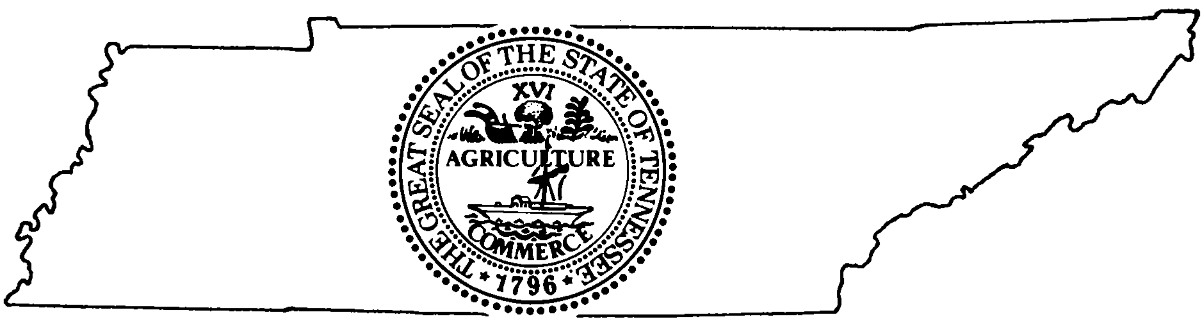
32nd Annual Highway Geology Symposium

PROCEEDINGS



May 6, 7, 8, 1981
Gatlinburg, Tennessee

Hosted By

TENNESSEE DEPARTMENT OF TRANSPORTATION
DIVISION OF SOILS AND GEOLOGICAL ENGINEERING

PROCEEDINGS

THIRTY-SECOND ANNUAL HIGHWAY GEOLOGY SYMPOSIUM AND FIELD TRIP

MAY 6-8, 1981
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CONTENTS

PAPERS

HIGHWAY GEOLOGY SYMPOSIUM:
Its History, Organization, and Function

Born of the need to establish a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium was organized and held its first meeting on February 16, 1950 in Richmond, Virginia. Since then, 31 consecutive annual meetings have been held in 22 different states. Between 1950 and 1962 the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Symposium moved to Lexington, Kentucky in 1965, Ames, Iowa in 1966, Lafayette, Indiana in 1967, back to West Virginia at Morgantown in 1968, and then to Urbana, Illinois in 1969. Lawrence, Kansas was the site of the 1970 meeting, Norman, Oklahoma in 1971, and Old Point Comfort, Virginia the site in 1972.

The Wyoming Highway Department hosted the 1973 meeting in Sheridan. From there it moved to Raleigh, North Carolina in 1974, back to the west to Coeur d'Alene, Idaho in 1975, Orlando, Florida in 1976, Rapid City, South Dakota in 1977, and then back to Maryland in 1978; this time at Annapolis. Portland, Oregon was the site of the 1979 meeting, Austin, Texas in 1980, and Gatlinburg, Tennessee in 1981. The 1982 meeting is scheduled for Vail, Colorado in September. Meetings in 1983 and 1984 are planned for Georgia and California.

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee made up of about 20 engineering geologists and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contributions to the symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. They may succeed themselves for one additional term.

A number of three-member standing committees handle the internal and external affairs of the organization. Some of these are: the By-Laws Committee, Public Relations Committee, Awards Selection Committee, Publications Committee, etc. Committees are held to a minimum for the most part, however, to avoid bureaucratization of the organization. The lack of rigid specialization, requirements, and routine and the relatively relaxed overall functioning of the organization is what attracts many of the participants.

Meeting sites are chosen two to four years in advance and are selected by the Steering Committee following presentations made by representatives of potential host states. These presentations are usually made at the steering committee meeting which is held during the Annual Symposium. Upon selection the state representative becomes the state chairman and a member pro tem of the Steering Committee. Depending on interest and degree of participation, the temporary member may gain full membership to the Steering Committee.

The symposia are generally set-up for two and one-half days, with a day-and-a-half for technical papers and a full-day for the field trip that usually occurs on the second day. In most cases the activities begin on Wednesday morning with the opening session. The field trip is usually set for Thursday, followed by the annual banquet that night. The final technical session usually ends by noon on Friday.

The field trip is the highlight of the meeting. In most cases, the trips traverse from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasional cultural stops are scheduled around geological and geotechnical points of interest. In Wyoming, for example, the group viewed landslides in the Big Horn Mountains; Florida's trip included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and Tennessee provided stops at several repaired landslides in Appalachia.

At the technical sessions, case histories and state-of-the-art papers are the norm. Highly theoretical papers are the exception.

For more information regarding the Symposium, contact any of the steering committee members listed in this publication.

