Lake member		
				Trail City member		
			Pierre formation	Elk Butte member		
				Mobridge member		Chalk
				Virgin Creek member	Upper zone	
					Lower zone	
				Sully member	Verendrye zone	
					Oacoma zone	Upper Mica. Bent.
						Big Bent. Bed
						Lower Mica. Bent.
					Agency zone	
					Crow Creek zone	Chalk
						Sandstone
				Gregory member		
				Sharon Springs member		Oil Shale
	Colo. Gp.		Niobrara formation	Smokey Hill member		
				Fort Hayes member		
		Benton	Carlile formation			
			Greenhorn formation			
			Graneros formation			
			Dakota formation			

TABLE 2
PIERRE SHALE EQUIVALENTS
IN
MISSOURI RIVER BASIN (PETSCH, 1946)

SE. WYOMING[1]	NE. MONTANA[2]	S. ALBERTA[3]	S. DAKOTA[4]	NEBRASKA[5]	KANSAS[6]
	Bearpaw	Bearpaw	Elk Butte		
Lewis			Mobridge		
				Undiffer-entiated	
	Judith River				
Mesaverde			Virgin Creek		Salt Grass
		Belly River			
					Lake Creek
			Sully		
					Weskan
	Claggett				
			Gregory	Gregory	Up. Sharon Springs
Steel					
	Eagle		Sharon Springs	Sharon Springs	Sharon Springs

1. W. H. Holmes, W. Cross and A. C. Spencer, Committee on Geological Names, U.S.G.S.
2. J. B. Hatcher and T. W. Stanton.
3. G. M. Dawson.
4. South Dakota Geological Survey.
5. G. E. Condra and E. C. Reed, The Geological Section of Nebraska.
6. M. K. Elias, Geology of Wallace County, Kansas.

Table 3
Soil Properties for Analysis

Soil Type	Wet Unit Weight, pcf	Effective Cohesion, psf	Effective Friction Angle, degrees
Fill	130	500	20
Till	135	500	15
Weathered Shale	135	100	12
Sheared Shale	135	0	7

HORIZONTAL DRAINS AS AN AID TO SLOPE STABILITY ON I-26 POLK COUNTY, NORTH CAROLINA

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The Interstate 26 project in Polk County, North Carolina was a source of grave concern for several years before its completion in late 1976. This project was scheduled to open in the early 1970's, but massive instability problems during construction forced its closing until the situation was evaluated and a solution to the problems could be found. Completion of the study, planning and design work took about four years from the time construction was halted until it was resumed.

The purpose of this paper is to review the massive instability problems and the remedial measures applied to their control, including horizontal drainage. The horizontal drainage on this project was quite extensive, possibly the largest single contract for horizontal drains in the United States; therefore, a great deal of interest was expressed in its progress. The installation of the drains will be discussed and an evaluation given of their success from information available to date.

SITE DESCRIPTION AND GEOLOGY

The I-26 project is located in Polk County, one of the southwestern counties in North Carolina. The area is about 40 miles south of Asheville, North Carolina and about 25 miles north of Greenville, South Carolina. The entire project was originally 12 miles in length, starting at the Eastern Continental Divide and running generally east, down the Blue Ridge Escarpment into Columbus, North Carolina, near the South Carolina state line. From there I-26 eventually leads to Charleston, South Carolina.

The major landslide problems occurred within a 1.2-mile segment from the crest of the Blue Ridge at Howard Gap, down the escarpment. As with the rest of the Blue Ridge, the ridges trend northeast-southwest; and the project passes obliquely down one of these, Miller Mountain, with a change in elevation of 500 feet. In this area the escarpment is some few miles southeast of the actual continental divide due to stream piracy of the Green River.

The project is located in a wide band of metamorphic and igneous rock mapped as the Mill Springs Gneiss. The unit consists of biotite granite gneiss, hornblende gneiss and mixtures of the two. This rock is highly jointed and generally trends northeast, dipping to the southeast.

Soils in the problem area are deep deposits of weathered and wet, clayey, sandy silt and silty, sandy colluvium, which contains jumbled rock fragments from gravel to boulder size. Under the colluvium micaceous, sandy, silty saprolites and highly weathered rock extend to great depths. Evidence of ancient mass movement can be seen, both on the ground surface and in subsurface samples.

HISTORY OF THE PROJECT

Construction on this segment of I-26 was started in the mid-1960's, and grading proceeded from the continental divide to Howard Gap without incident. Sliding began in November of 1968 as cuts down the Blue Ridge Escarpment were started. Although the contractor attempted to remove the slide material, the failures continued and enlarged.

By September of 1969 the problem was overwhelming. In addition to the cut slides, a large waste area showed signs of instability; and siltation below the project was extensive. At this point, the contractor was instructed to stop work, shape up the drainage and apply erosion controls while an investigation was made towards finding some solutions to the landslide problems.

The results of studies done by a consultant and Department of Transportation geologists during the spring and summer of 1970 indicated that a more detailed investigation was needed. In the summer of 1971 a second consultant started an extensive program of drilling, sampling, mapping and stability analyses. This included the installation of piezometers and slope inclinometers to monitor groundwater levels and earth movements. Their report and recommendations were submitted in the spring of 1974.

The slides were now more closely defined into seven major areas with different characteristics and problems. Included also was the large waste area and slides on an adjacent Y-line. The final recommendations incorporated a redesigned alignment and grade through the effected areas, which allowed completion of the route without triggering additional sliding or construction of expensive structures to retain the slide masses. The design grade was raised and the roadway shifted away from the mountainside to avoid additional excavation and to leave space between the roadway and the toe of sliding material. Remedial measures applied to each individual slide area depended on its characteristics. Some cut slopes were benched and flattened, while others were restrained through the use of a retaining structure or counterweight. The natural ground under large fill sections was benched and covered with a blanket of sand and stone in order to drain water from below and out of the embankment material. To drain subsurface water from the mountainside, 401 horizontal drains were planned for a total of 117,000 feet of drilling.

INSTALLATION OF HORIZONTAL DRAINS

The basic unit is a hydraulically run drill on a crawler tractor. Mounted horizontally alongside the track is a long carriage, pivoted at the mid-point, allowing it to be set at the desired gradient. A large hydraulic motor, rotating the drill rods, is propelled along rails on the carriage to advance and withdraw the drill string. Drill rods are ten-foot lengths of three-inch pipe, joined by coarse, tapered threads. A water pump mounted on the tractor supplies the drilling water (Figures 1 and 2).

The hole is started by moving the rig into position and setting the gradient. A tri-cone bit is fitted to the end of the first length of drill rod and the rod threaded to the hydraulic motor. The motor is then brought forward, advancing the drill rod into the slope. As each length is fully advanced. the joint is broken by reversing the motor's rotation and additional rods are added. To remove the drill rods, these operations are performed in reverse. This system is quite efficient, and little time is lost adding or removing drill steel.

During the drilling, periodic checks are made to determine the elevation at the end of the hole. The drill rods are pumped full of water which is allowed to run back into a plastic hose attached to the rear of the rig. The hose is moved upslope; and when the water in it levels at the same elevation as the end of the hole, that elevation is recorded.

At the desired depth into the slope, the slotted plastic pipe which serves as the drain is inserted inside the drill string before it is withdrawn. The bit is removed from its socket-like adaptor (Figure 3) on the end of the drill string by reversing the direction of rotation. It is then pushed off the end of the drill string with the plastic drain pipe and left in the hole. The drain is held in place as the drill rods are withdrawn. After the rods are removed, the space around the plastic drain as it projects from the slope is packed with a quickswelling clay to form a tight seal.

The tri-cone bits are quite satisfactory in saprolites and weathered rock, but quickly wear out in rock. To penetrate fresh, hard rock more efficiently, the driller used an air hammer which could be adapted to the end of his drill rod in place of a tri-cone bit (Figure 4). With little modification to the rig, compressed air is fed through the drill rods to operate the hammer deep within the hole. Using this system, he was about to penetrate hard rock with little difficulty.

The majority of drains used on I-26 are one- and one-half-inch plastic pipe, cut with two rows of slots, 42 slots per inch (Figure 5). Slot openings are 0.020 inch, which was considered to be the most suitable for the existing soil and water conditions. Although a few drains with 0.050-inch slots were installed, little difference in their performance and those with the smaller openings was noticed. The ends are machined to slip together, leaving the outside diameter flush. Joints are glued and riveted together. Some two-inch steel drains were used, but were found to be difficult to handle and more time-consuming to install.

To keep drain water out of sliding material and off erodible slopes, a system to collect this water and divert it away is necessary. One side of a tee connector is connected to the mouth of each drain. The other side is fitted with a removable cap which permits inspection or cleaning (Figure 6). A plastic line runs from the shank of the tee to an eight-inch galvanized pipe, anchored on a bench under the drains. The eight-inch line is run to the nearest point at which water can be safely discharged.

As a series of drains are completed and the collector assembled, a numbered plate is attached to each drain. A record is kept for each drain, including its location, the completed depth and percent grade, its starting and ending elevation and, of course, the amount of water it recovers.

SLIDE AREAS

Instability on the project was of such a large magnitude that it was necessary to divide the slides into specific areas and deal with them individually. Slide Areas 1 through 6 progressed numerically from Howard Gap down the Blue Ridge Escarpment, a distance of about 1.2 miles. Slide Area 7 and other slides occurred on a Y-line descending the escarpment, generally parallel to the interstate. The large waste area was located between these two alignments and endangered both (Figure 7).

The overall remedial concept was to avoid further significant cutting through the use of a revised grade and alignment; however, additional measures, including horizontal drainage, were necessary in each slide area, depending on its characteristics. When conditions prevented the use of proposed solutions or when unforeseen problems occurred during construction, it became necessary to revise and modify the plans to correct the situation. A brief outline of the problems encountered in each slide area and the remedial measures applied to them is given in order to evaluate the success in controlling movement throughout the entire problem area.

RAINFALL AND EROSION

Before reviewing the slide areas, it should be mentioned that problems created by unexpectedly heavy rainfall and the resulting erosion had to be contended with throughout the construction period. Rain gauges on the project in 1975 recorded rainfall in excess of 87 inches, 25 inches over the yearly normal of 62 inches.

Table 1

I-26

1975 Rainfall in Inches

January	5.23
February	4.42
March	13.95
April	1.25
May	7.68
June	9.95
July	7.30
August	2.00
September	16.40
October	8.40
November	4.32
December	6.05
TOTAL	87.06

Much of this came as heavy rain over short periods of time when drainage structures were incomplete and large areas of cut and fill slopes were unprotected. As a result, some slopes were badly eroded, which in turn, created a serious siltation problem below the project. Movement renewed or enlarged in some of the then uncontrolled slide areas, adding to the already serious problems.

SLIDE AREA 1

Instability was first apparent in late November of 1968. In one month, the failure progressed 600 to 625 feet upslope from the centerline, assuming a generally rectangular shape about 400 feet wide. Slope inclinometers installed during the subsurface investigation recorded movements 20 to 30 feet deep in material consisting of colluvium and residual soils, including some saprolites which showed evidence of ancient displacement.

To stabilize Slide Area 1, all slide material closest to the roadway was removed to provide an interceptor bench in the event of further movement. Slopes were flattened and benched to catch material and provide access for horizontal drilling. As the slopes were flattened, horizontal drains were added to the planned system as the need for them was seen. Some completed slopes were further flattened to gain borrow quantities.

A slide did break out in the upper part of the cut during its excavation. However, the failure was shallow, occurring along thin, richly micaceous foliation planes. The slide eventually fell back to sound material and presented no more problems.

SLIDE AREA 2

Movement in Slide Area 2 first started in March of 1969 when the ground began to pull away from rock cliffs high above the roadway. A tension crack, 30 to 40 feet deep and nearly 1,000 feet long, developed at the base of the cliffs, 450 feet right of the centerline. Slope inclinometers within the slide mass showed movement to depths of 41, 44, 64 and 79 feet.

To control this slide, the construction of a counterweight or reinforced earth wall on the berm was considered. Upon investigation it was found that neither of these proposals were feasible, so the majority of the unstable material was removed back to the rock cliffs. Some difficulty was experienced where removal had to end and a dirt slope started against the rock face. This slope gave trouble throughout its construction and probably owes its present stability to horizontal drainage. In the event of continued failure, ample space is provided at the toe to isolate slide material from the roadway.

SLIDE AREA 3

Slide Area 3 consisted of widespread colluvial deposits, 15 to 45 feet thick, containing perched groundwater. Initial failure was caused by undercutting these deposits, which were only marginally stable before construction started. Stability analyses showed that failure would probably continue upslope if left alone, and massive failures would occur if further excavation was attempted.

It was recommended that remedial measures be applied which would require no additional excavation of the slope. This was to be accomplished by using the revised grade and alignment, diverting surface runoff, utilizing horizontal drains to remove subsurface water, providing interceptor benches for slide debris and finally, if these measures were unsuccessful, to be prepared to construct extensive retaining structures to contain the slide mass.

Further study showed that drainage alone would not reduce the rate of movement to an acceptable level, and there was no adequate foundation for retaining structures. At this point it was decided to begin cutting high on the mountainside on a slope which generally conformed to the slope on rock cliffs above. Although staked on 3/4:1, this slope was expected to fall back to a stable rock plane. After this decision was reached it became a matter of simple, but large-scale excavation, extending at one point, nearly 1,000 feet right of the centerline. When a satisfactory plan was reached, a wide interceptor bench was left and a much flatter slope extended on to grade. Thus, of all alternatives considered in Slide Area 3, slope flattening proved to be the most effective (Figure 8).

SLIDE AREA 4

This area was similar to Slide Area 3 in that it consisted of wet, colluvial deposits and dislocated saprolite. By May of 1969, sliding occurred throughout nearly the entire 1,500-foot length of the area, and additional excavation would surely enlarge the slide upslope. It was the consultant's opinion that two slide mechanisms were operating in Slide Area 4, both triggered by loss of toe support. The first was the presence and strength-reduction effects of water in the colluvial deposits. The second was a gravity-driven wedge of colluvium and dense saprolite blocks moving towards the roadway on high-angle joints and schistose mica zones. As these two mechanisms operated together, remedial measures had to be compatible to both and adaptable to transition from one to the other.

No additional cuts or slope flattening was desirable, so remedial measures included the revised grade and alignment to provide a wide interceptor bench at grade. Removal of subsurface water by horizontal drainage was started in colluvial deposits within the area. In one 900-foot section, two levels of drains were completed for a total of 6,795 feet of horizontal drilling. To the present, the maximum amount of water ever to run from these drains has been about five gallons per minute. As the drilling was in progress, vertical borings showed the drains to be well above the water table and it was realized the majority of drains would likely be dry. However, this was in the early stages of the drilling program and during the dry summer months, so it was decided to complete all the drains in the event the water table did rise during rainy periods. This has not happened since their completion in late 1974, and information from slope inclinometers shows that the slope continues to creep at a very slow rate.

All the drains in Slide Area 4 were not unsuccessful; it was an area where drainage was altered in the field to meet the existing subsurface conditions. Through the use of inclinometers and piezometers installed as horizontal drilling was in progress, drains could be added, relocated or adjusted for maximum efficiency.

The remedial concept applied to the large sliding mass of colluvium and saprolite was not an attempt to halt downhill movement, but to isolate it from the roadway. Although no additional cutting into this mass was desirable, it was necessary to construct working space for horizontal drilling and access into other areas. The effects of this minor construction, combined with heavy rainfall, renewed sliding, shearing horizontal drains and slope inclinometers. Movement along the back scarp averaged 0.3 to 0.4 feet daily, eventually leaving a scarp about 20 feet high. The slide continued to creep, but at an acceptable rate, so no attempts to stop it have been made.

In an effort to divert surface runoff in Slide Area 4, open ditches were constructed uphill from the slides which consisted of overlapping sections of half-round, galvanized metal culvert pipe. It was felt that the metal culvert would be sufficiently flexible to withstand some movement and still function. Unfortunately, these ditches became filled with leaves and forest litter, causing them to overflow and become undermined. The pipe then sagged and pulled apart, diverting a concentrated flow of surface water directly into the slides. Use of half-pipe elsewhere on the project produced similar results, so it was abandoned.

The use of horizontal drainage to stabilize Slide Area 4 was generally unsuccessful. The majority of drains in colluvial deposits were nonproductive. However, those drains that were successful did lower groundwater significantly and are certainly contributing to the stability of parts of Area 4. Movement has not been entirely halted, but it has been slowed to a tolerable and acceptable degree.

SLIDE AREA 5

The original design in Slide Area 5 was a pre-split rock cut, some 50 feet in height above the roadway, with a 30-foot-wide bench on top of rock. The slopes above the bench were to be cut on 1½:1.

Sliding started in April of 1969 when the cut was within ten feet of bench grade. The major crack was 500 feet right of the centerline, well outside the original cut limts. Slope inclinometers in the slide mass showed movements to 43 feet in depth. The moving material was described as bouldery colluvium and displaced saprolite, with finer parts of the colluvium having a moisture content at or above the liquid limit. Numerous springs and wet areas were present in the mass, and groundwater varied from the surface to 20 feet. The failure was attributed to high groundwater levels and a slope cut too steep for the material's strength.

Recommendations for this area were to attempt no significant slope flattening, but to leave the bench in rock to catch any further debris and to install four levels of drains across the slide--one at the highest scarp, one at bench grade and two in between.

Horizontal drilling was started on the highest bench in the spring of 1974. Existing boreholes behind the slide were checked carefully to see what effect this work would have on groundwater levels. The drains on this bench were successfully installed by early summer, and groundwater was lowered about five feet.

Moving lower in the slide mass presented some difficulties in constructing drill benches, as the material was too wet to support the necessary heavy equipment. If saturated soil conditions halted construction within a few feet of the planned bench elevation, it was possible to adjust the gradient of the drains to achieve the planned back-of-hole elevation. On occasion it was necessary to allow the material to dry for several weeks before construction could resume.

After moving to the third level bench, the first few weeks of drilling produced no significant drop in groundwater, although the drains were producing water. On July 30, groundwater in Hole L-501 was 36 feet below the surface. Five days later it had dropped from 36 feet to 71 feet, a lowering of 35 feet. By mid-August it had lowered to 75 feet and has remained at about that level to the present (Figure 9).

The results of this were seen in the slope between drill benches, which dried significantly. Drains on the upper bench slowed, eventually stopping altogether. As drains were installed in the lower benches across Slide Area 5, the results were much the same. The slide material has dried remarkably, and there have been no signs of further movement, either on the ground or in the slope inclinometers.

Over the entire project the slide in Area 5 should be regarded as the most successful use of horizontal drainage to stabilize a large mass of saturated material. By fall of 1976 the work in Slide Area 5 had been completed for some time, allowing groundwater levels to stabilize. At that time, all the drains in the area were producing at the rate of approximately 55 gallons per minute or nearly 80,000 gallons every 24 hours.

SLIDE AREA 6

The original design in Slide Area 6 called for a 1½:1 slope going into the cut. One-third of the way in, a pre-split rock cut and 30-foot bench leading into Slide Area 5 began. Slopes above the bench were to be cut at 1½:1, with a maximum height of 115 feet.

Sliding was first noticed in early 1969 as the 1½:1 cut approached bench grade. By spring of 1970 cracks were well outside the proposed cut limits, material was in the haul road and had the consistency of a semi-fluid. Slope inclinometers showed movement 22 to 34 feet deep within the slide mass, while one, L-601, behind the major scarp showed none. Groundwater levels ranged from the surface to 13 feet. Again, failure was due to high groundwater and a slope too steep for the material's strength.

After construction was restarted in 1974, nothing was attempted in Area 6, as the material was virtually unworkable. In March of 1975 rainfall totalled 15 inches, with 7.5 inches of that falling within a 48-hour period. As a result, the main body of the slide again began to move. Some break-back along the major scarp occurred and a tension crack extended 120 feet farther upslope. The slope inclinometer, L-601, showed a cumulative deflection of 0.7 inch, even though it was located outside the crack. Groundwater upslope from the slide rose five to ten feet (Figure 10).

To dewater the area behind the slide and to prevent further enlargement in that direction, horizontal drilling was started at the toe of the highest scarp. The number of drains, their depth, location and direction were determined in the field, depending on results as drilling progressed.

By mid-summer of 1975 the work seemed to be producing satisfactory results. Groundwater had lowered 10 to 25 feet, and no movement in the slope inclinometer was detected. A collector system was installed to divert drain water from slide material. By late summer of the same year the slide material seemed dry enough to attempt slope flattening and the construction of drill benches lower in elevation.

Unfortunately, this activity, combined with 16 inches of rain in September, renewed sliding. The first indications of movement appeared as the collector system began to pull away from the drains. By late fall, several drains were sheared and the collector completely wrecked, adding water from the drains to the already saturated slide mass. The slide extended laterally to some extent, and the material was wet enough to make walking difficult (Figure 11).

Fortunately, there was no movement upslope or enlargement in that direction. The inclinometer showed none, nor could any be detected on the ground. Even though some drains were sheared, they were still functioning, stabilizing the slopes above the slide.

Again, it was agreed to dewater the slide material with horizontal drains. Due to the condition of the material, it was impossible to get into the area with any kind of heavy equipment, so drains were installed in the slide from stable ground along its perimeter. As the slide material dried somewhat, a drill bench was started into the slide itself. When the bench began to cave or the backslope to slump, grading stopped and drilling began. In this way, drains were eventually installed across the main body of the slide, drying it significantly and slowing the movement. The sheared drains and their collectors were repaired to eliminate that source of water.

The horizontal drains have, by no means, solved all the problems in this area, but they have made an enormous improvement in a bad situation. The largest part of the slide and the slopes above it appear to have stabilized. Some minor slumping is presently occurring in areas which could not be reached with the drill rig, but these could be controlled with proper drainage. Slide Area 6 in no way resembles its planned configuration. As events occurred, they were dealt with in the field in the best way possible.

SLIDE AREA 7

Sliding in Area 7 was initiated by the construction of cuts on a Y-line near the crest of the Blue Ridge. The upper limit of the slide was 400 feet left of the centerline and within 150 feet of an existing road above. Equally important was the location of a high-voltage steel transmission tower, 50 feet behind the slide scarp.

To contain this slide mass, the Y-line was relocated and a counterweight constructed on the abandoned Y-line roadway (Figure 12). Horizontal drains were installed in the toe of the counterweight and into natural ground above it, though they produced little water. It was recommended that the steel transmission tower be moved to another location, but this was never done. Slope inclinometers installed at the base of the tower and in the counterweight have detected no movement since the work was completed.

Y-12 SLIDE

Y-12 is a two-lane facility which generally parallels the interstate down the Blue Ridge Escarpment, with a grade sometimes exceeding 15 percent. A 200-foot cut section in colluvium and saprolite along this roadway had been unstable since it was first excavated. Sliding usually coincided with periods of heavy rainfall and on occasion, nearly blocked the narrow roadway. Efforts to clear the road resulted in further loss of toe support, and the slide migrated upslope. The upper scarp reached to within three feet of a high-voltage transmission pole which had to be relocated.