*HIGHWAY GEOLOGY SYMPOSIUM

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K. B. Woods	-	1971
R. J. Edmonson	-	1972
C. S. Mullin	-	1974
A. C. Dodson	-	1975
Burrell Whitlow	-	1978
Bill Sherman	-	1980
Virgil Burgat	-	1981

*In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis, Indiana designed a plaque to be presented periodically to individuals who have made significant contributions to the HGS over a period of years. The award, a 3½" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

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UNSTABLE ROCK SLOPES ALONG INTERSTATE 40 THROUGH PIGEON RIVER GORGE, HAYWOOD COUNTY, NORTH CAROLINA

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ABSTRACT

Since Interstate 40 through the 20 mile length of the Pigeon River Gorge was opened in the late 1960's, rockfalls and slides have been a frequent and major problem. After a large slide in 1978, an investigation was initiated to determine the condition of remaining cut and fill slopes within the area.

Cut slopes through the Pigeon River Gorge range in height from 100 to 300 feet and expose both closely jointed metasediments and colluvial deposits. Cuts generally start at the ditchline, leaving no catchment area at grade for falling material. Failures in the past have ranged from isolated rockfall incidents, to slides blocking one to four lanes of traffic.

This paper describes the methods used during the investigation, the reasons for instability, and the failure modes detected. The alternative remedial measures are presented, and the advantages and disadvantages of each are discussed.

INTRODUCTION

In western North Carolina, Interstate 40 follows the Pigeon River for about 20 miles through some of the most scenic terrain in the Southern Appalachians. Unfortunately, this section has been plagued by rocks falling from the cut slopes and by slides which have in some cases blocked two or more travel lanes. The problem was of such a serious nature that it became necessary to establish and maintain a 24 hour patrol to watch for slides and remove debris from the roadway. In 1978, another in a long history of rock slides occurred, this time raising the question of the extent of instability in the cut slopes throughout the Pigeon River Gorge. In an attempt to respond, the Geotechnical Unit of the North Carolina Department of Transportation was requested to determine problem areas in all cut and fill slopes along the Interstate system in western North Carolina. Due to the obvious stability problems along Interstate 40 through the Pigeon River Gorge, the investigation was concentrated in this area.

SITE DESCRIPTION

The study area begins approximately 30 miles west of Asheville, North Carolina, where the Pigeon River flows in a northwesterly direction through the 20-mile length of its gorge into Tennessee. Interstate 40 is the major link between western North Carolina and eastern Tennessee, with an average daily traffic count of 10,900 vehicles in 1980. Estimated traffic for the year 2000 is 22,200 vehicles. The route crosses some of the most rugged terrain in the