To drain the slide material, debris was cleared from the toe and the roadway excavated to allow installation of horizontal drains below grade. When the drains were completed, they were run to a collector which was buried as the roadway was brought back to its original elevation. The major source of water in the material came from a spring above the slide. Water from this spring was diverted and to provide additional drainage, horizontal drains installed around it.

Dewatering of the slide did not entirely halt movement, so a restraining structure was necessary. At the toe, steel H pilings were driven on four-foot centers and reinforced with batter piles. Timbers placed between the flanges of the pilings created an effective and relatively inexpensive retaining wall (Figure 13). Since the completion of the wall, the slide has caused no more trouble.

WASTE AREA

During the first phase of construction in the late 1960's, the contractor wasted approximately 400,000 cubic yards of excess roadway and slide material in an area between the main line and a Y-line below. The area was cleared, provided with drainage features and material end-dumped with no particular compactive effort being made.

Stability analyses made in 1972 gave safety factors ranging from 0.91 to 0.97, indicating the mass was on the verge of failure. Assuming groundwater rose ten feet above the base of the waste, the figures fell to as low as 0.64. No water was detected in the material during investigations made in 1970 and 1972; however, borings done later showed that groundwater was indeed starting to rise. As the revised alignment extended well into the unstable area, it was necessary to remove the waste down to sound material and to rebuild an embankment to support the roadway.

During removal of the waste, the source of groundwater appeared in the form of bold springs and wet zones. These springs showed no signs of diminishing for as long as the excavation stayed open. The water sometimes impeded grading operations. It was hoped that much of the waste material could be used elsewhere, but the majority of it was unsuitable and had to be wasted a second time.

To rebuild the necessary roadway embankment benches were constructed in the backslope of the excavation as waste was removed. The configuration of the benches was variable, dictated by the suitability and moisture content of the material they were constructed in.

To provide drainage under the embankment, a one-foot thickness of underdrain sand was spread over all benches and slopes, followed by one foot of No. 11 or No. 13 stone. After perforated underdrain was laid along the benches and connected into a master drainage system, benches, slopes and underdrain were covered with an additional foot of stone (Figures 14 and 15); and construction of the actual embankment began. The material around extremely wet areas and springs was excavated and, if possible, groundwater was run directly into underdrains. The excavation was filled with sand and stone, tying it in with the rest of the blanket.

After periods of heavy rains, the flow of water from underdrains increased considerably, and water was observed to flow from the sand-and-stone blanket. The underdrains were two feet or more above the base of the embankment, so groundwater had risen by at least that amount.

To intercept surface runoff on the finished fill slope, half-sections of galvanized culvert pipe were installed on a narrow bench across the face of the embankment. The resulting erosion from heavy rain in March of 1975 completely destroyed the system. After its replacement it was again destroyed by rainfall in September. Thereafter, the use of half-pipe was discontinued over all the project, and it was replaced by paved ditches.

To further control surface runoff, underdrain was buried around the contour of the fill slope, just below the surface, in a shallow trench filled with sand. These drains intercepted both water on the surface and that flowing just beneath the surface. The performance of these drains was greatly improved by placing sheets of polyethylene in the bottom and downslope side of the trench to create an impermeable barrier, forcing water into the underdrain. These drains worked so well that they were built along several elevations as the fill was brought up.

OBSERVATIONS AND CONCLUSIONS

On a horizontal drainage project of this magnitude, a thorough preliminary engineering investigation is essential to make the most effective use of the drains. Good subsurface information from cased boreholes, piezometers and slope inclinometers is not only necessary to plan the drilling program, but is valuable later as drilling is in progress. Using this information, the success of the program can be monitored and revisions made in the field as changes in subsurface conditions are seen. Drains can be added and deleted as necessary, or their length and gradient altered to increase productivity.

It was expected that there would be many unsuccessful, or dry, drains. However, it was felt that all planned drains should be installed; and if some were dry, perhaps they would produce water during periods of wet weather. As it turned out, few of them did. Generally, wet drains sometimes dried up, but dry drains usually never began to flow.

Of the successful drains, some proved to be very sensitive to rainfall, which was reflected in their changing rate of flow in periods of wet and dry weather. Other drains pointed out the unpredictable path of subsurface water. A series of drains in one particular area have caused a corresponding drop in the water table as much as a thousand feet away.

Many drains did not reach their planned depth due to refusal of the tri-cone bit in rock. During the later stages of the project, this became less of a problem as the air hammer was used. The unused footage from short holes was not wasted, however, being used to supplement planned drains or in problems that came up during construction.

The design of the collector system was somewhat unsatisfactory, as any sort of movement pulled it apart. Material eroding from slopes above the collector often was enough to push it out of alignment. The plastic drains themselves are flexible enough to withstand a considerable amount of movement; and should one become sheared, the distance back to the break gives a point on the slide plane.

The horizontal drainage program on I-26 has provided a wealth of experience to be used on future projects. The drainage of this project was possibly overdesigned and fewer drains could have accomplished the same result. However, this area was such a problem, it was felt that any measures necessary to stop the sliding were worth the cost. If that cost seems excessive now, it did not during the planning stage.

Has the sliding stopped? Not entirely and probably never well completely. Minor slumping and slides will likely continue for some time, but hopefully no massive failures as in the past will occur. The height of slide-prone slopes has been held to a minimum, benches in the slopes and at road grade are provided for slide material, and horizontal drainage has contributed greatly to the stabilization of the large masses of wet material. In a mile-and-a quarter of slides, any one considered major, future problems may be expected; but the situation is under control and should present no danger to the public.

When the project was completed, 686 horizontal drains had been installed, with a total of 124,546 feet of drilling, about 23.5 miles. The total volume of water produced by the system is difficult to calculate due to variations in the weather; but on the average, it produces about 320 gallons per minute, 19,000 gallons per hour, or nearly half a million gallons every 24 hours.

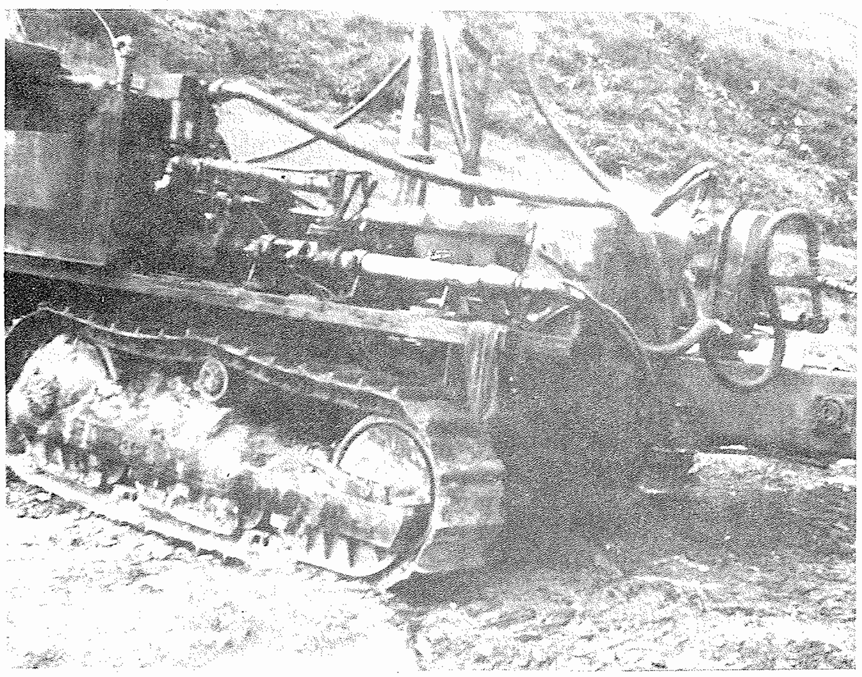


Figure 1. Drill unit mounted on a crawler tractor.

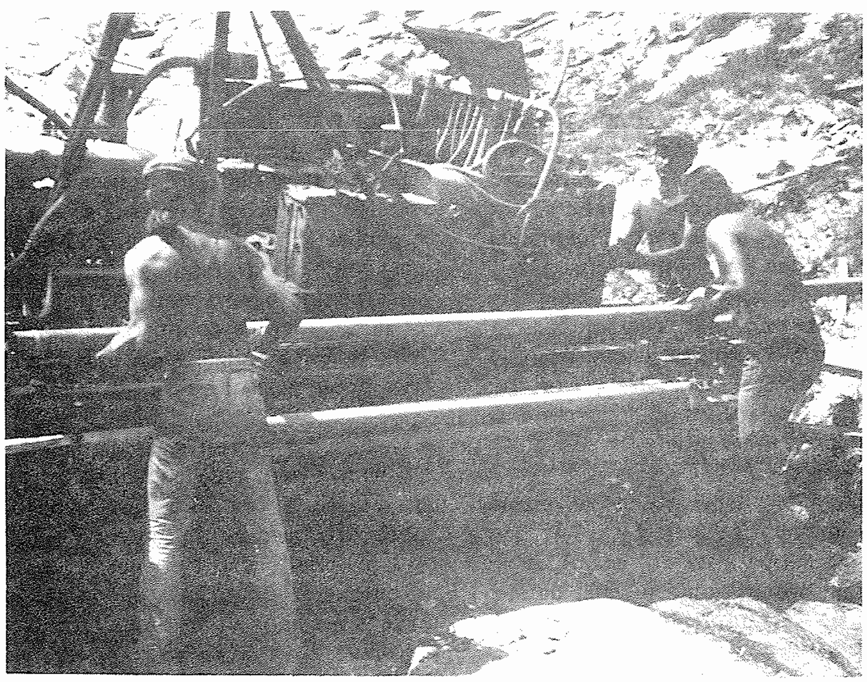


Figure 2. Drill rods being added.

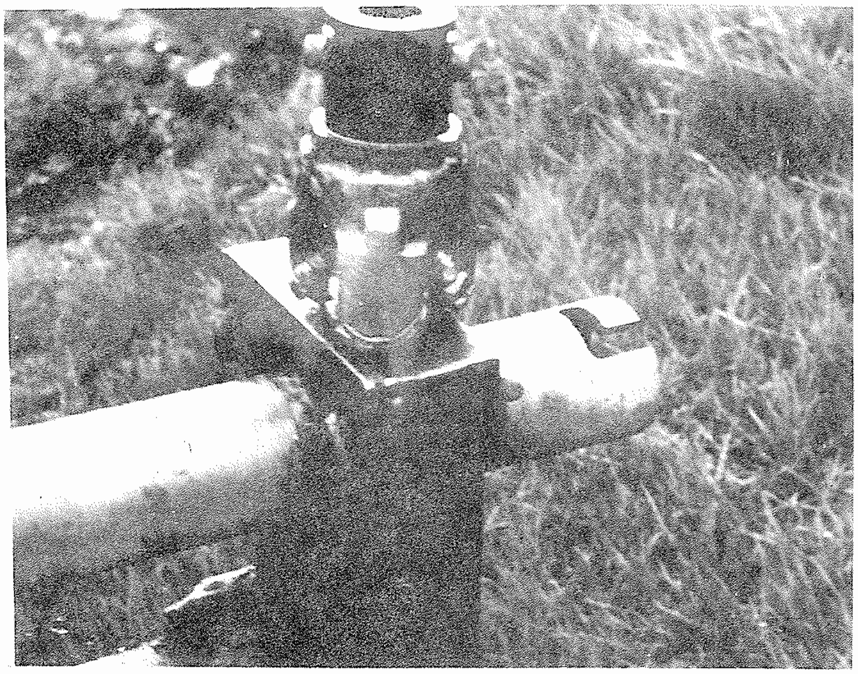


Figure 3. Tri-cone bit and its adaptor.

Figure 4. Air hammer on the end of the drill rod is being positioned to start a hole.

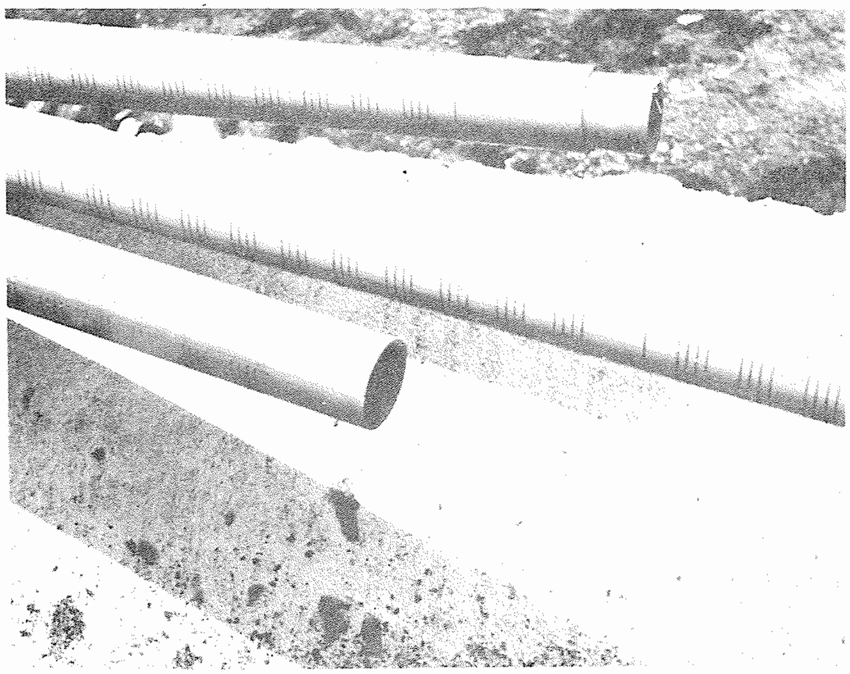


Figure 5. 1½ inch plastic drains. Slot openings are 0.020 inch, 42 slots per inch.

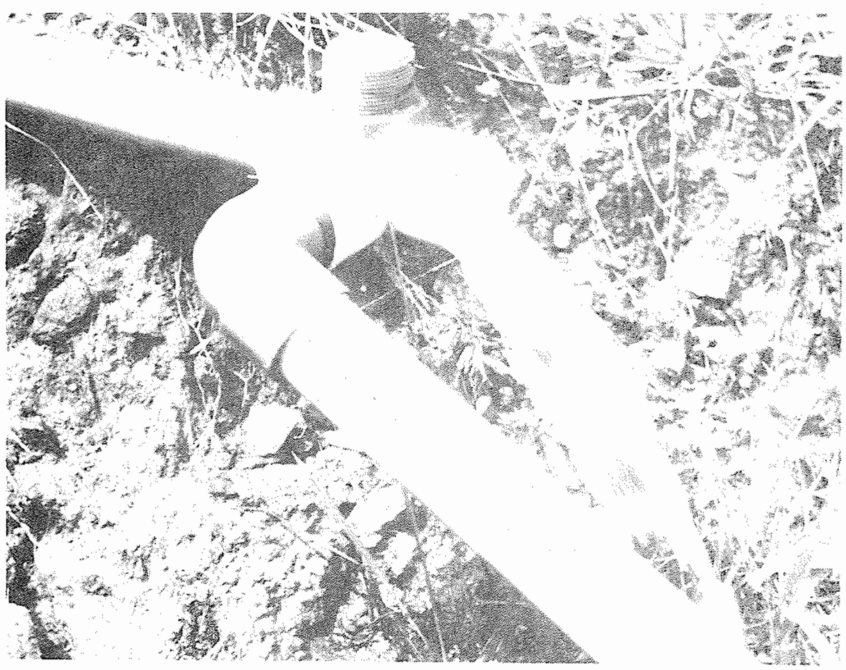
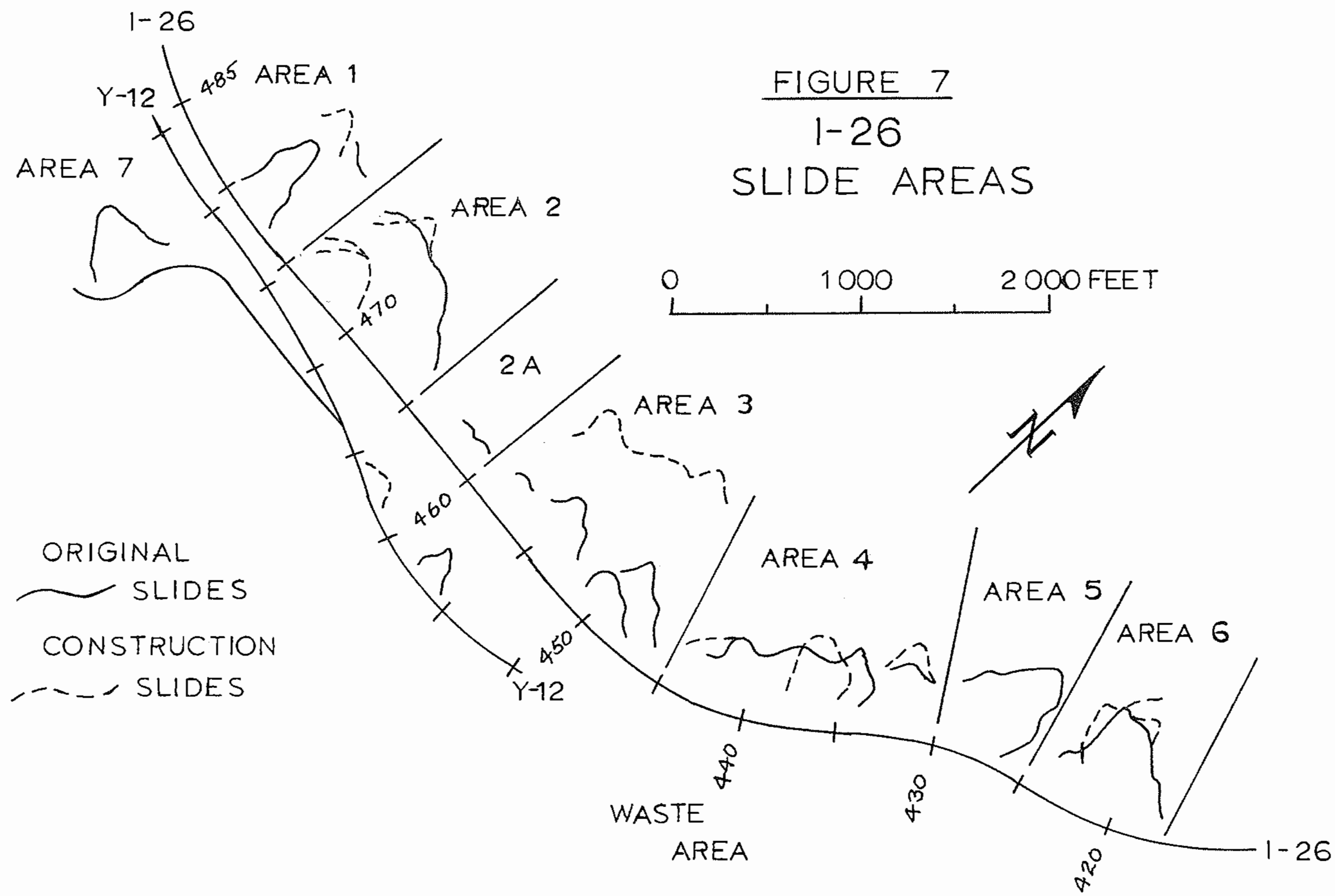


Figure 6. Tee connector on the end of each horizontal drain. Removeable cap permits inspection or cleaning.

FIGURE 7

I-26

SLIDE AREAS



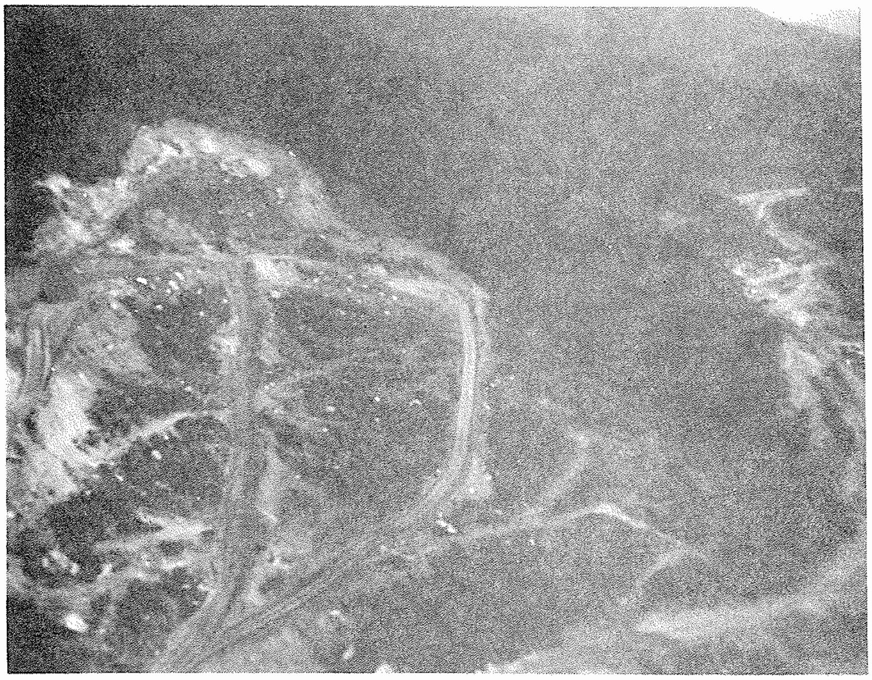
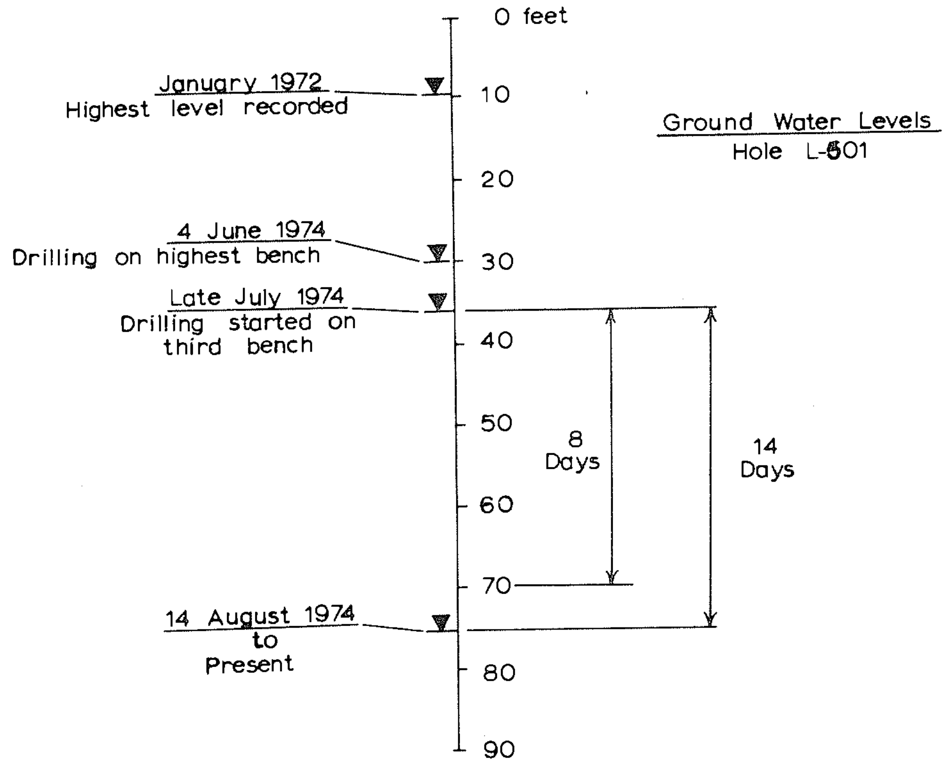


Figure 8. Slide Area 3 as the upper slope is started.
Although staked 3/4:1, it was expected to fall back to a stable plane.

FIGURE 9


Ground Water Levels
Hole L-501
0 feet
10
20
30
40
50
60
70
80
90
January 1972
Highest level recorded
4 June 1974
Drilling on highest bench
Late July 1974
Drilling started on third bench
14 August 1974 to Present
8 Days
14 Days

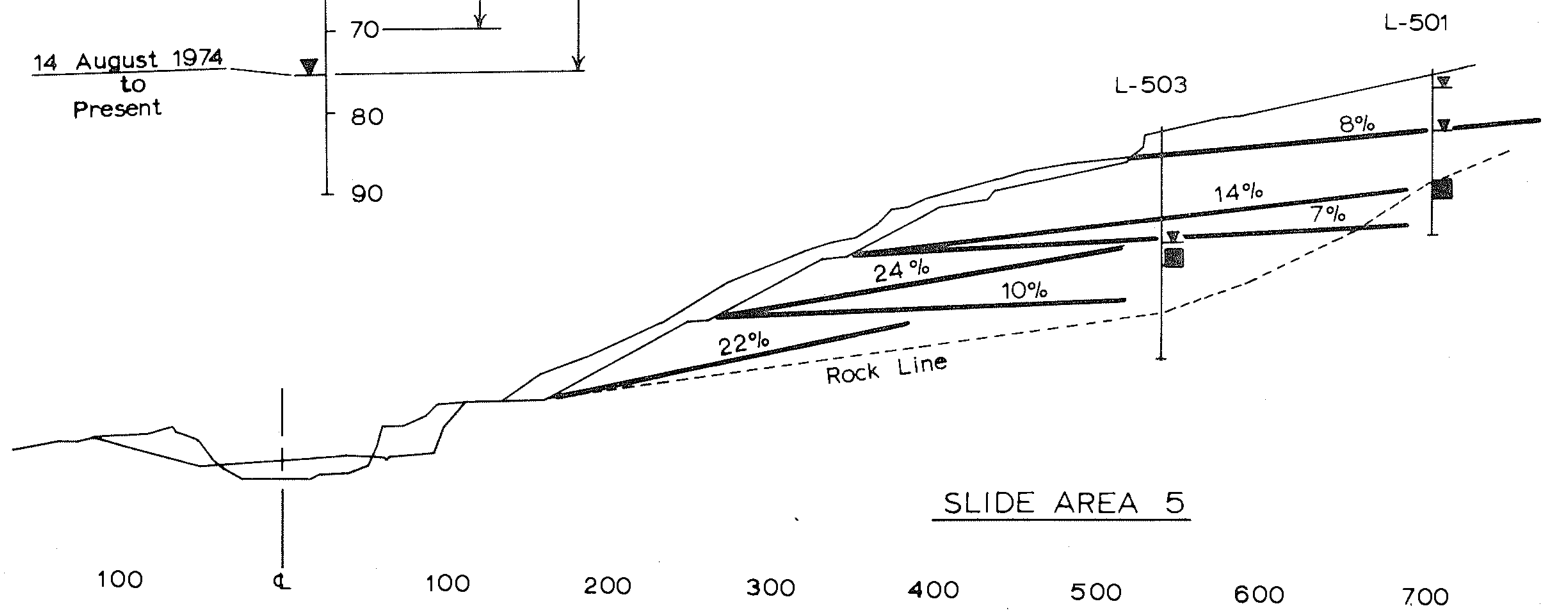

L-501
L-503
8%
14%
7%
24%
10%
22%
Rock Line
SLIDE AREA 5
200
100
℄
100
200
300
400
500
600
700

FIGURE 10

SLIDE AREA 6

SPRING 1975

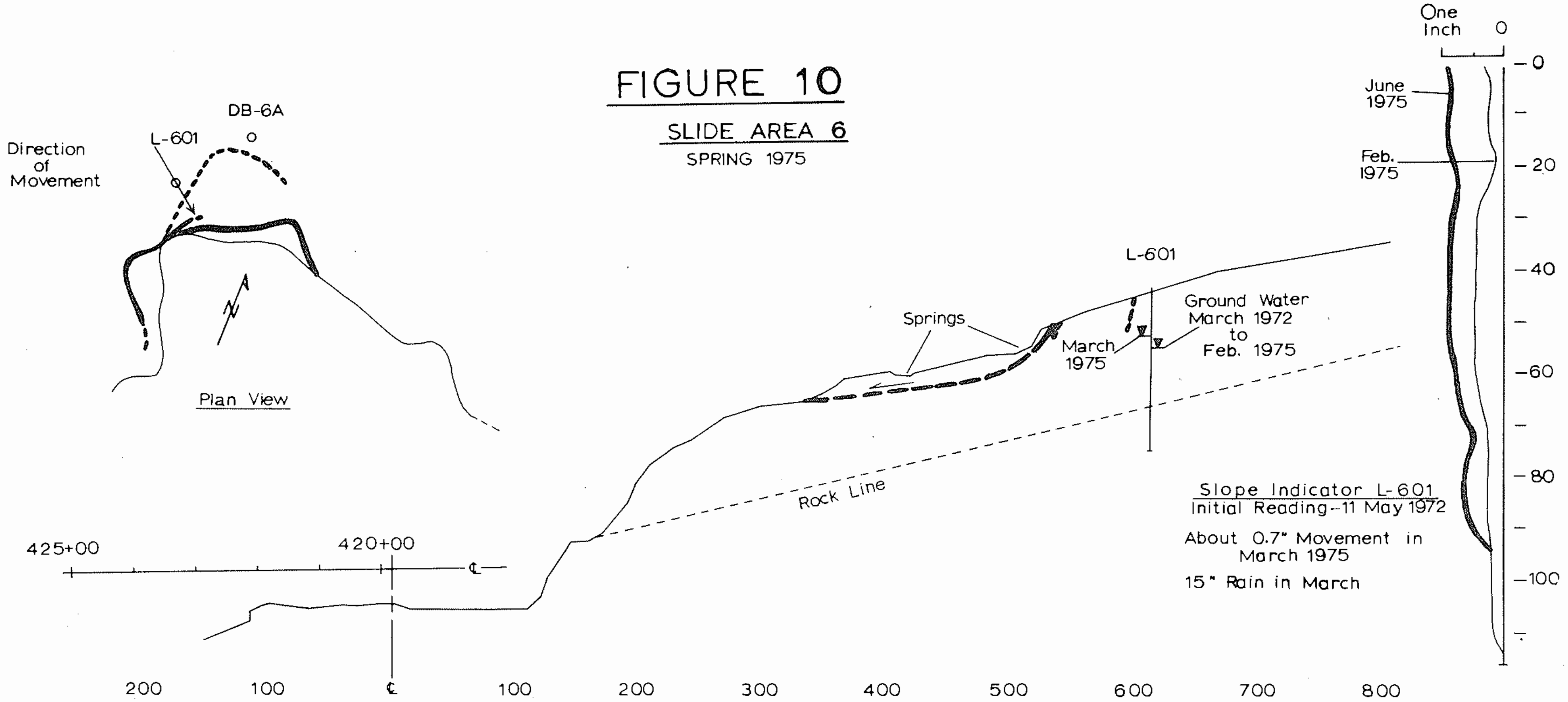


FIGURE 11

SLIDE AREA 6

FALL 1975

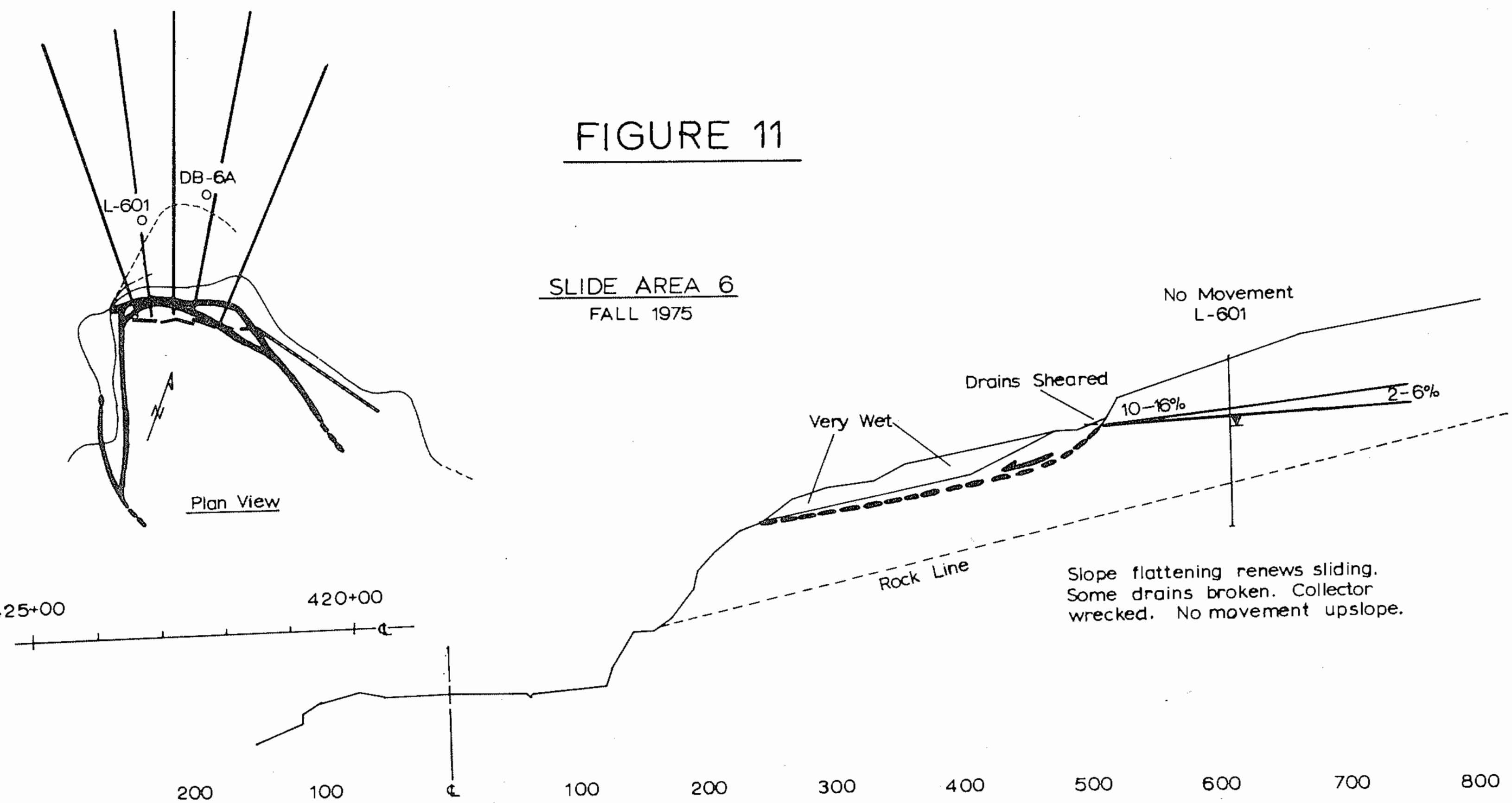


Figure 12. Construction of the counterweight at the toe of Slide Area 7. Tower is about 50 feet behind the slide.

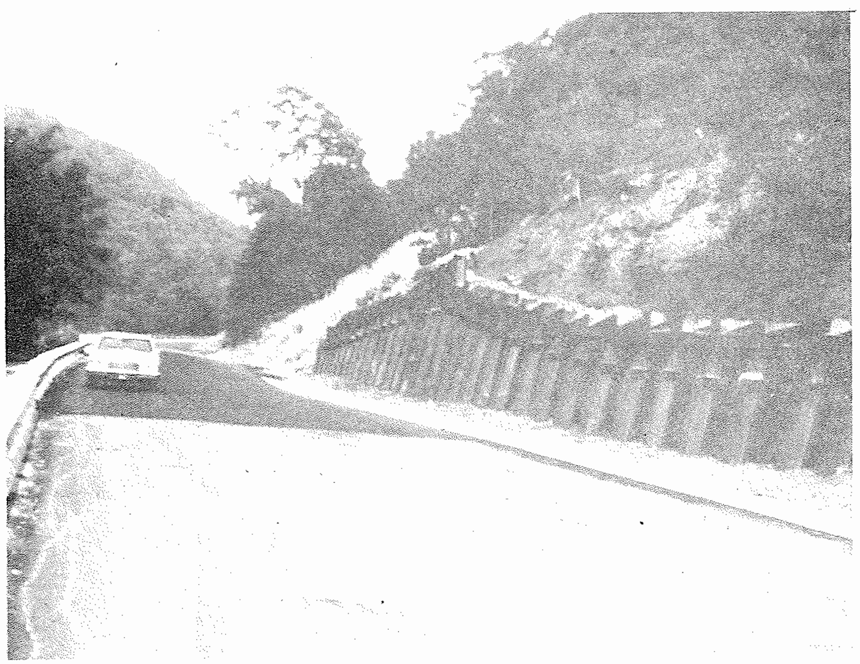


Figure 13. Timber and piling wall restraining a slide on Y-12.

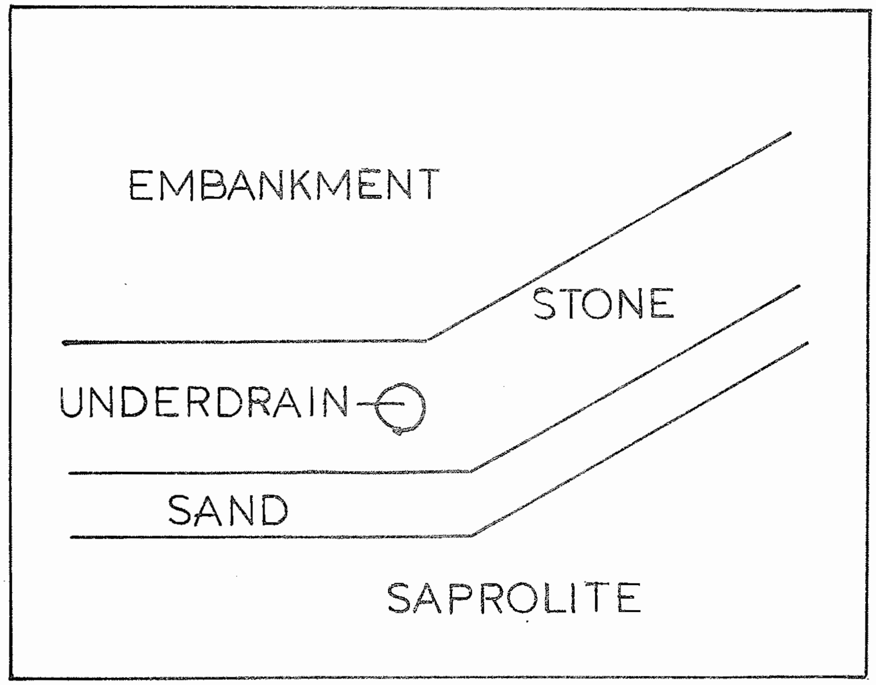


Figure 14. To provide drainage, a layer of sand and stone was placed under large embankments.

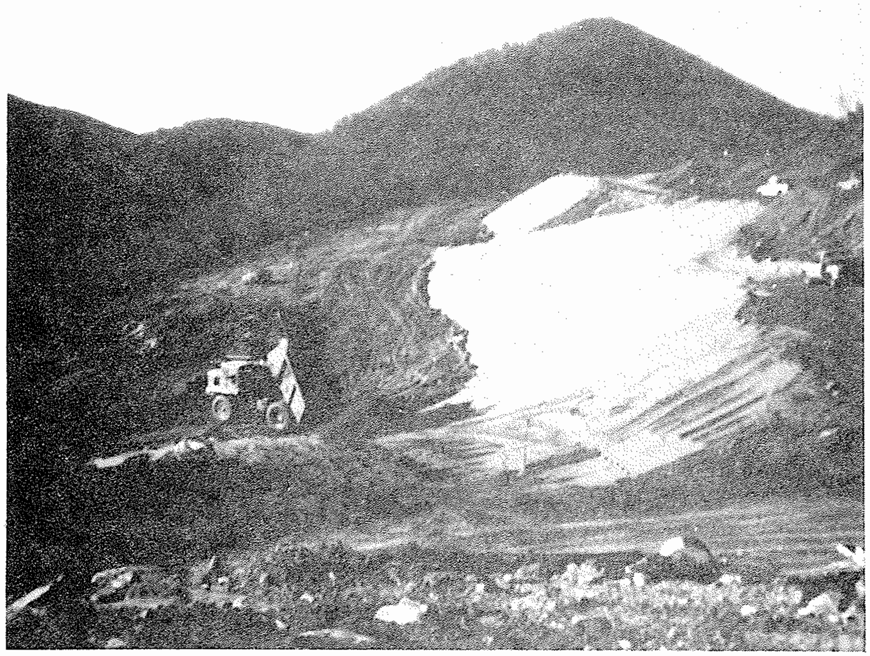


Figure 15. Embankment material being placed over sand and stone blanket. Lighter material is stone; the darker, sand.

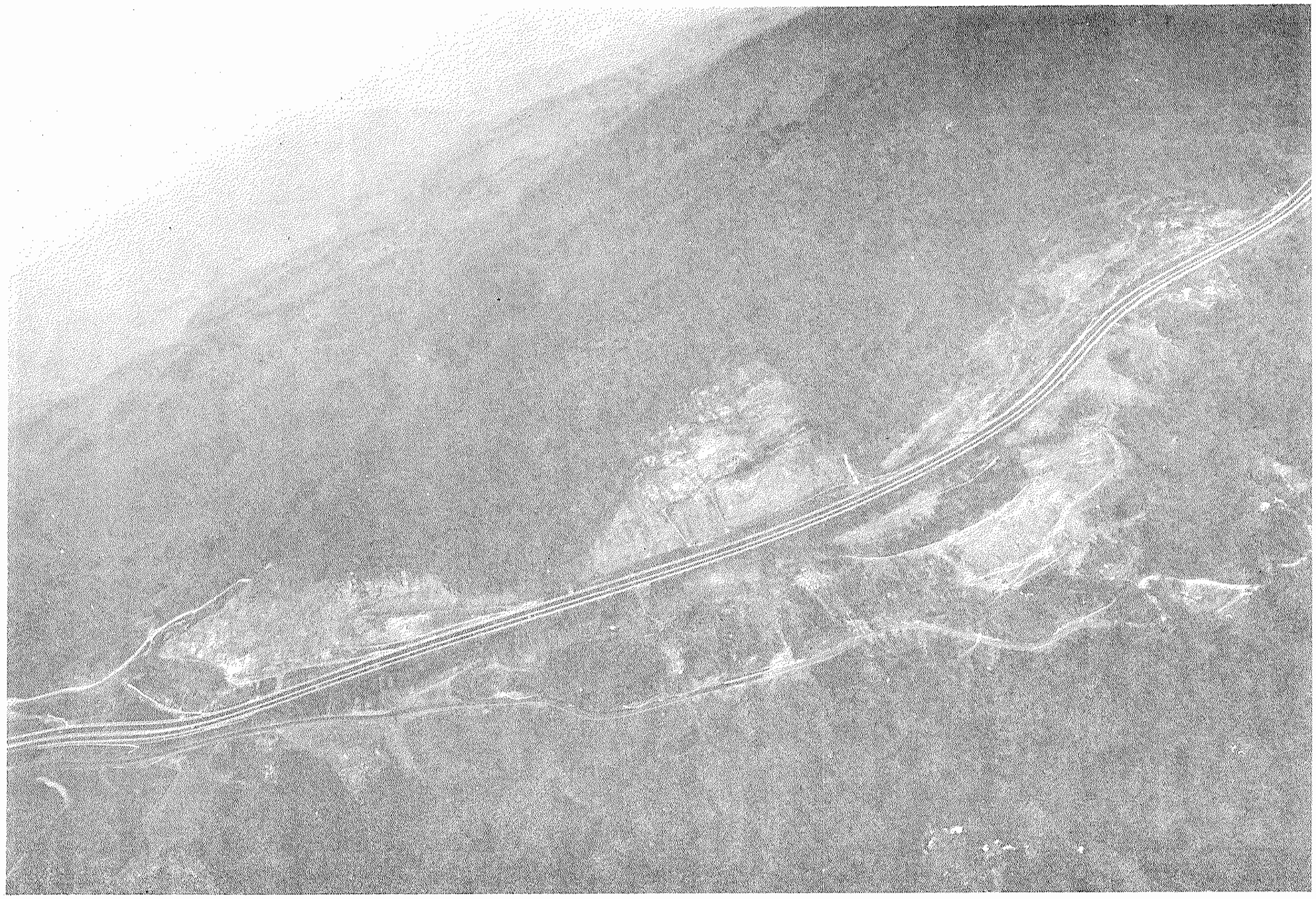


Figure 16. Interstate 26 Slide Area, May, 1977.

SOME OBSERVATIONS ON THE USE OF HORIZONTAL DRAINS IN THE CORRECTION AND PREVENTION OF LANDSLIDES

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ABSTRACT

Subsurface water may act in any number of ways to reduce the stability of cuts and embankments. Among these are decrease in cohesion, subsurface erosion, lateral pressure in fractures and joints, and excess pore water pressure.

One way of removing subsurface water is through the use of horizontal drains, which are holes drilled into an embankment or cut slope and cased with a perforated metal or slotted plastic liner.

The equipment, materials and procedures used in the drilling and installation of horizontal drains have been improved and refined considerably since the California Division of Highways first introduced their "hydrauger" in 1939. The development of PVC pipe, improvements in drill bits and drill stem, and the development of drilling machines capable of producing high thrust and torque has made subsurface drainage a significant and economical alternative in the repair and prevention of some types of landslides.

The Tennessee Department of Transportation has installed more than 46,000 m (150,000') of horizontal drains since 1972 in the stabilization and prevention of landslides involving several different geologic materials and conditions.

INTRODUCTION

Water in all of its forms (rain, fog, snow, ice, etc.) and in all of its occurrences (streams, lakes, oceans, the subsurfaces, etc.) is the single most troublesome and perplexing substance that must be dealt with by transportation engineers. Of all of these occurrences, subsurface water is probably the most perplexing because it is the least predictable, especially as it relates to the stability of cuts and embankments in geologically complex areas.

Subsurface water may act in any number of ways to reduce the stability of cuts and embankments. These include subsurface erosion, lateral pressure in fractures and joints, decrease in cohesion, reduction in moisture tension, viscous drag due to seepage flow, and excess pore water pressure. By far the most common and significant of these is excess pore water pressure, which is also referred to as neutral pressure or neutral stress and is defined as "the stress transmitted through the fluid that fills the voids between particles of a soil or rock mass" (Glossary of Geology, 1972, p. 479). Pore water pressure increases in a cut or embankment when what may be termed as the normal infiltration--migration--discharge balance is upset. This can result rather suddenly during periods of heavy rains when there is high infiltration or percolation or it may develop over longer periods of time due to blockage resulting, for example, from consolidation along the contact or along a zone between the embankment and its underlying foundation. A reduction in stability, oftentimes to the point of failure, frequently accompanies excess increases in pore water pressure.

One way of reducing excess pore water pressure, high seepage forces created by perched water tables, or lowering the normal water table is through the use of horizontal drains. Horizontal drains are holes drilled into an embankment or cut slope and cased with a perforated metal or slotted plastic liner.

While horizontal drains have been used in the stabilization of landslides since about 1939 when California introduced their "hydrauger" (Cedergren, 1967), the method did not begin to gain wide acceptance for use on a large scale by highway engineers, at least in the eastern states, until many years later. The Tennessee Department of Transportation first used horizontal drains in 1972 when a series of embankment failures occurred along Interstate-75 in Campbell County (Royster, 1973). Drains on this project, totaling approximately 18,288 m (60,000'), were used in conjunction with rock buttresses. Some drains extended up to 183 m (600 feet) in length and initially produced flows up to .44 liters/sec (7 gallons/min). Since 1972, the Tennessee D.O.T. has installed horizontal drains totaling more than 46,000 m (150,000 feet).

This paper describes some experiences of the Tennessee Department of Transportation in the use of horizontal drains in the stabilization and prevention of landslides and embankment failures involving various geologic materials and conditions.

HISTORY OF DEVELOPMENT: EQUIPMENT, MATERIALS, AND PROCEDURES

According to Smith and Stafford (1957), the first horizontal drilling rig used in California (the hydrauger) was a rotary drill mounted on a racked frame in such a way that permitted a revolving drill bit to be advanced into the slope with a hand-operated ratchet lever while water was pumped through the drill rod to cool the bit and wash the cuttings from the hole. Five-foot sections of drill rod were added as the drilling proceeded. The first holes were drilled with a 51 mm (2") bit and then reamed to 381 mm (6") prior to casing with a 102 mm (4") perforated metal pipe. It was soon determined that it was more practical to perform the drilling in one operation. This resulted in the development of a 102 mm (4") modified fishtail bit which was improved through the years by hardening with various types of alloys. In 1949 the tricone roller bit became available, and was used in drilling harder materials.

With the improvement of the fishtail bit and the development of the roller bit came still further advances. For example, the 102 mm (4") casing was replaced by 51 mm (2") casing. It consisted of standard black pipe perforated with 10 mm (3/8") diameter holes on 76 mm (3") spacings drilled in 3 rows at the quarter points. The pipe was furnished in 4.8 m to 7.3 m (16- to 24-foot) lengths without threads or couplings. The casing was then butt-welded as it was jacked into the hole. Smith and Stafford (1957) state that the reason for welding the joints rather than using couplings was to hold the perforation rows in alignment and in an up position.

Since the hydrauger was limited to the more cohesive soils and soft rock, the California Division of Highways developed a more powerful and versatile machine. This drill, first used in 1951, was a self-propelled unit, powered by a 60 horsepower gasoline engine and equipped with a hydraulic feed to advance the continuous flight augers which did not require a drilling fluid. In 1953 this machine was further modified so that regular rotary drilling could be accomplished using N-rod and roller bits. Still further improvements were made over the following several years. This involved the design and construction of a machine with such features as a transmission that permitted control of speed of rotation over a wide range; a hydraulic feed with a minimum 1.8 m (6') stroke, and capable of exerting a 17,792 N (4000 lb.) thrust; a chuck that could be easily interchanged to accommodate A-rod, N-rod or casing; and spuds for maintaining alignment.

PRESENT DAY EQUIPMENT, MATERIALS AND PROCEDURES

The equipment, materials and procedures used in the drilling and installation of horizontal drains have changed considerably since the pioneering efforts of the California Division of Highways in the 1950's. The most significant changes have involved the development of plastic PVC (polyvinyl chloride) pipe for use as a liner for the drilled hole; heavy-walled, flush-coupled drill rod; expandable drag and roller bits; and a drilling machine capable of developing extremely high thrust and torque. The most frequently used drilling procedure is depicted schematically in Figures 1 and 1a. It involves the use of an expendable roller or drag bit (Figure 2), which is attached to the drill stem with a slotted adapter. Drill stem is added in 3 m (10') sections as the hole is advanced. Water, pumped at the rate of about 35 gpm is used to cool the bit and flush the cuttings from the hole. Once the required depth is reached, the bit is "knocked off" by reversing the rotating direction of the drill stem. Slotted PVC pipe is then inserted through the drill stem as it is extracted from the hole.

The plastic pipe which is used as a liner is Schedule 80, Type II, PVC with a 38.1 mm (1.5") inside diameter, conforming to ASTM D1785. It is supplied in 3 m and 6 m (10' and 20') lengths. Slot widths usually vary from .25 mm (.010") to 1.27 mm (.050"), depending on the type of material to be drained (Figure 3). The smaller slot sizes are used in fine-grained materials while the larger are used in the coarser materials. A two-slot circumferential configuration was used on the various Tennessee projects. The specification called for 22 slots per row, per foot using the 1.27 mm (.050") slot size and 42 slots per row per foot using the .25 mm (.020") slot size. The two rows of slots were cut with a 120-degree center-to-center separation. The sections of PVC pipe were cemented together in the field with a fast-setting plastic pipe cement.

The drilling equipment used on the Tennessee projects were crawler tractors with the drilling unit mounted on the side (Figure 4). The drill apparatus, except for the chain-driven drill carriage, is hydraulically powered. The drill spindles on these machines are capable of producing 2992 N-m (2200 ft. lbs.) of torque at 150 rpm. The drill carriage, which has a 3.35 m (11') stroke, has the capacity of applying up to 40,940 N (9200 lbs.) of thrust to the drill bit. Thrust as well as torque is reduced somewhat with depth due to pipe friction. This may become especially significant where there is considerable drift or hole deflection.

Water has been the principle flushing and cooling agent or drilling medium used in horizontal drilling; however, air is being used more and more. In down-hole percussive drilling, for example, the material being drilled is fragmented by a slowly rotating air-driven piston. This method has proven to be successful in shallow drilling (up to 61 m (200')) in rock formations that have a relatively consistent hardness. And while this method has not been used in Tennessee, one contractor reports considerable success in drilling medium to hard granitic and gnessic materials in North Carolina (verbal communication from John Jensen of Jensen Drilling Company, June 1976). Air was used on this project, according to Jensen, because the drill holes in the softer, more friable materials eroded so severely under the action of the return drill water, producing deep channels beneath the drill stem. As pressure was applied to the drill bit, the string of drill stem would flex into the eroded channels and break. Jensen states that many holes and several hundred feet of drill stem were lost as a result.

Two Federal Highway Administration reports, published in 1975 (Report Nos. FHWA-RD-75-95 and FHWA-RD-75-96) give a good state-of-the-art assessment of various drilling methods and equipment used in horizontal bore holes. While these reports are concerned principally with deep pilot bores for tunnels, much of the information is applicable to horizontal drilling for drainage purposes.

CASE HISTORIES

State Route 1 By-pass, Dickson, Tennessee:

As stated previously, the Tennessee Department of Transportation has utilized horizontal drains on a number of projects since 1972, principally in the correction and prevention of landslides. One rather unique application, however, involved the use of drains to reduce seepage forces along a severely deteriorating box culvert. On this project, 31 drains were installed along the top and sides of a double 2.44 m X 3.05 m (8' X 10') box culvert that had been constructed originally as a simple roadway underdrain, but which later became an extension of an overflow system for a secondary reservoir for the city of Dickson, Tennessee (Figures 5 and 6).

The culvert was constructed originally in 1931 to carry a small tributary of East Piney River under State Route 1, just west of Dickson (Figure 7). In 1955, the city of Dickson constructed an earth dam approximately 61 m (200') upstream from the State Route 1 crossing (Figure 8). In 1963, two lanes were added to the roadway by filling-in the area between the original roadway fill and the dam (Figure 6). Surface water between the east- and west-bound lanes was carried into the west side of the culvert by way of a drop inlet.

A routine inspection of the culvert in late 1973 revealed that both sides had deteriorated almost to the point of collapse (Figure 9). Most of the damage was centered in an area 9 m to 15 m (30' to 50') on either side of the drop inlet.

It was determined that the near-collapse of the culvert could be attributed to several factors; the principle one being that the water table which had risen significantly in the embankment (now an extension of the earth dam) was under considerable pressure due to the close proximity of the impounded lake. This was evidenced by artesian flow from the floor of the culvert and the development of stalactites up to 102 mm (4") in length along the roof. Over the years the weepholes in the sides of the box had become blocked and hairline cracks developed in the roof and sides due to aging and differential settlement in the three separately constructed sections. As these cracks enlarged the reinforcing steel began to corrode, resulting in spalling of the concrete.

The repair measures involved the installation of a corrugated metal liner backfilled with concrete through the deteriorated sections in both sides of the culvert. Since it was considered infeasible as well as objectionable to cut weepholes in the liner, some means had to be found to reduce the seepage forces around the box to prevent a recurrence of the same problem, and also to reduce the possibility of slope failure as a result of a buildup of pore pressure in the outside fill slope. This was accomplished by the use of 31 horizontal drains installed on .9 m (3') centers along the top and sides of the box to a depth of 38 m (125'). The drains had to be deep enough to reduce the seepage forces but at the same time not so deep as to affect the level of the lake.

A 1.2 m to 1.8 m (4' to 6') temporary earth fill had to be constructed across the drainage area at the outlet end of the culvert in order to reach the holes across the top (Figure 10). Drag bits were used to penetrate the cherty clay materials that made up the embankment. Rock bits were used to drill through the wing walls for the side holes. The holes along the top of the box were drilled with little or no slope, and the side holes were drilled with about 5% slope. Penetration rates were very high due to the softness of the clay and the many voids that had developed because of erosion along the top and sides of the culvert. A two-man drill crew using a single rig completed the drilling in approximately twelve days, for an average of about 91 m to 96 m (300' to 315') per 8-hour day. This included two or three moves and setups per day.

Because of the fine-grained materials in the embankment, slot openings of .25 mm (.010") to .51 mm (.020") were used. Figure 11 is a view of the outlet end of the culvert after completion of the drilling, which, as planned, was finished prior to installing the liner. Twenty-five of the 31 drains were moderately active (less than .0006 liters/sec (.01 gpm) to .01 liters/sec (.08 gpm) 24 hours after completion. All of these have remained active to some degree since completion of the project. It is significant to note that while there was considerable flow in the form of steady drips and constant seepage from the roof and sides of the culvert prior to installation of the drains, this had decreased to only very minor seepage at the time the liner was being installed. Water levels in piezometers no. 2 and no. 1 (Figure 6) had also dropped .9 m (3') and 1.2 m (4') respectively and with no noticeable drop in the lake level.

Interstate-75, Campbell County:

The horizontal drains installed in Campbell County were a part of the remedial measures used in several embankments that failed approximately four to five years after the roadway had been opened to traffic. Failure was attributed to the saturation and deterioration of highly slaking shales from which the embankments had been constructed. The embankments along the interval between Caryville and Jellico traversed many drainage swales and deep ravines. Very often these were crossed in such a way as to impede or block the natural drainage. While most of the surface water was controlled and disposed of through pipes and culverts, the subsurface drainage outlets, such as springs, seeps, etc., were very often sealed off by the embankments. These embankments then actually began to act as earth dams, blocking off the flow and causing a rise in the water table. As the water tables rose, the embankments became essentially saturated, and since they were constructed principally of slaking shales, they began to deteriorate, lose strength and fail (Figure 12). The water level in one 55 m (180') fill, for example, was measured to be 19 m (60') above the original groundline. Such a process in many cases is quite slow which is the reason for some of the embankments not failing for as long as five or six years after construction.

Repair measures involved removal of all the failed material--plus that which showed signs of distress but had not failed--installation of horizontal drains, and buttressing of the material on the flanks of the fills that had not failed, as well as buttressing the replaced material. The specification for the buttress material called for non-degradable limestone having a maximum dimension of .9 m (3'), with 50% sized between .3 m and .9 m (1' and 3') and no more than 10% passing the 51 mm (2") screen.

Removal of the failed material was the first task to be accomplished. In one embankment (N.B.L. station 1464+00), however, the flanks of the slide had to be buttressed before all of the failed material could be removed. Due to the saturated condition of the embankment, particularly in the toe area, the steep side slopes (1½:1), the height of the embankment, and the way that the excavation had to proceed, the slide continued to enlarge rather rapidly as excavation progressed. New scarps in the form of arcuate cracks in the roadway continued to develop as each segment or unit of the slide was removed. To stop or at least reduce this movement, it was determined that the flanks of the slide had to be buttressed before excavation could proceed (Figure 13). Once the flanks were stabilized, the failed material was removed without further upslope movement. Two levels of horizontal drains were installed on 6 m (20') centers, separated in elevation by 7.6 m (25'). The 13 holes on this particular slide penetrated to depths between 53 m and 61 m (175' and 200'). All were drilled on 5% slopes. As the drillings in the lower level were completed, the construction of the buttress proceeded. The drains were encased in 152 mm (6") corrugated metal pipe and extended through the buttress prior to building the buttress to the next drainage level (Figure 14). All drains in this slide (1464+00) were quite active initially, with flows up to .25 liters/sec (4 gpm) in at least half the drains. Figures 15, 16, and 17 depict conditions prior to the slide, after failure, and after repair.

More than 15,000 m (50,000') of drainage was installed in a twin fill (centered at station 1380+00) which had failed on both outside slopes because of conditions similar to those encountered at the 1464+00 slide. Ten levels of drains were installed on one side and four on the other. On both sides a total of 189 drains were installed on 3 m and 6 m (10' and 20') centers with 3 m to 6 m (10' and 20') vertical separations between each level. The average depth was 82 m (270'), which included minimum depths of 30 m (100') and maximum depths of 183 m (600'). Penetration rates were quite high with depths up to 152 m and 183 m (500' and 600') being achieved by one rig in one 8- to 10-hour shift.

The embankment was composed essentially of highly weathered shale, with some sand and silt materials, and occasional small to medium--.3 m to 1.5 m (1'-5')--sandstone or siltstone boulders. A .51 mm (.020") slot size was used for all drains on the project. To expedite the work and to avoid delaying the embankment replacement and buttress construction, the contractor frequently ran two 8-hour shifts, using as many as three rigs per shift.

Initial flows in the lower drainage levels along the north-bound-lane were very high. Forty-nine of the fifty-two holes (94%) were producers with initial flows ranging from .0013 liters/sec (.02 gpm) to .47 liters/sec (7.5 gpm). The average initial flow of these 49 holes was 0.1 liters/sec (1.58 gpm). Only 42 of the 75 holes (56%) placed in the upper five drainage levels were producers. The overall success rate (number of initial producers in total drilled) for the N.B.L. drains was 72%. It is important to note that while some holes were dry initially, they later became active, especially during periods of heavy precipitation.

Flows from most drains dropped off significantly 24 to 72 hours after completion. Table 1 shows the flows from selected drains along the north-bound-lane over a period of about three to four months.

Most drains in the lower drainage levels were installed on 5% grades. Grades of as much as 20% were used, however, in an attempt to pick up water along the sometimes steep and highly irregular embankment-natural ground interface. The precise sources of groundwater for this particular ravine were rather difficult to determine because of the complexity of the geology. The direction of dip of the interbedded sandstone, shale and siltstone strata that underlie the area is approximately S-50°-E at angles of between 20° and 35°. Subsurface water is believed to migrate along strike toward the ravine through at least two sandstone aquifers that are sandwiched between thick--15 m to 46 m (50'-150')--units of shale and siltstone. These lithologies vary considerably, however, and they tend to thicken and thin along strike. This, coupled with the ruggedness of the terrain and the fact that the surface exposures had been covered by up to 55 m (180') of fill, made source predictions even more difficult. As a result, many of the drain locations were made on a trial and adjustment basis. Where good flows were obtained, drains were added; where little or no water was found, the number was reduced. Precise locations and depths of all drains were established in the original design; however, many adjustments were made as construction progressed.

None of the three embankments have shown any sign of distress since of remedial repairs were completed in July 1973.

Interstate-40, Roane County:

Without question, the most complicated and frustrating landslide problems in the history of highway construction in the State of Tennessee have occurred along a 6.4 km (4-mile) section of Interstate-40 near Rockwood (Royster, 1973). Construction on this segment began in late 1967 and, beginning with the failure of a massive fill in January 1968 along the east-bound lanes between stations 2003+00 and 2018+00, more than 30 slides had to be corrected before all four lanes could finally be opened to traffic in the late summer of 1974 (the west-bound lanes were opened to two-way traffic in December 1972). Remedial measures included partial to total removal; minor grade and alignment changes; various restraint devices, such as rock buttresses, gabion walls, a Reinforced Earth structure, and soil berms; as well as various dewatering systems such as French drains, vertical wells equipped with automatically actuating pumps, and horizontal drains.

Horizontal drain installations totaled more than 15,240 m (50,000'); approximately half of which were installed in 1972-73, and the remainder in 1975-76. Most of the drains were used as the corrective measure, or in conjunction with other measures, in both cut and embankment failures. Some approximately 2400 m (7875'), were used, however, to prevent failures from occurring along cuts and fills that were determined to have materials and conditions similar to those that failed.

The materials and conditions in Roane County are considerably different from those along the Dickson and Campbell County projects. The I-40 alignment at Rockwood traverses the Eastern Cumberland Escarpment through approximately 244 m (800') of elevation in the 6.4 km (4-mile) problem area (Figure 18). The rock formations include limestones near the base of the escarpment, shales, siltstones, sandstones and conglomerates that, because of rather severe faulting in the near vicinity, are often badly folded, jointed, and weathered. In many places these lithologies are overlain by thick accumulations--up to 15 m (50')--of colluvium. Colluvium, or slope debris', can be very unstable in a side-hill environment when it is underlain by weaker, less pervious materials such as clay or weathered shale. Water, migrating along the interface of these materials, reduces the shear strength which oftentimes results in failure where loads are added, as in the placement of an embankment (Figure 19), or where toe support is removed, as in the excavation of a cut (Figure 20). Horizontal drains were used at Rockwood in an attempt to capture water moving along the interface or to divert it before it reached the interface. In some cases excellent results were obtained; in others, the benefits were negligible.

As with the Campbell County project, groundwater sources were very difficult to pinpoint; mainly because the outcrops were obscured by colluvial materials. Springs exiting from the colluvium along the face of the escarpment may actually have emerged from conduits in the inplace rock 50 m to 150 m (165'-500') or more upslope. The thickness of the colluvium also varied considerably over relatively small areas; for example, from 1.5 m to 15 m (5'-50') over a distance of only 30 m to 61 m (100'-200'). This variance was due more to the troughs and lows filled with colluvium in the highly undulating and irregular sub-topography of the area than to build-ups of colluvium over a uniform under-surface. This condition was one of the principle reasons why it was so difficult to measure and predict and thus quantify the forces and resistances for engineering analyses throughout the project.

The first drains used at Rockwood were installed in an effort to stabilize an approximate 12 m (40') cut in front of a gabion wall. Failure resulted when drainage behind the wall became blocked, allowing water to percolate through the lower cells of the wall into the foreslope, which consisted of colluvium overlying weathered shale. Failure, as determined by slope inclinometers, occurred approximately 1.8 m (6') below the colluvium-shale interface, or at an in-slope depth of 5.5 m (18') (Figure 21). Seventeen horizontal drains were installed on 6 m (20') centers and 5% grades to depths of 24 m to 37 m (80' to 120') along the base of the slope. All but four of the drains (13 of 17) were active initially, producing flows from .001 liters/sec to .13 liters/sec (.02 gpm to 2 gpm). In addition to the horizontal drains, a 1.2 m X 1.2 m (4' X 4') trench was cut parallel to the toe of the slope, a perforated .46 m (1.5') pipe installed, and the trench backfilled with drainage stone (Figure 21). This was done to hasten drying in the toe area and to provide an outlet for seepage from the slope below the drains. The work was done in November and December 1972. By late spring of 1973 the slope had become essentially stable despite the fact that precipitation during that time was considerably above normal (Royster, 1975). No additional movement has been observed in the more than four years since the drains were installed.

Horizontal drains were also used in an attempt to stabilize the right flank of a large horseshoe-shaped slide in which all of the slide material between the outermost scarps had been removed during repair in the fall of 1972. During the heavy spring rains in 1973, portions of the right flank (as viewed from the crown) began to break and flow laterally (Figures 22 and 23). As in most of the Rockwood slides, the failure involved colluvium moving over weathered shale. A total of 20 drains were installed to depths of 46 m (150') on 10% to 20% grades in the three slumped areas. The holes were randomly spaced with the locations of most being determined by the results from three or four test holes. Initial flows, while not spectacular, were on the order of .006 liters/sec to .019 liters/sec (.1 gpm to .3 gpm). The right flank has essentially stabilized over the years, and while it cannot be stated with certainty that the increased stability was totally a result of the drainage, it surely must have been a contributing factor.

As stated previously, approximately 2400 m (7875') of drainage was installed to prevent failures along cuts and fills that were determined to have materials and conditions similar to those that had failed. Most of this drainage was installed in and beneath embankments in approximately 12 separate locations to prevent or minimize pore pressure buildup at the colluvium-weathered shale interface which had been the cause of most of the previous failures (Figure 19). All of this drainage was installed in 1972-73, and as of this writing (summer 1977), only one failure had occurred in those areas; that being in March 1977, along the east-bound lane near station 2139+00.

The most troublesome interval on the Rockwood project has been along the east-bound-lane between approximately stations 2003+00 and 2018+00. Construction began in this area in May 1967 with clearing and grubbing. Grading began in August 1967. Initial failure occurred in January 1968 with the embankment about 27 m (90') high or 15 m (50') below planned grade. Some adjustments were made in the alignment and gradient and shallow interceptor drainage trenches were constructed in an effort to stop the movement. With continuing movement into the spring 1968, however, a consultant was retained to investigate the failure and develop remedial measures. Several alternatives were developed and each analyzed as to cost and degree of effectiveness. The alternatives proposed included total removal and replacement, deep drainage with galleries and wells, drilled shaft restraint, relocation, and a minimal stabilization plan that included drainage trenches above and below the fill. Because the costs, the minimal stabilization plan was chosen even though the factor of safety was calculated to be only on the order of that of the original slope prior to failure. This meant that movement would probably continue, especially during wet periods, but it would be within tolerable limits and could be maintained by periodic patching of the pavement on each side of the slide area (Figure 24).

While no major movements occurred once the embankment was constructed to the redesigned grade--12 m (40') below the originally planned grade--minor movements continued to occur that totaled 152 mm to 203 mm (6"-8") vertically per year between 1969 and 1972.

It was during the latter part of this period that the Department began installing the first horizontal drains in the state on the I-75 Campbell County project. Since the initial results appeared to be good on I-75, it was felt that some additional stability could be gained by installing drains along the base of the 2003+00-2018+00 fill. As a result, forty-four drains were installed on 5% to 10% grades to depths of 23 m to 46 m (75'-150') throughout the problem interval. Only approximately one-half of the drains were producers initially, but some of these produced flows of as much as .03 liters/sec (0.5 gpm). Most of the drains that were dry initially, however, became active during wet periods.

Between 1972 and 1975 movement continued but to a lesser degree than before. Vertical movement, measured along the top of the subgrade, was on the order of 76 mm to 127 mm (3"-5") per year.

The failure mechanism of the 2003+00-2018+00 slide is somewhat different from virtually all of the other failures at Rockwood. While most of the failures have occurred along the colluvium-weathered shale interface, this failure involves both this zone and a deeper failure zone that passes through sandstone, siltstone, shale, and limestone formations that have been severely folded, jointed and fractured. While movement in the shallow zone occurs when there is average infiltration, movement in the deeper zone apparently does not occur except during periods of heavy and prolonged precipitation. In an effort to locate the zones or strata where water pressure was the highest, pneumatic piezometers were installed. These devices were installed at one and in some cases two levels in six different core borings. The levels at which the piezometers were set were chosen on the basis of relative porosity and permeability as evidenced by lithology, structure (faulting, jointing and fracturing), and degree of weathering, staining, solution, etc.

Except for one piezometer, the results obtained were essentially inconclusive. The failure to pinpoint the zones of high water pressure came as no surprise, for this is frequently the case where there are interbedded lithologies that are badly jointed, fractured, and folded. Unless there is unusually good geologic control in the form of borings and outcrops, it is virtually impossible to identify and isolate the more active aquifers and conduits in such areas.

It was later concluded that most of the water affecting the stability was migrating along strike of the steeply dipping beds, collecting in a colluvium-filled trough--that possibly had developed along a cross fault--centered beneath the embankment.

In August 1975, the Department let to contract a project to install horizontal drains in the trough area below the fill, as well as in the three other areas on the project. Along the 2003+00-2018+00 slide, the contract called for the placement of 33 drains fanned out from two centers below the fill (Figure 25). Ten holes were drilled at Pad "A" and twenty-three at Pad "B", all on 5% grades and to depths ranging from 27 m to 146 m (90' to 480'). Depths up to 183 m (600') were planned for several holes, but these simply could not be achieved under the conditions and with the equipment and procedures available at that time. Of the 4842 m (15,885') of drilling planned for Pads "A" and "B", only 2973 m (9,755') was completed for a total completion rate of 61%. This is not considered good by today's standards, but as stated, it was probably all that could be reasonably expected with the geologic conditions at the site and the available rotary equipment. Most of the holes were terminated because of the inability to penetrate relatively thick 2 m to 5 m (6.5' to 16') zones of well-indurated sandstone.

Initial rates of flow for most drains, especially at Pad "B", were very high, ranging from approximately .06 liters/sec (1 gallon/min) to .95 liters/sec (15 gallons/min). For the most part, the drains have been very active since installation (Table 2). For example, on March 30, 1976 following a 2.2 inch rainstorm, the total output of the 23 drains at Pad "B" was 1.78 liters/sec (28.2 gallons/min). This can be compared to more average total flows of 0.74 liters/sec (11.8 gallons/min) on March 10, 1976 and 1.06 liters/sec (16.7 gallons/min) on May 21, 1976 (Figure 26). It is interesting to note that while most of the drains at least doubled their flows during or following rainstorms, some flows actually diminished. For example, following the March 30th rainstorm, the flow of drain no. 10 at Pad "B" was only .0019 liters/sec (.03 gallons/min), while during more average times such as March 10th and May 21st it was flowing at a rate of .03 liters/sec (.5 gallons/min) and .026 liters/sec (.42 gallons/min) respectively. In addition to no. 10, six other drains at this location also had flow reductions immediately following the March 30th rainstorm. And, as was the case with no. 10, they also returned to higher average flows soon thereafter. There is probably no important significance that can be attached to this, other than that it reconfirms the fact that gound water flows in such environments are not easy to analyze and interpret and most assuredly are not always predictable.

In the more than a year since completing the drainage installation (March 1976), total vertical movement measured along the right scarp where it crosses the roadway (at approximately 2004+00), has been about 15 mm (.6"). Since this is the total movement that has occurred over two wet seasons (the springs of 1976 and 1977), the results must be considered excellent.

Interstate-40, Cocke County:

The Interstate-40, Cocke County slide--more often referred to as the Hartford Slide--has been active since soon after construction began in 1962. A segment of the cut adjacent to the west-bound lanes failed in June 1962, causing a temporary disruption of construction. The roadway was brought to final grade in the fall of 1964. Shortly thereafter, the west-bound lanes at the toe of the cut began to rise vertically. Since 1964, almost continuous maintenance involving the removal of broken pavement, regrading, and resurfacing has been necessary to keep the west-bound lanes open.

The Hartford slide is rather unique in that only the west-bound lanes have been affected. The reason for this is that movement is occurring at depth within a V-shaped trough of highly fractured and weathered rock that interfaces with a zone of relatively unweathered rock along a line that essentially parallels the centerline of the roadway. The slide may be described as a wedge failure, with the lower part of the wedge being buttressed by the zone of unweathered rock. This unweathered rock zone also serves to block subsurface water as it migrates through the trough toward the river. During periods of heavy precipitation the ground water in the "confined wedge" rises rapidly, and since it is under hydrostatic pressure, the roadway is heaved upward. In addition to heaving the west-bound lanes, there is usually some encroachment of the cut slope into the ditchline (Figure 27).

In 1972, the Department drilled three vertical wells near the center of the "V", or wedge, located on a line between the toe of the cut slope and the ditchline. The wells which were drilled on 7.5 m (25') centers, were equipped with ½ h.p. submersible automatically actuating pumps. An observation well was drilled between two of the pumping wells to monitor fluctuations of the water table. When the pumps were installed in December 1972 the water table stood 2 m (7') below the surface (top of observation well). For approximately one year after activating the pumps the water level varied between 4 m and 6 m (14' and 20')below the surface. According to maintenance records, only one heaving or bulging of the pavement occurred during that time, and that on March 16, 1973 following a 3.5" rainstorm. Until the pumps were installed, heaving occurred at least 3 or 4 times each year and during periods of considerably less rainfall than that which fell on March 16th.

In May 1973 three horizontal test holes were drilled at roadway level near the center of the slide. Using a standard rotary drill and the drilling procedure depicted in Figures 1 and 1a, the maximum depth that could be achieved in either of the holes was 17 m (55'). One hole produced .63 liters/sec (10 gpm) initially, and has not fallen below .32 liters/sec (5 gpm) since. Nevertheless, since the 61 m-107 m (200'-350') depths believed to be necessary to increase stability in the slide could not be achieved, the project was discontinued.

Heaving of the roadway and minor encroachment of the slope continued during periods of heavy rainfall to the point that by early 1974 the three pumps began to malfunction; two through wear and the other through severance or disruption by the slide mass (the third pump could not be recovered due to crimping and deflection of the casing).

Since the wells had proved to be beneficial and since it was apparently not possible to advance horizontal borings deep enough to effectively dewater the slide, eight new wells were drilled on 4.5 m (15') centers across the "V". This was done during the winter of 1975-76.

By the spring of 1976, it became apparent that another effort should be made to drain the slide with horizontal drains, and another test hole was drilled; this time with a drilling rig with higher thrust and torque, a heavier drive chain, and heavier drill steel. This hole was advanced 107 m (350') into the cut slope at roadway level. The initial flow from the drain was .9 liters/sec (15 gpm). Subsequent readings showed flows as high as 3.78 liters/sec (60 gpm).

In December 1976 the Department awarded a contract for the installation of 7,513 m (24,650') of horizontal drains to be installed on four different levels throughout the slide (Figure 28). The contract called for 21 of the drains to be installed at seven separate locations just above the river level along that portion of the roadway that was carried by fill. The idea was to advance the holes through the fill, the unweathered rock zone, and into the weathered and fractured rock zone near the center of the wedge (Figures 29 and 30). The contract also stipulated that the lowermost drains be installed first due to the likelihood that the upper drains would be sheared and thus lost if movement occurred in the lower part of the wedge during periods of high infiltration.

Work began on February 19, 1977, but before any progress could be made on the project the river flooded and capsized the barge and drill rig, washing them approximately 183 m (600') downstream. Just prior to the flood, however, and after considerable effort, one hole was advanced to 59 m (192'), which was just beyond the unweathered rock-weathered rock interface. The boring had been relatively dry to that point, but as soon as the weathered and broken rock zone was penetrated, flowage began at the rate of 6.3 liters/sec (100 gpm). This occurred on Friday, March 11, 1977, and the flood took place on the following Sunday (13th). Prior to the flood, the plan had been to re-enter the hole on Monday (14th) and continue the drilling after the flowage had dissipated. Work was not resumed, however, until July 5th because of the considerable time required to fabricate a new barge and recover and repair the drill rig. By then the boring had "silted-up" badly and, according to the driller, it was not possible to re-enter and complete it. As of this writing, one other hole had been advanced to a depth of 70 m (229') with an initial flowage of 1.26 liters/sec (20 gpm). But like the first hole, there have been many problems as a result of back-pressure, hole collapse, loss of circulation, silting, and binding of the drill steel, etc. At this point there is some question as to whether the 21 holes at the base of the fill slope can be drilled to the required depths with the equipment available. There is no question as to the design concept and plan, however, for the results to date have reinforced the original premise that there is entrapped water in the lower part of the wedge that must be removed if stability is to be achieved.

CONCLUSION

Horizontal drains are not a panacea for correcting or preventing landslides; yet, they are a very viable alternative that most assuredly should be given due consideration in most cases, either as a single remedial or preventive measure or in conjunction with other measures. The reason is simple: water is the principle cause or a principle factor in most all landslides, which means that some form of dewatering must nearly always be considered. Furthermore, aside from minimum periodic maintenance on slides that must be "lived with", correction by dewatering with horizontal drains is usually the next least expensive of all available remedial measures.

As with other remedial or preventive measures, an analysis must be made as to effectiveness. It is in this, however, that the difficulty lies when drainage and only drainage is being considered, because it is not always possible to analyze benefits and effectiveness in a strict quantitative sense. There are just too many variables and unknowns, particularly in geologically complex areas. The decision to use drainage in these areas, therefore, must be based on experience and a sound knowledge and understanding of the geology and geologic structure in and around the slide area.

As to the general state-of-the-art, much more needs to be learned about horizontal drains and horizontal drilling, particularly in terms of equipment capabilities, drilling methods and techniques, hole stabilization, and borehole guidance procedures and capabilities. More information is also needed on existing horizontal drain installations in various soil and geologic materials and environments. There are very few published case histories and current practice papers concerning horizontal drilling and horizontal drains. There are also apparently no textbooks that cover the subject to any significant degree. But in spite of this sparsity of information, it would appear that the use of horizontal drains is on the increase. This is especially true in the eastern half of the United States where usage in the past has been far below that of the western states. As confidence is gained and as information is disseminated about successful installations, horizontal drainage will no doubt become a principle alternative for consideration in the repair and prevention of many types of landslides.

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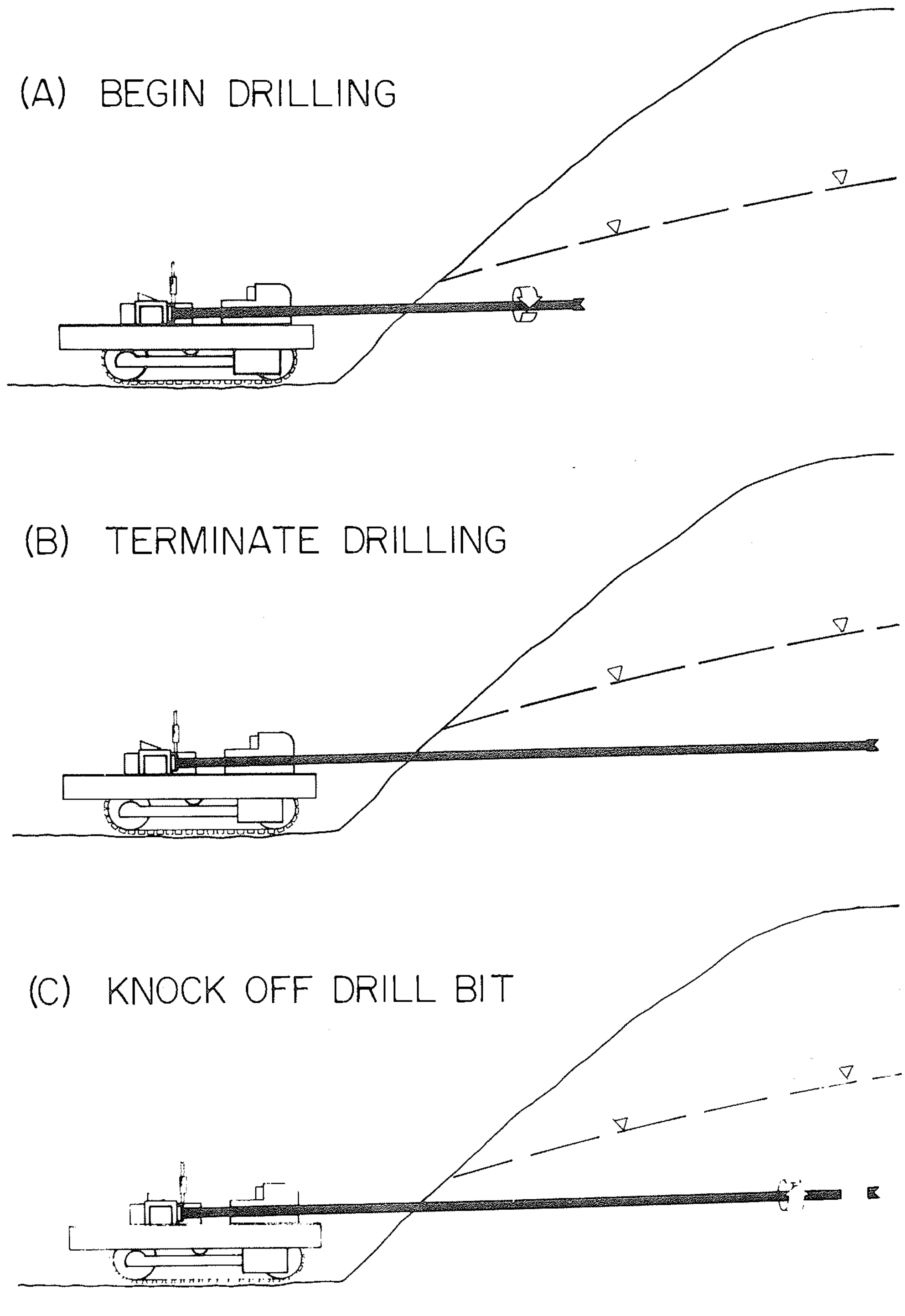


Figure 1. Horizontal Drilling Procedure.

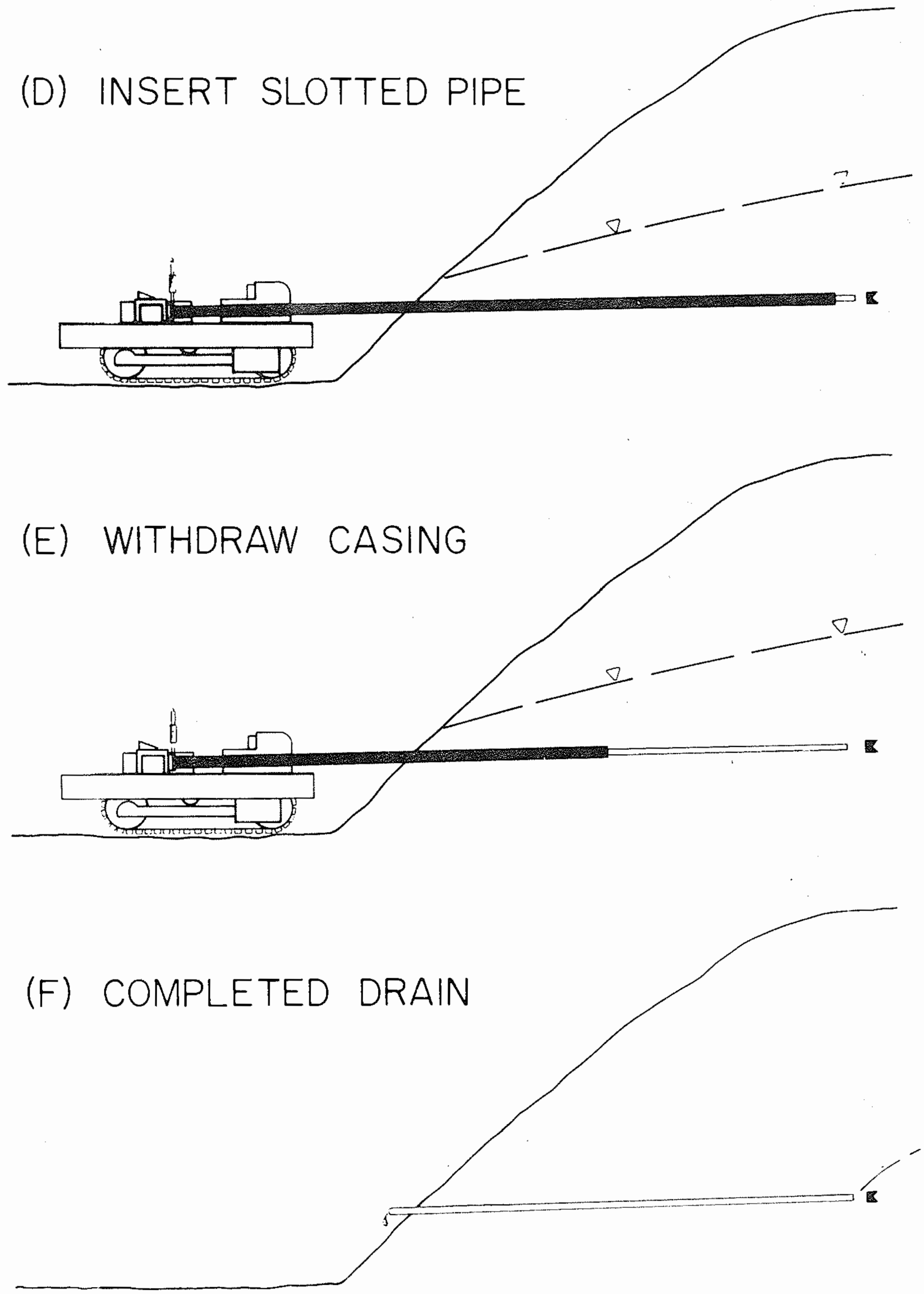


Figure 1a. Horizontal Drilling Procedure (Continued).

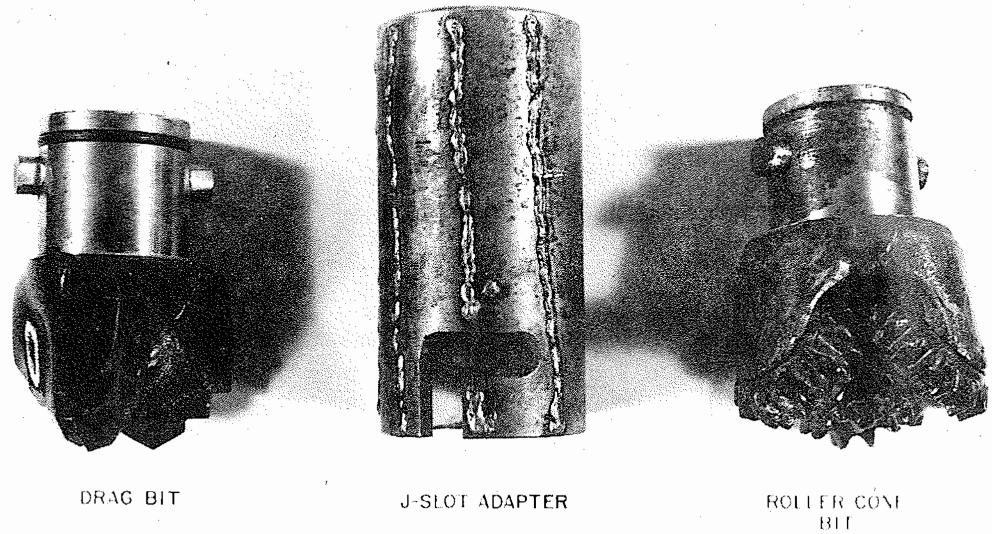


Figure 2. Soil and Rock Bits and Slotted Adapter for Attachment to Drill Stem.

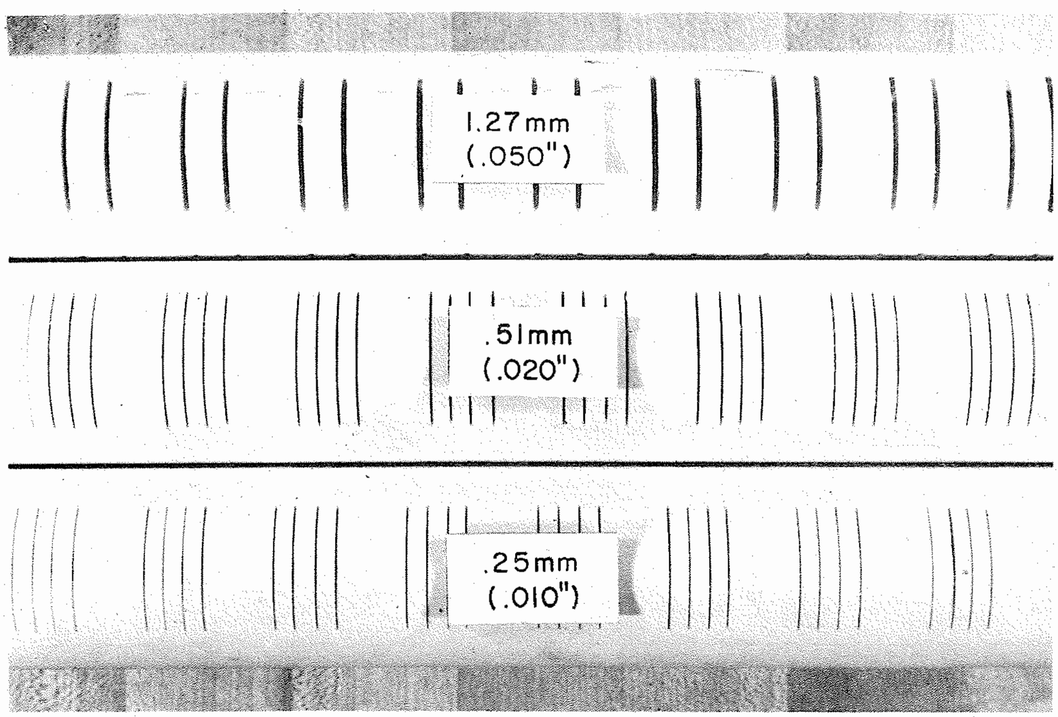


Figure 3. Slot Sizes and Spacings for 38 mm (1.5") I.D. PVC Pipe.

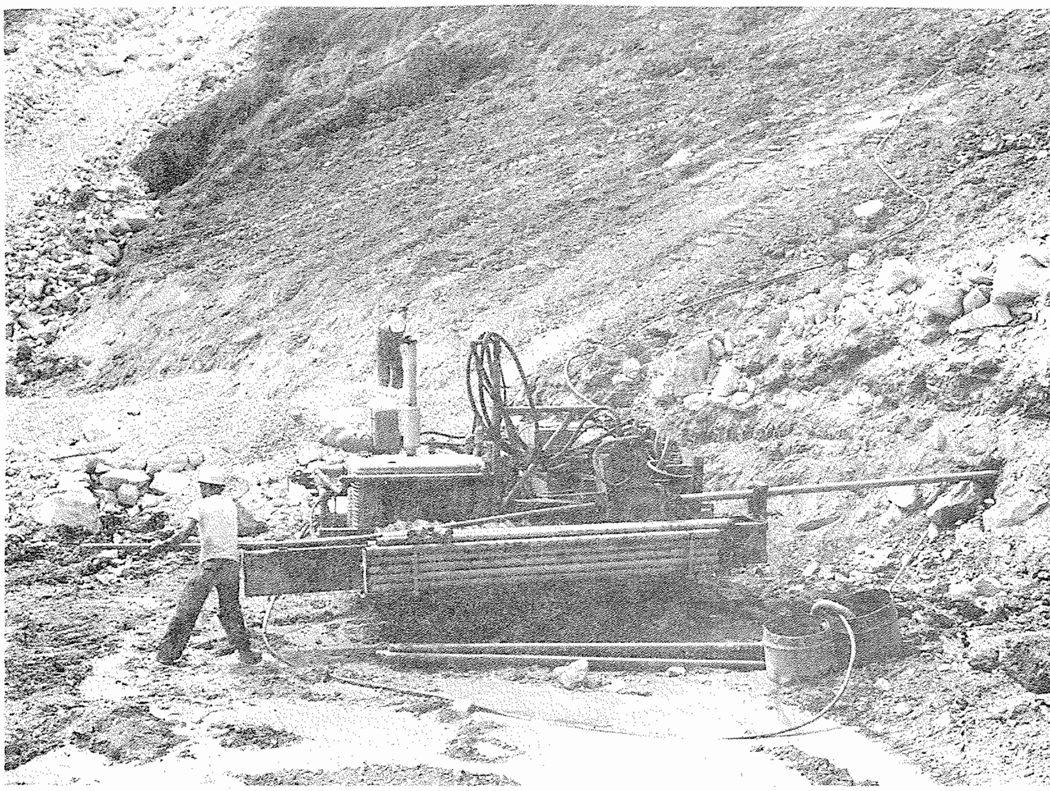


Figure 4. Side view of drilling rig.

Figure 5. Aerial View of State Route #1 Horizontal Drain Project. (Note outlet end of Box Culvert in lower part of photo.

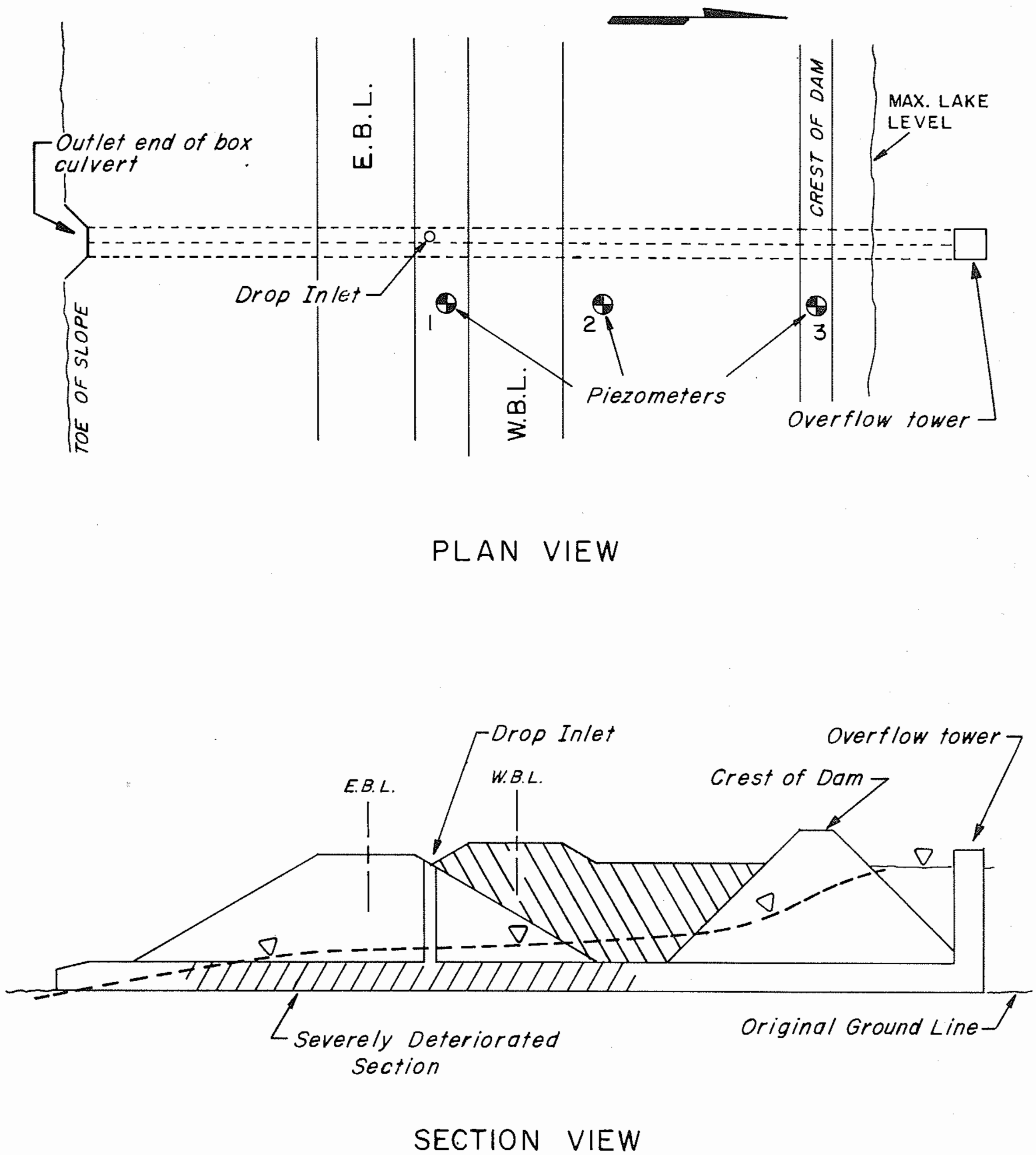


FIGURE 6. Plan and section view of box culvert repair project along State Route 1 By-Pass near Dickson, Tennessee.

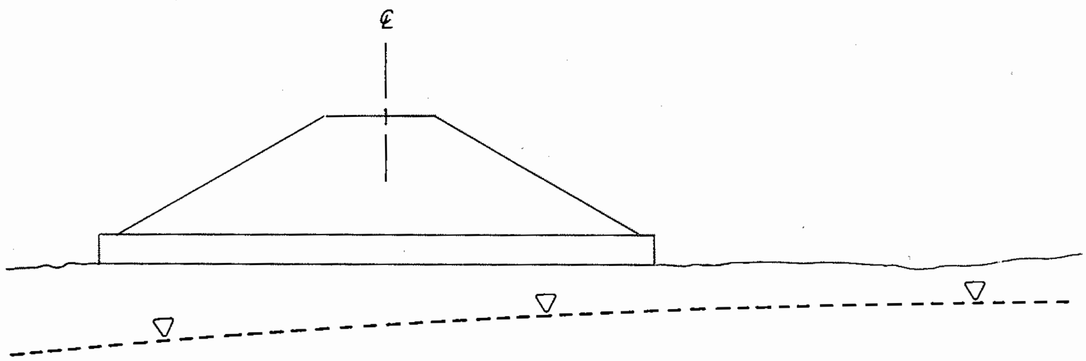


FIGURE 7. Roadway as originally constructed.

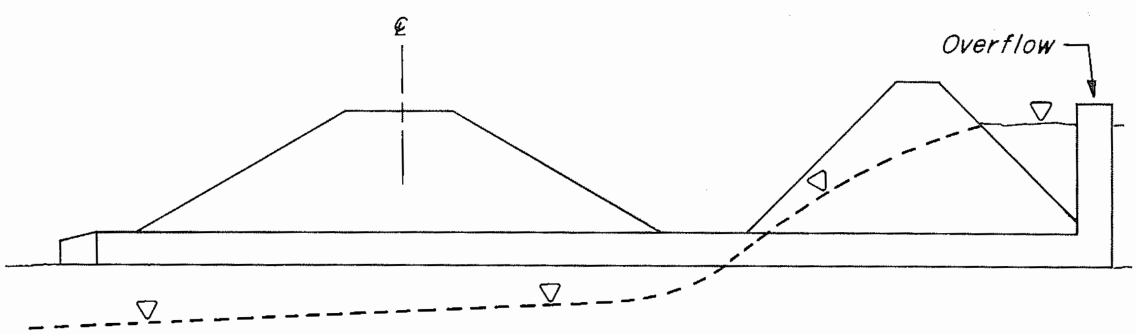


FIGURE 8. After construction of Earth Dam.

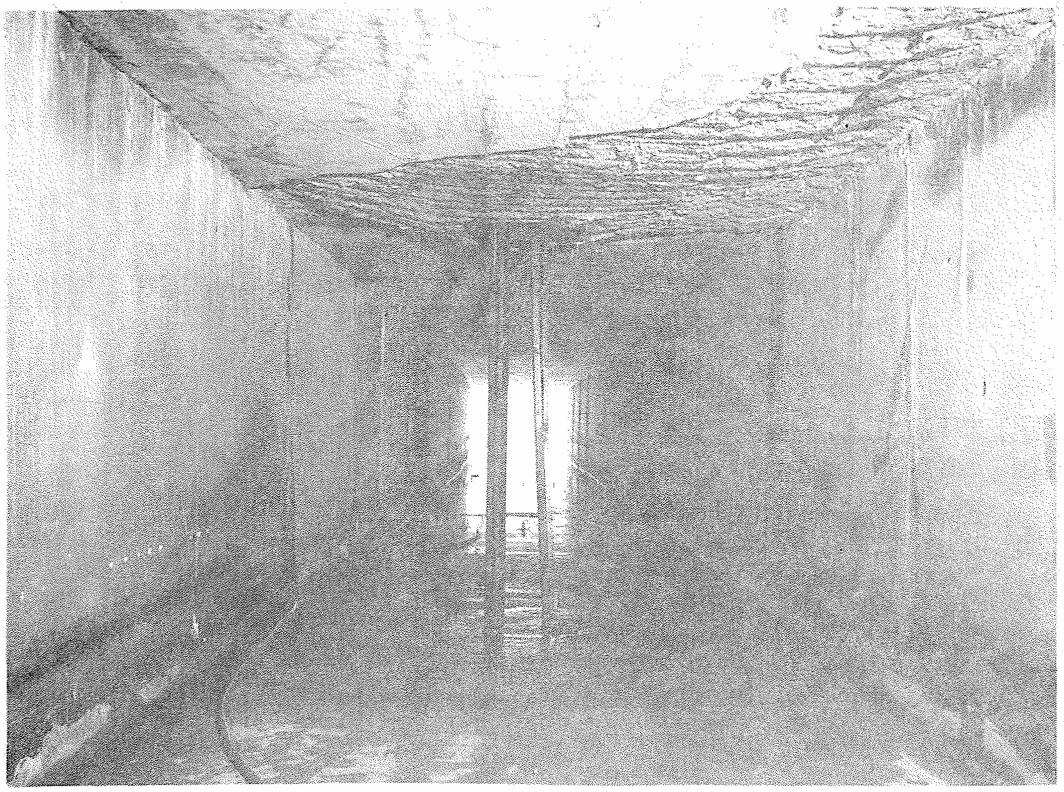


Figure 9. View Inside Deteriorated Box Culvert.

Figure 10. A Temporary Earth Fill had to be Constructed in order to Drill the Line of Holes Across the Top of the Culvert.

Figure 11. View of the Culvert After Completion of Drilling. Note Drains protruding from the wing walls and along the top of the Box.

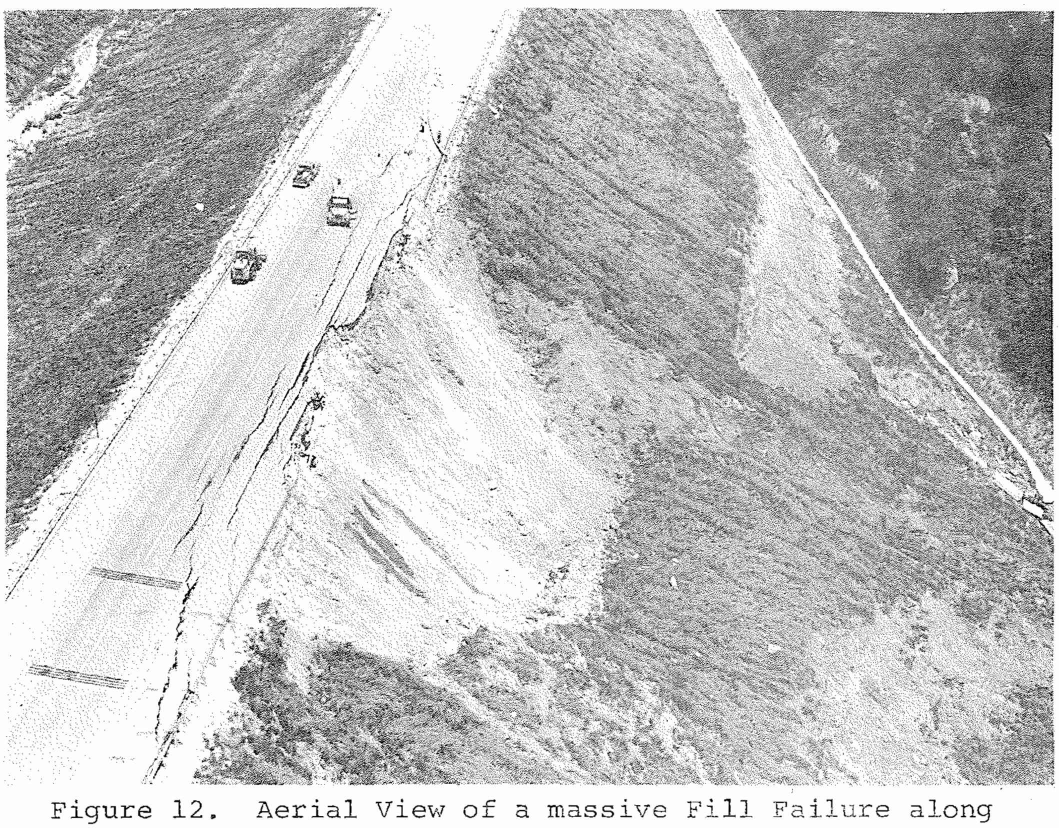


Figure 12. Aerial View of a massive Fill Failure along I-75 in Campbell County.

Figure 13. View of the 1464+00 slide along I-75 in Campbell County.

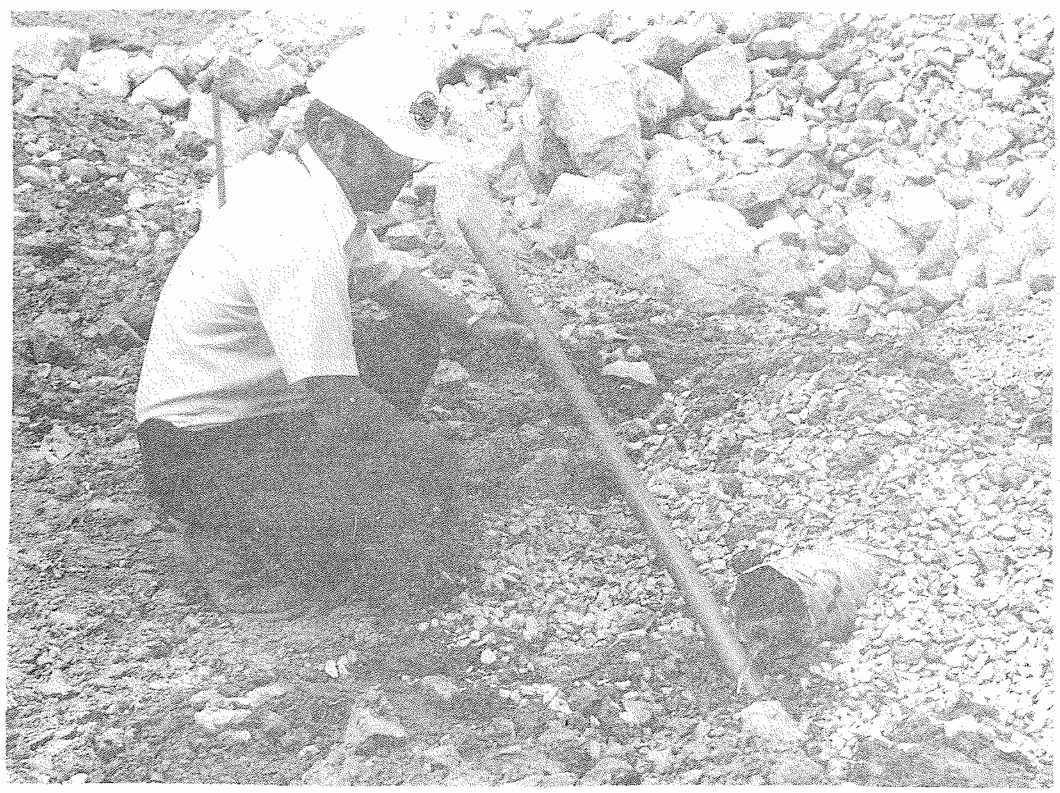


Figure 14. Water issuing from a Horizontal Drain installed in the 1464+00 slide.

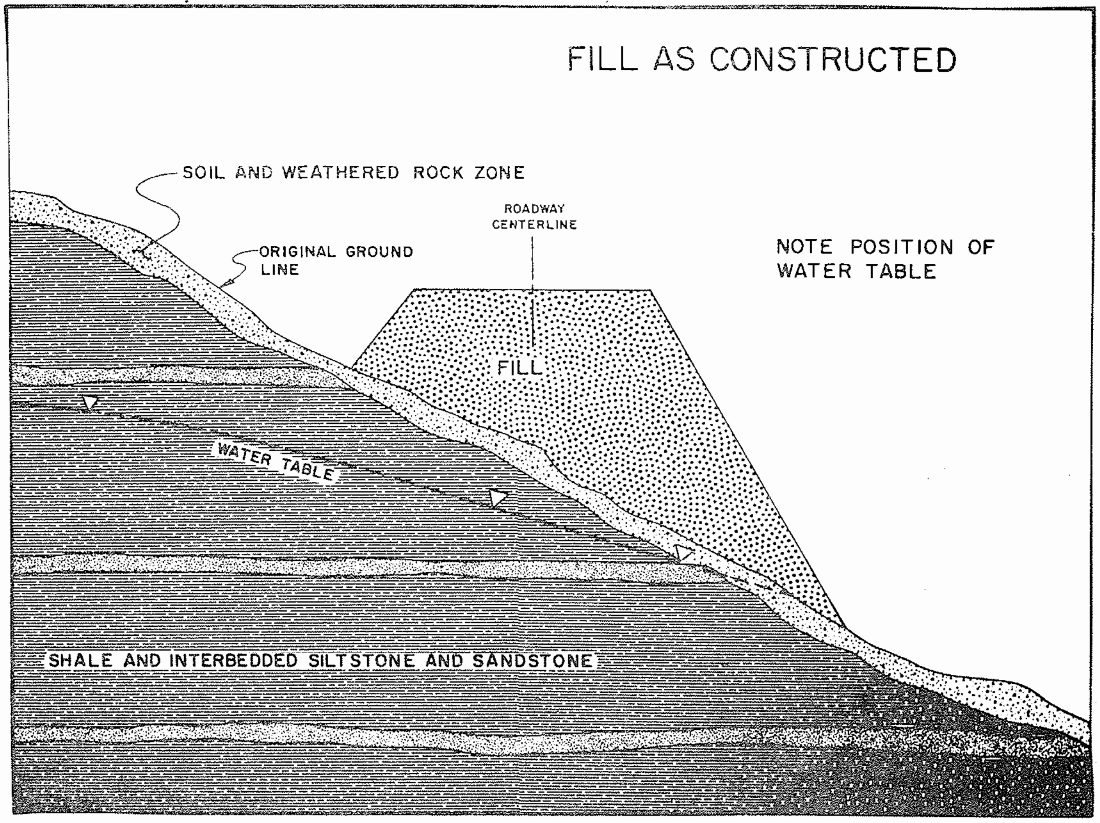


Figure 15.

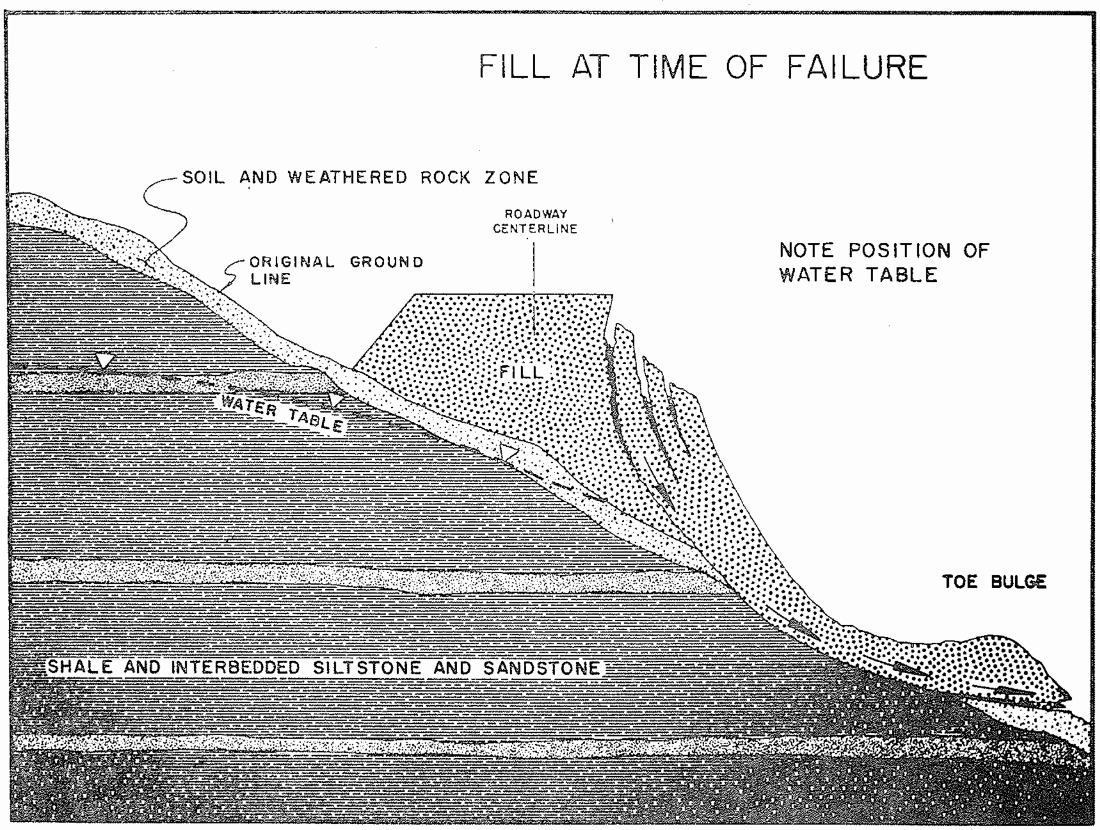


Figure 16.

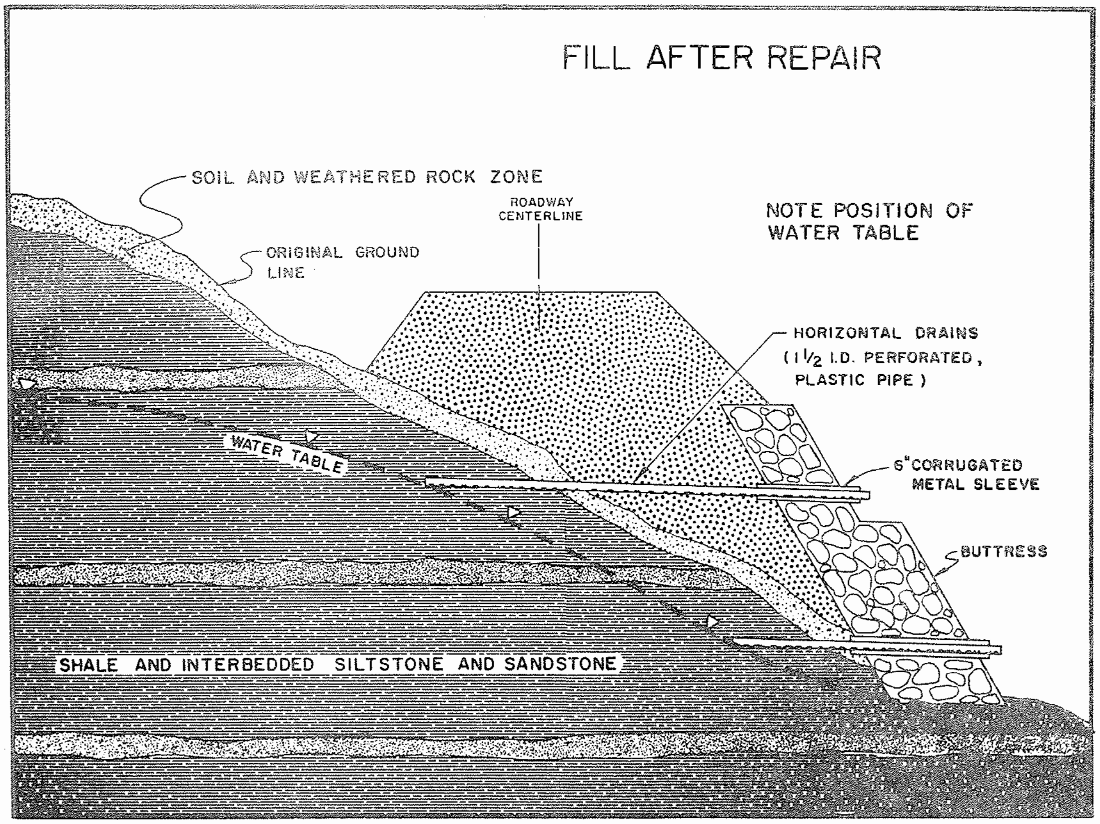
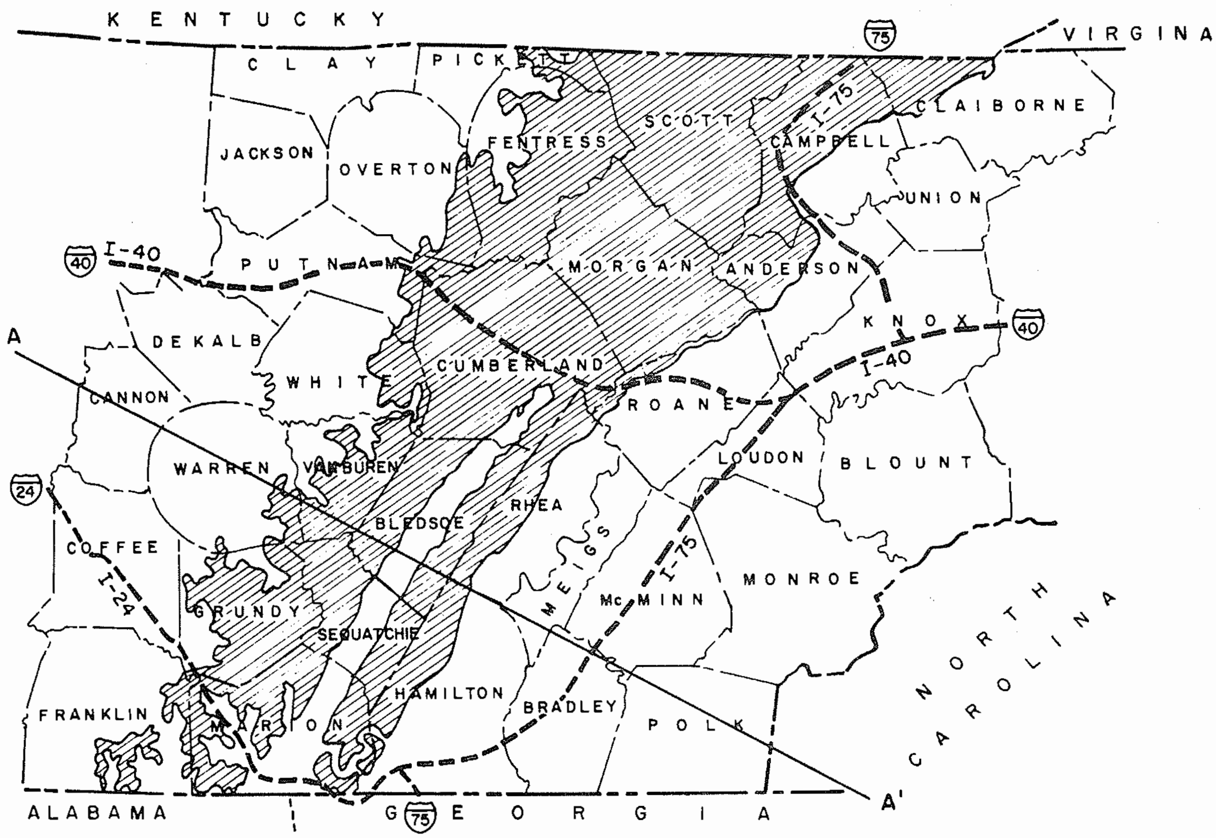
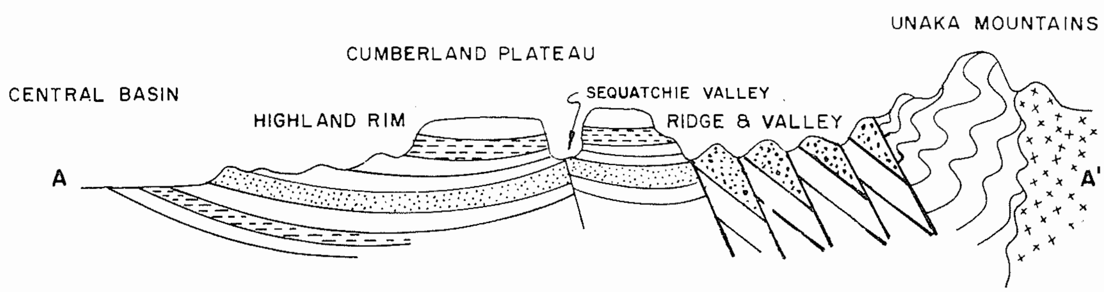

FILL AFTER REPAIR
SOIL AND WEATHERED ROCK ZONE
ROADWAY CENTERLINE
NOTE POSITION OF WATER TABLE
ORIGINAL GROUND LINE
HORIZONTAL DRAINS (1 1/2 I.D. PERFORATED, PLASTIC PIPE)
WATER TABLE
6" CORRUGATED METAL SLEEVE
BUTTRESS
SHALE AND INTERBEDDED SILTSTONE AND SANDSTONE

Figure 17.

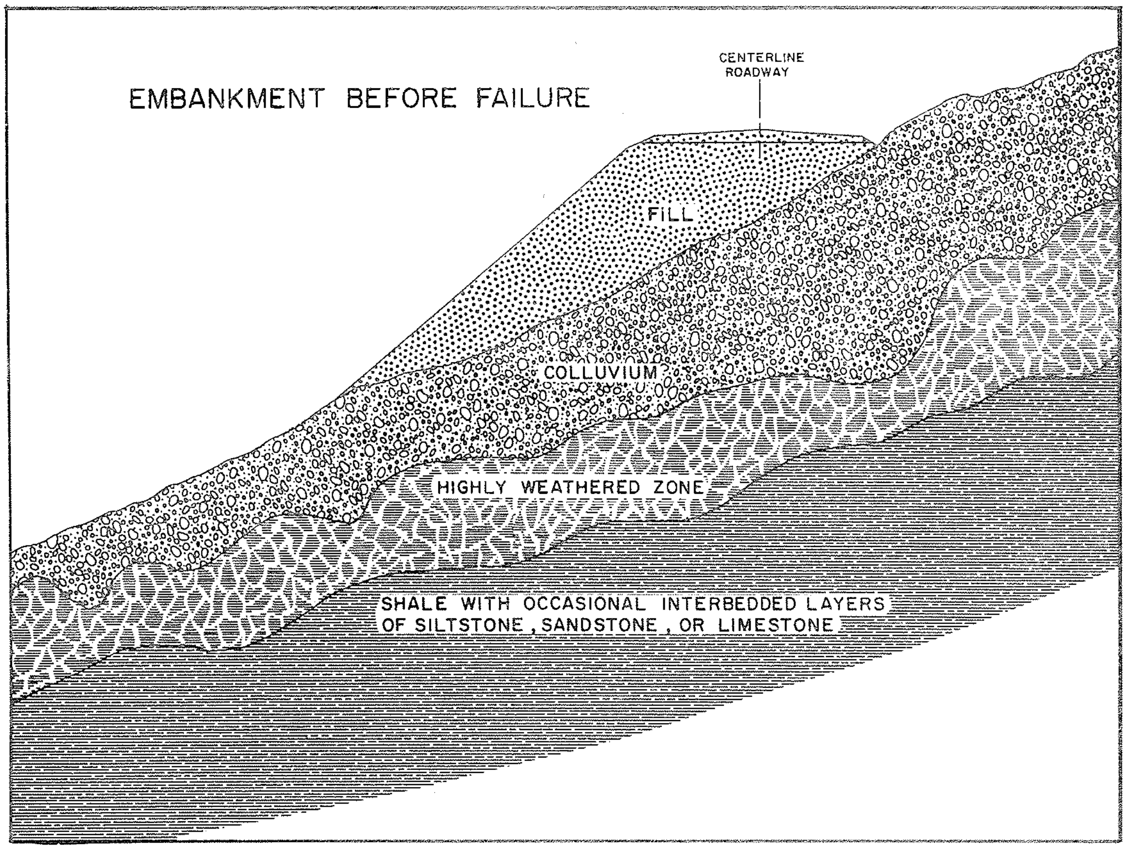




GENERALIZED NORTHWEST - SOUTHEAST CROSS SECTION ALONG A-A'

(Vertical scale highly exaggerated in relation to horizontal scale)

Figure 18.



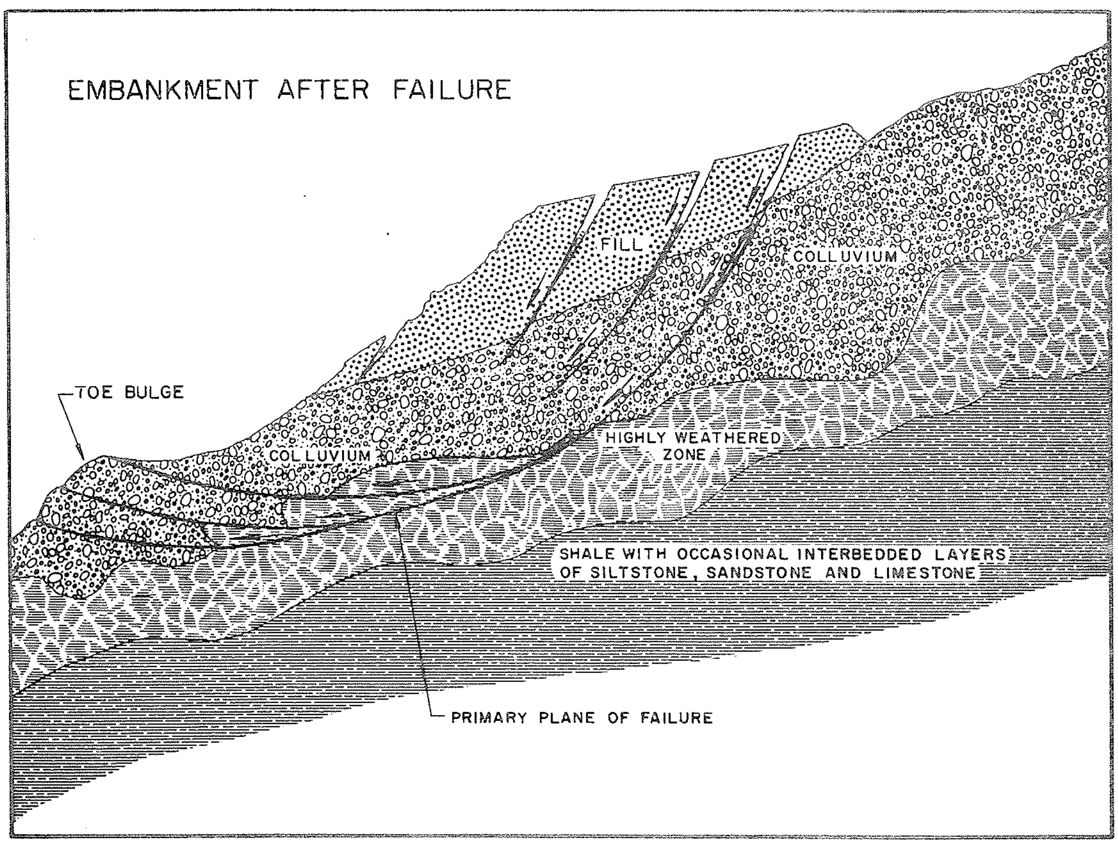


Figure 19. Embankment as viewed before and after failure.

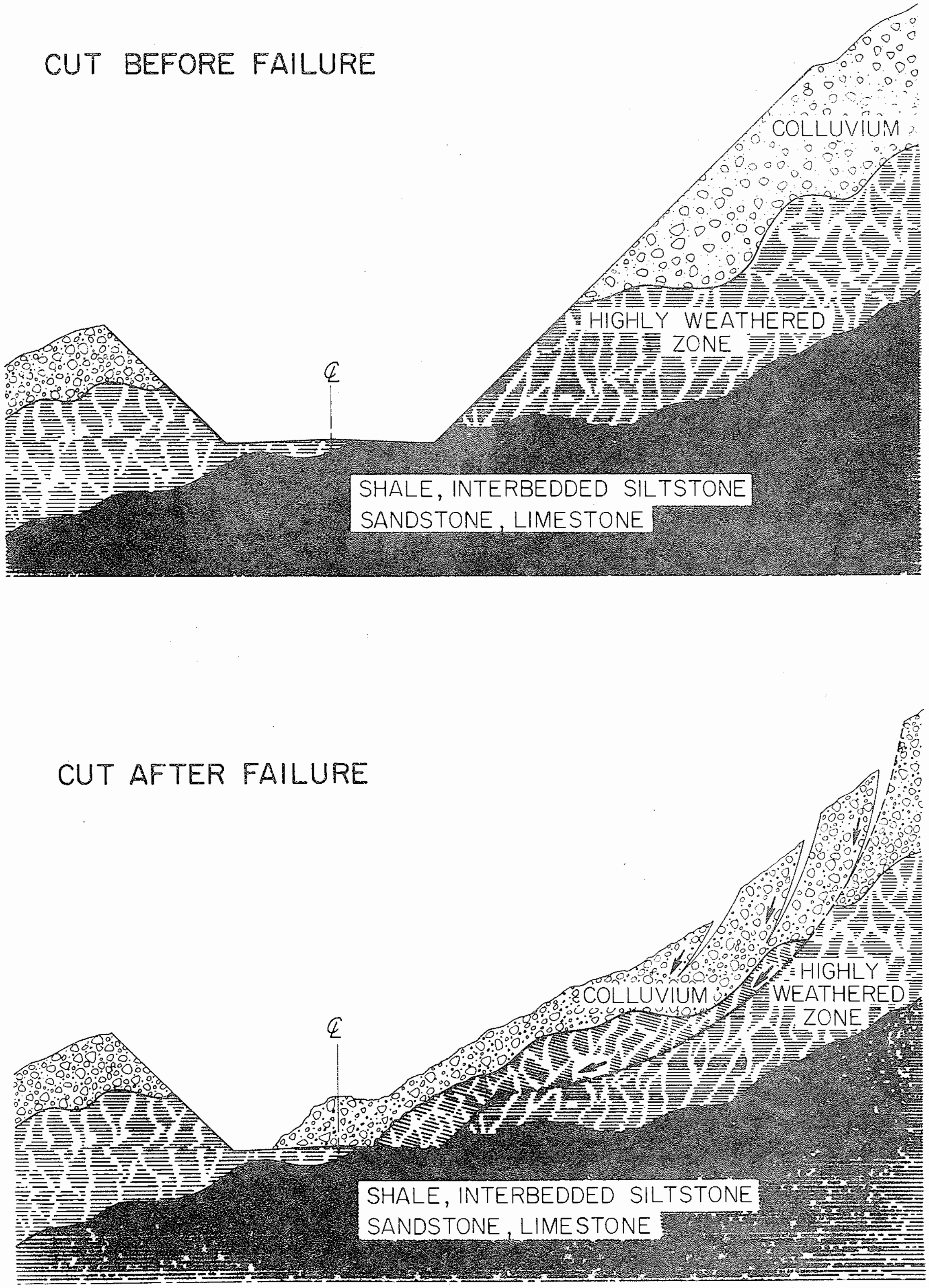


Figure 20. Cut slope before and after failure.

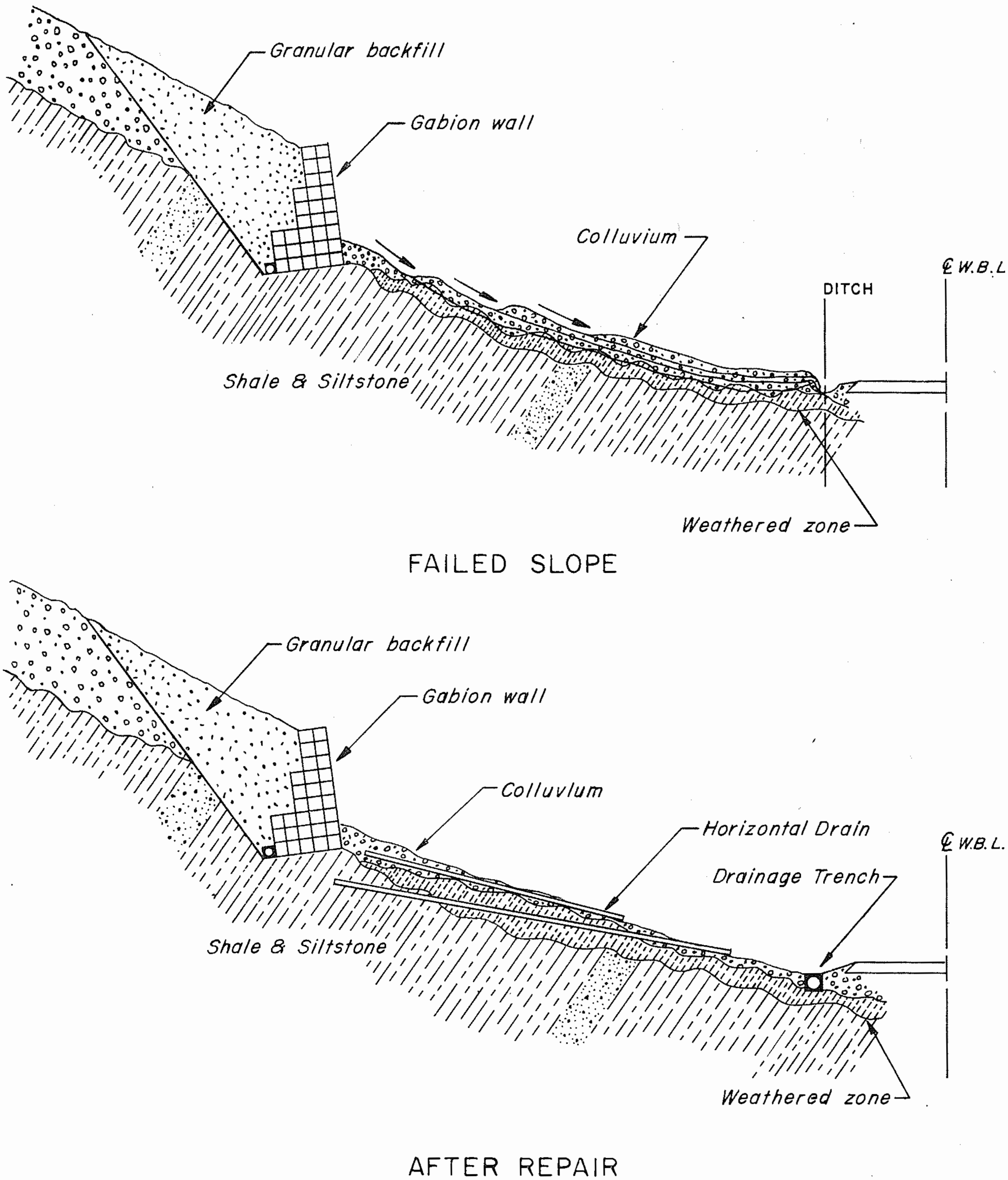


FIGURE 21. Schematic views of slide 4-W at Rockwood before and after repair.

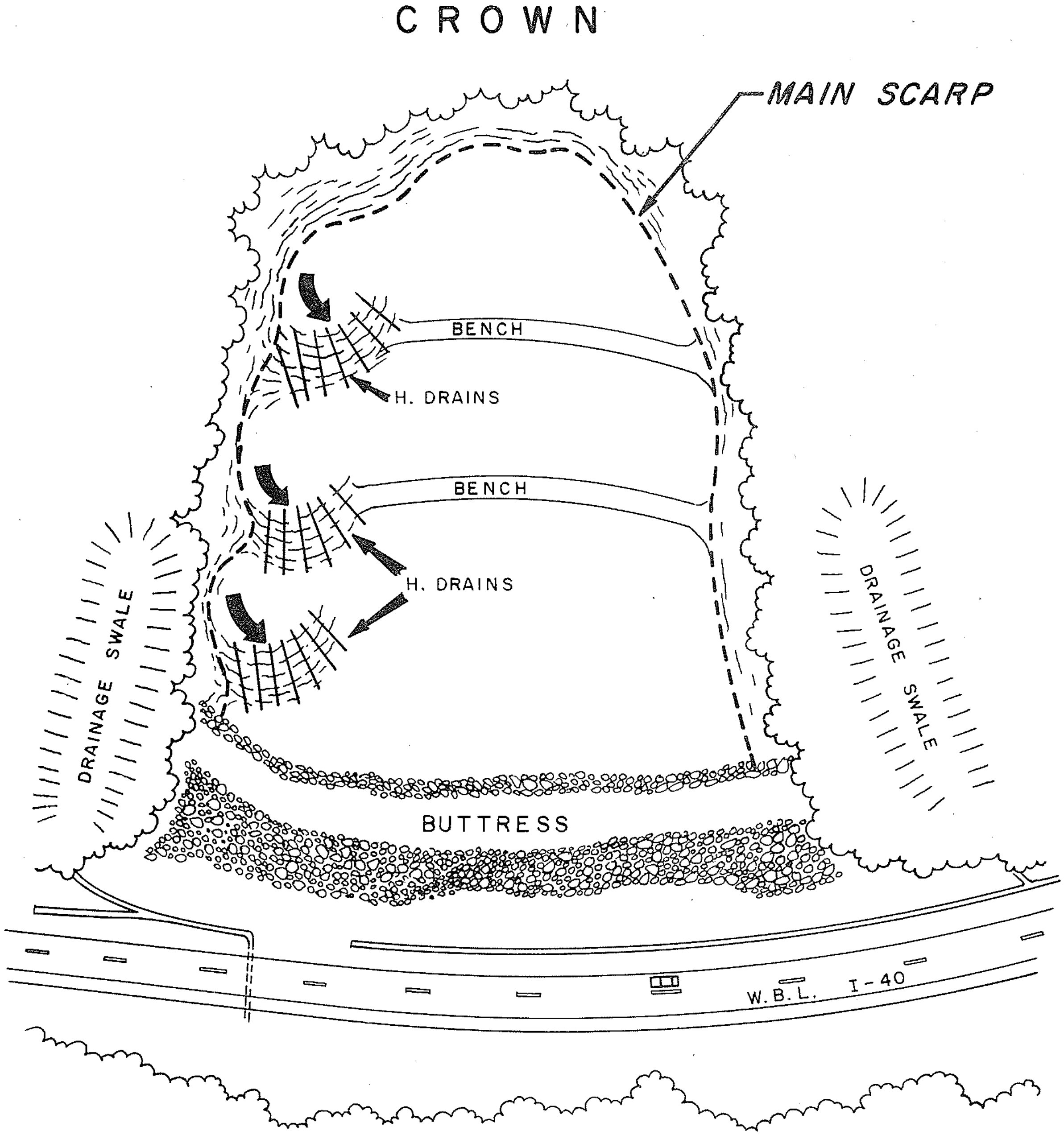


FIGURE 22. Horizontal Drains were used in an attempt to stabilize the right flank of this major slide at Rockwood.

Figure 23. Aerial View of Repaired Slide #8 Along I-40 at Rockwood (see Figure 22).

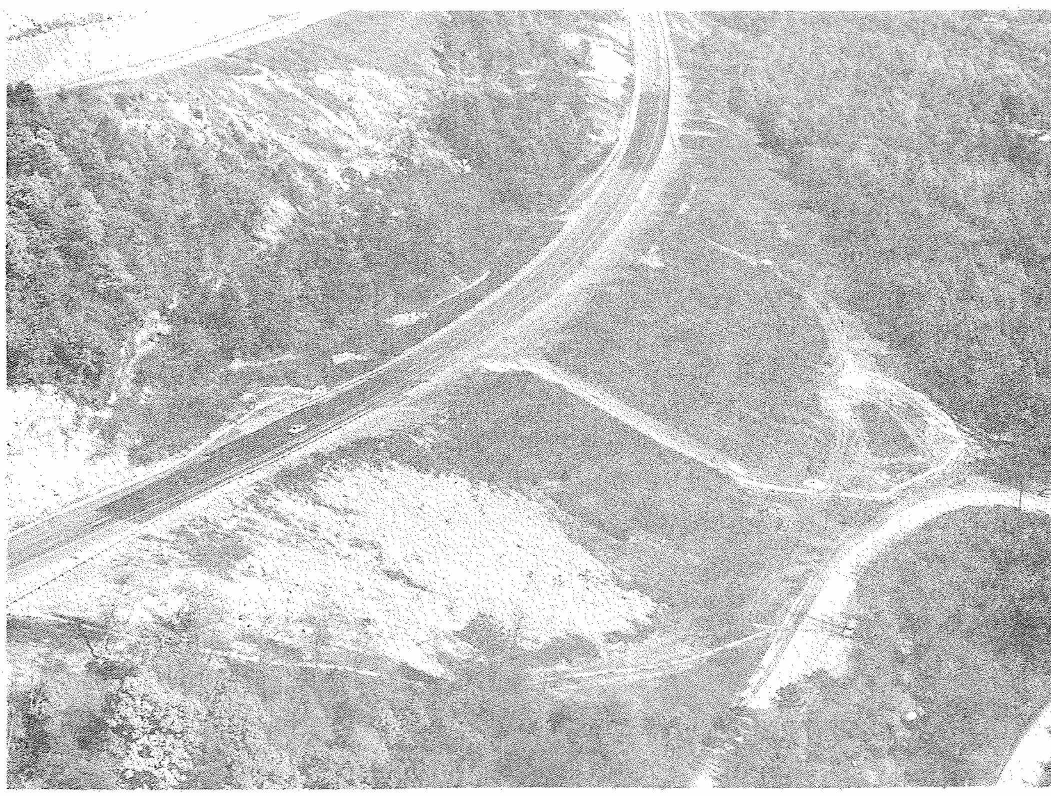


Figure 24. The 2003+00-2018+00 Fill Slide at Rockwood. Newly paved areas on each end of the embankment delineate the lateral limits of the slide.

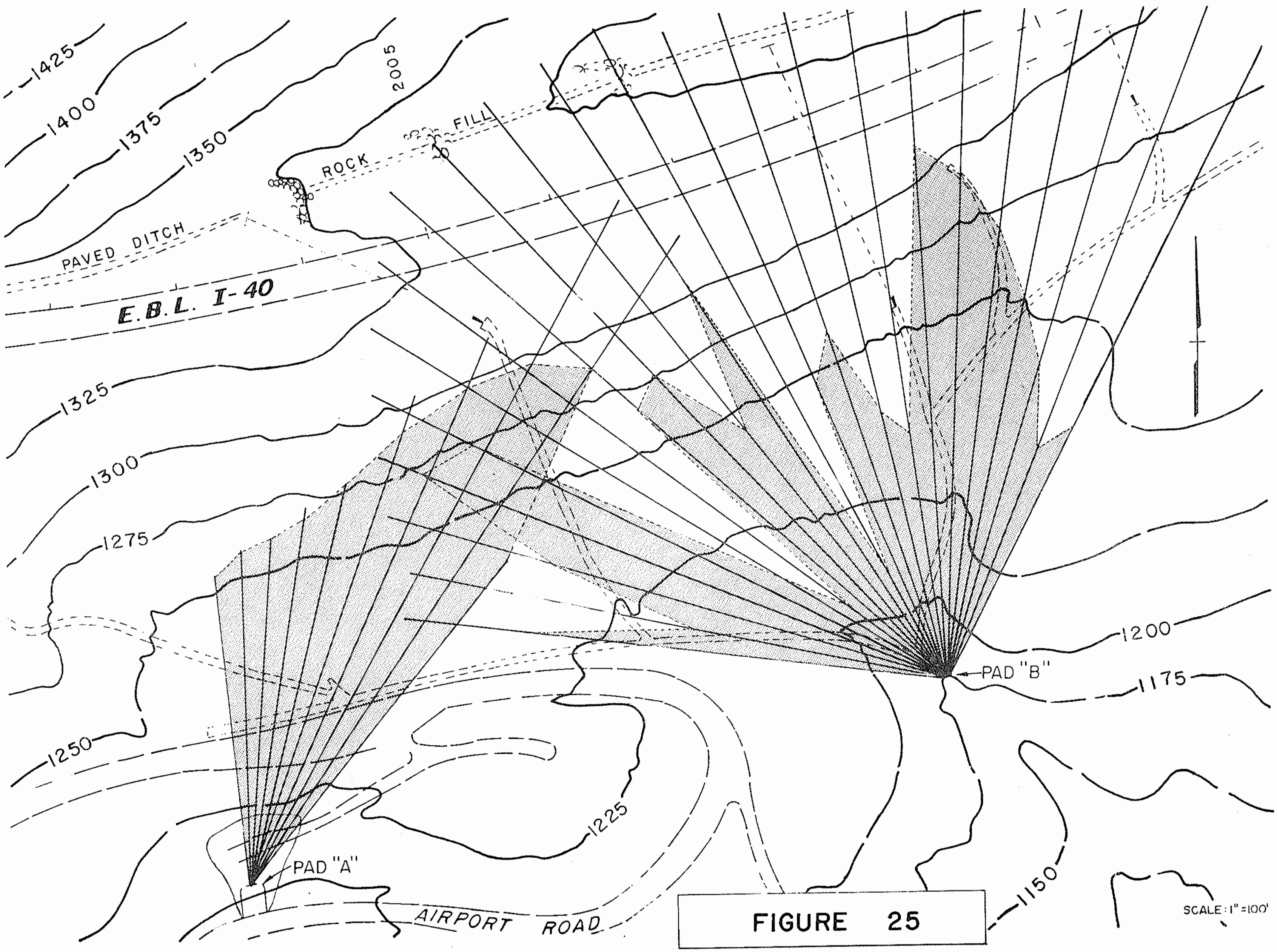

1425
1400
1375
1350
2005
ROCK
FILL
PAVED DITCH
E.B.L. I-40
1325
1300
1275
1250
1225
1200
1175
1150
PAD "A"
PAD "B"
AIRPORT ROAD
SCALE: 1" = 100'

FIGURE 25

Figure 26. Horizontal Drain Installation at Rockwood (see Pad "B", Figure 25).

Figure 27. Aerial View of the Hartford Slide.

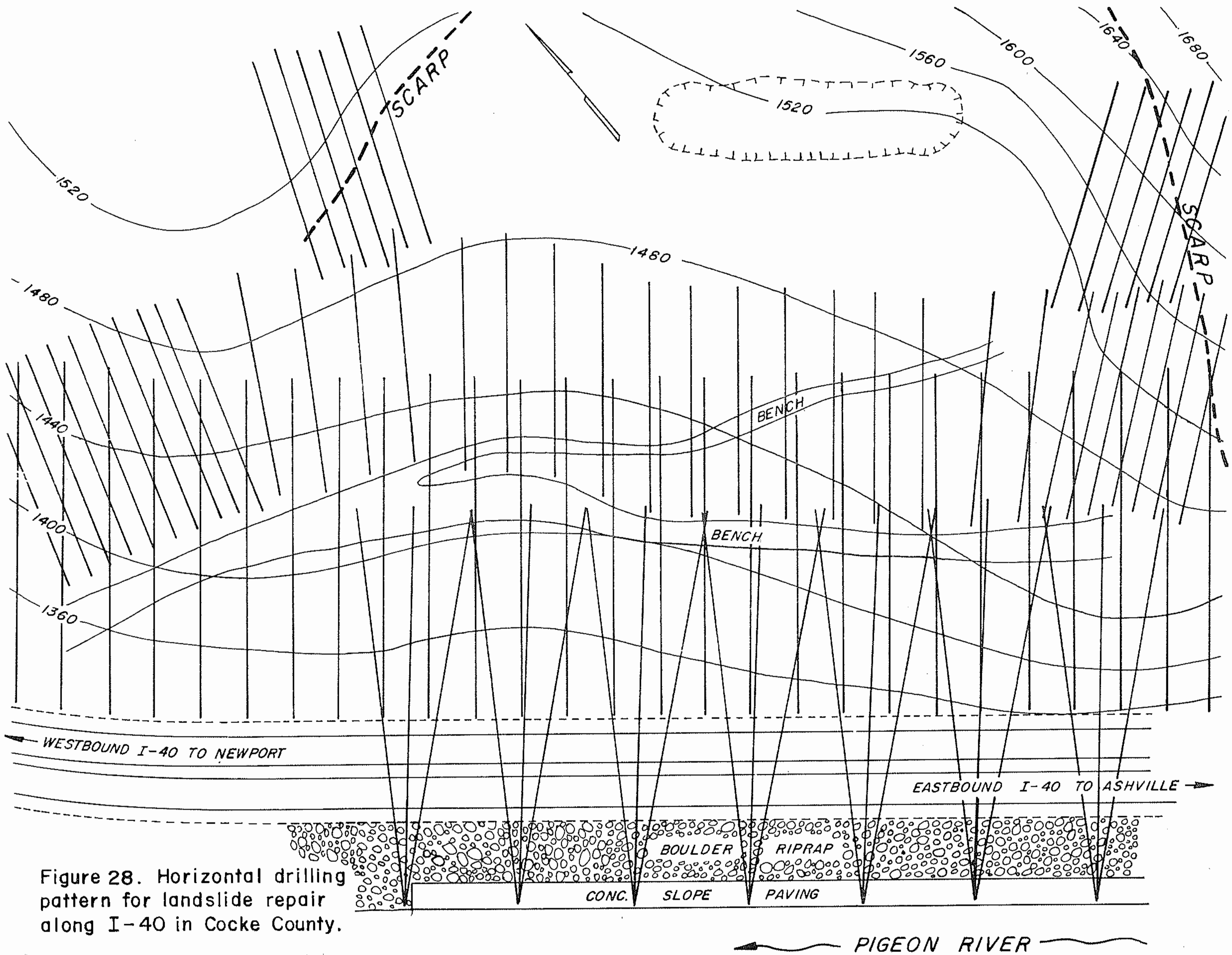


Figure 28. Horizontal drilling pattern for landslide repair along I-40 in Cocke County.

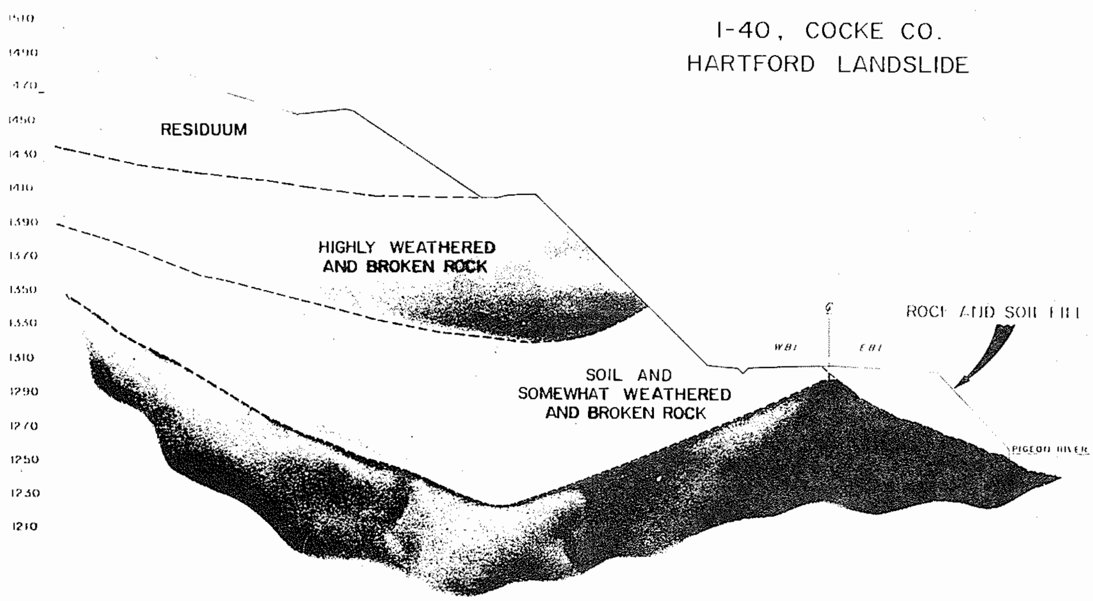


Figure 29. Generalized Cross Section of the Hartford Slide.

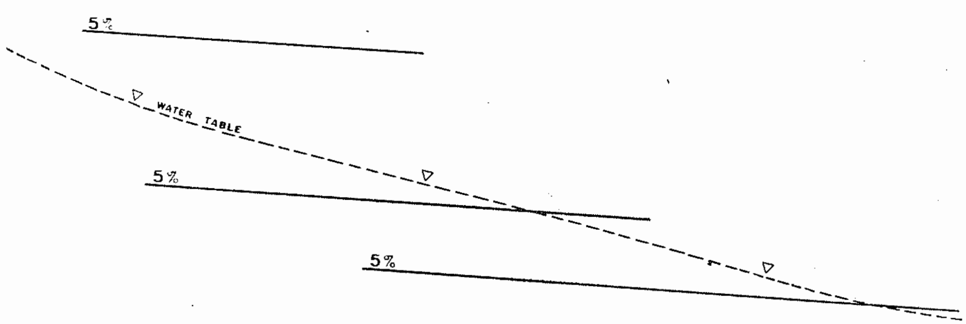


Figure 30. Generalized Cross Section showing the position of the drainage levels.

TABLE I

Rates of Flow of Horizontal Drains Along N.B.L. (Station 1380+00±)

Liters/sec (Gallons/min)

Elevation	Drain Number	DEPTH Meters	DEPTH Feet	DATE	FLOW	DATE	FLOW	DATE	FLOW	DATE	FLOW	DATE	FLOW
1750'	7	107	350	10/31/72	.08(1.3)	11/6/72	.08(1.2)	11/15/72	Dry	1/16/73	-(.06)	3/7/73	-(.06)
1750'	8	143	470	11/1/72	.19(3)	11/6/72	.16(2.5)	11/15/72	.02(.25)	1/16/73	-(.06)	3/7/73	-(.05)
1760'	2	137	450	11/8/72	.42(6.7)	11/16/72	.01(.09)	11/22/72	.01(.08)	1/16/73	-(.05)	3/7/73	-(.03)
1760'	8	140	460	11/11/72	.42(6.7)	11/17/72	.05(.72)	11/22/72	-(.05)	1/16/73	.09(1.5)	3/7/73	.01(.13)
1760'	10	168	550	11/6/72	.03(.5)	11/15/72	.01(.13)	11/22/72	.03(.5)	1/16/73	.01(.13)	3/7/73	-(.03)
1760'	12	165	540	11/10/72	.03(.5)	11/17/72	.47(7.5)	11/22/72	.05(.83)	1/16/73	.05(.83)	3/7/73	.06(1)
1760'	14	137	450	11/8/72	.47(7.5)	11/15/72	.19(3)	11/22/72	.06(1)	1/16/73	.32(5)	3/7/73	.32(5)
1770'	8	107	350	11/15/72	.26(4.2)	11/22/72	.16(2.5)	12/1/72	.01(.12)	1/16/73	Trace	3/7/73	Trace
1770'	9	107	350	11/15/72	.26(4.2)	11/22/72	.14(2.2)	12/1/72	.14(2.2)	1/16/73	.06(1)	3/7/73	.06(1)
1780'	7	91	300	12/1/72	.16(2.5)	12/4/72	.02(.25)	1/16/73	.01(.08)	2/1/73	.01(.08)	3/7/73	-(.02)
1780'	8	104	340	12/1/72	.16(2.5)	12/4/72	.05(.77)	1/16/73	Dry	2/1/73	Dry	3/7/73	-(.03)
1800'	14	116	380	1/13/73	.03(.5)	1/14/73	.03(.5)	1/20/73	Dry	2/2/73	Dry	2/19/73	-(.03)

TABLE 2

Rates of Flow of Horizontal Drains Along E.B.L. (Station 2007+50)

Liters/sec (Gallons/min)

Elevation	Drain Number	DEPTH Meters	DEPTH Feet	DATE	FLOW	DATE	FLOW	DATE	FLOW	DATE	FLOW	DATE	FLOW
1175'	3	98	320	10/6/75	.13(2)	10/10/75	.05(.9)	10/13/75	.04(.7)	10/22/75	.06(1)	11/4/75	.04(.6)
1175'	5	125	410	10/8/75	.05(.85)	10/10/75	.05(.85)	10/15/75	.06(1)	10/22/75	.04(.67)	11/5/75	.03(.52)
1175'	6	46	150	10/8/75	.04(.67)	10/10/75	.04(.67)	10/15/75	.03(.55)	10/22/75	.03(.55)	11/5/75	.02(.33)
1175'	12	27	90	10/30/75	.63(10)	10/31/75	.63(10)	11/16/75	.04(.67)	11/13/75	.95(15)	11/24/75	-(.03)
1175'	13	76	250	10/29/75	.95(15)	10/30/75	.04(.68)	11/6/75	Dry	11/24/75	.01(.17)	12/1/75	-(.02)
1175'	15	64	210	10/24/75	.07(1.1)	10/27/75	.03(.52)	11/3/75	.01(.17)	11/10/75	-(.03)	11/24/75	-(.02)
1175'	18	119	390	10/17/75	.32(5)	10/20/75	.16(2.6)	10/27/75	.1(1.6)	11/3/75	.06(1)	11/24/75	.06(1)
1175'	19	107	350	10/16/75	.63(10)	10/17/75	.32(5)	10/24/75	-(.03)	10/31/75	-(.03)	11/24/75	-(.03)
1175'	20	91	300	10/15/75	.82(13)	10/16/75	.82(13)	10/23/75	.06(1)	10/30/75	.05(.75)	11/24/75	.02(.25)
1175'	21	76	250	10/14/75	.95(15)	10/15/75	.57(9)	10/21/75	.15(2.4)	10/28/75	.01(.13)	11/24/75	-(.02)
1175'	22	58	190	10/10/75	.06(1)	10/13/75	.06(1)	10/17/75	.45(7.2)	10/24/75	.02(.32)	11/10/75	.03(.45)
1175'	23	67	220	10/10/75	.11(1.7)	10/13/75	.06(.92)	10/17/75	.82(13)	10/24/75	.02(.37)	11/10/75	.01(.2)