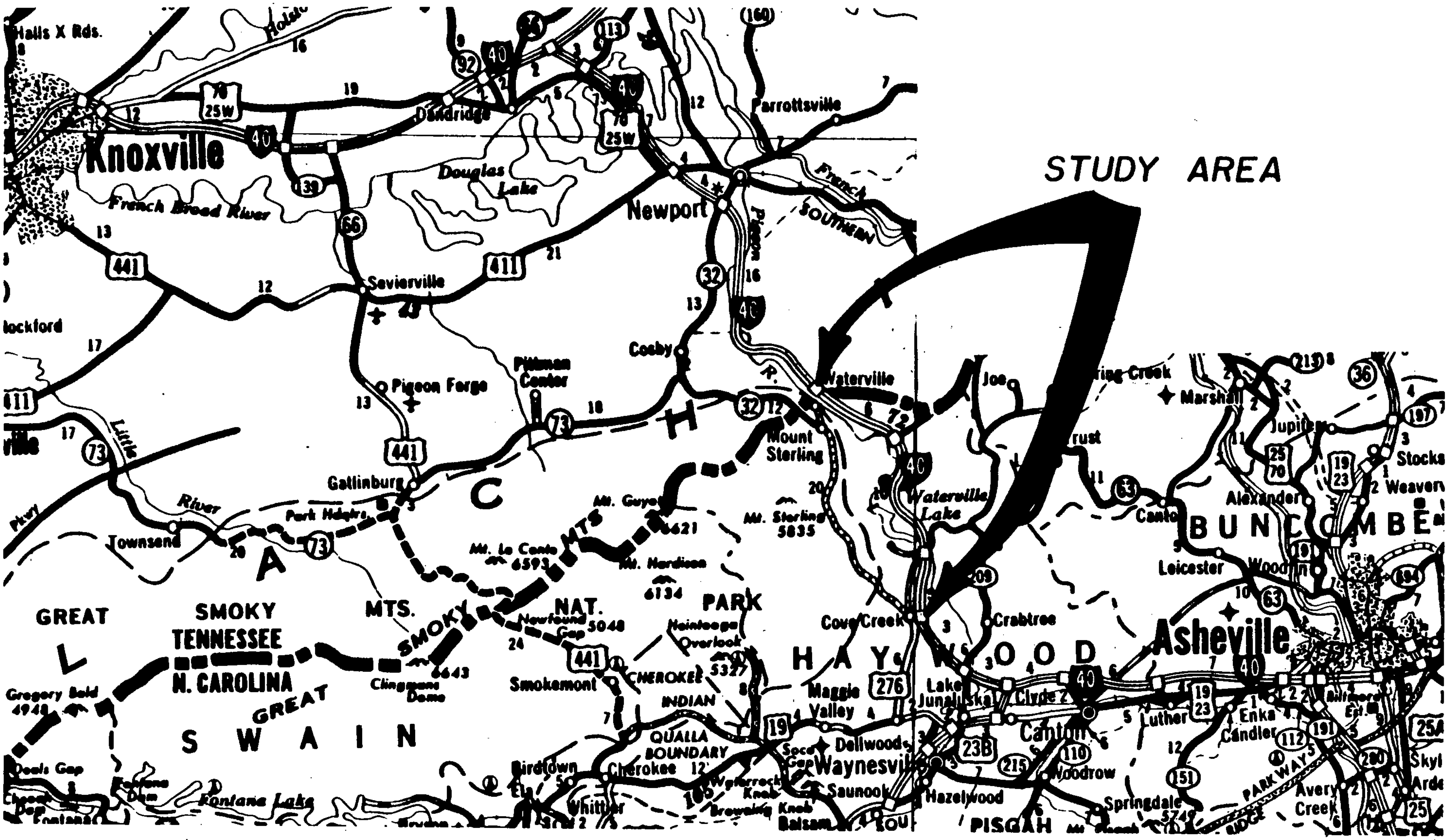

FIGURE 1. INTERSTATE 40 THROUGH THE PIGEON RIVER GORGE.

Smoky Mountains. Within the Pigeon River Gorge, relief between the river and surrounding mountains is greater than 2000 feet. The area lies within an isolated portion of the Pisgah National Forest which contains no commercial development (Figure 1).

As the Pigeon River Gorge is very narrow, construction of Interstate 40 resulted in high, steep cut and fill slopes. Since the route follows the river, it was necessary to cut across the noses of many steep sided ridges and fill across the adjacent narrow stream valleys. Cuts up to 300 feet in height and as great as 1000 feet in length are not uncommon. Long narrow fills toe up in the river and range from 80 to 250 feet in height (Figure 2).

A typical section of Interstate 40 within the gorge area consists of four 12-foot paved travel lanes with 12 foot shoulders. East and west bound lanes are separated by a six foot median containing a three foot high cast-in-place concrete wall. Most cuts start at, or slightly behind the ditchline, leaving only the width of the ditch and shoulder between the slopes and the travel lanes. Mileposts begin at zero on the North Carolina-Tennessee State Line and increase eastwards towards Asheville.

GEOLOGY

Rocks throughout most of the Pigeon River Gorge are late pre-cambrian metasediments belonging to the Snowbird Group of the Occoee Supergroup. Formations exposed, beginning with the oldest, are the Wading Branch Formation, Longarm Quartzite, Roaring Fork Sandstone and the Pigeon Siltstone. With the exception of the Longarm Quartzite, these formations are made up of metamorphosed, interbedded siltstones and sandstones. The Longarm Quartzite, with its high percentages of quartz is easily detected when drilling with diamond bits.

The regional structure is a broad, northeast-southwest trending synclinal fold with its axial trace a short distance west of the State Line. From the State Line eastward, older rock units are exposed as the road crosses the southeast limb of the syncline. Near Mile 12.5, these rocks are overridden by basement gneisses along the Greenbrier Fault.

Discontinuities within the rock units are numerous and well developed. As most of the cut slopes are entirely in rock, stability is controlled by discontinuities in the form of bedding, joints, foliation, minor faults and sheared or weathered zones. Upon close examination of the cuts, every failure mode known to be formed by intersecting discontinuities can be detected.

HISTORY

Construction was first started in the 1950's when a rough, two lane road was graded through the Pigeon River Gorge into Tennessee. This route was later selected as a part of the Interstate system; therefore, construction was begun in the early 1960's to upgrade the highway to Interstate specifications. Access was extremely difficult at the time, resulting in design work being done without benefit of a detailed geotechnical investigation. The project was opened to traffic in mid-October, 1968.

Cost of construction for 12.7 miles through the roughest portion of the project, from the State Line eastward, was $14.5 million. The total quantity of excavation was 11.2 million cubic yards, of which nearly 90 percent was rock. The project included three tunnels which are over 1000 feet in length.

Apparently, many slides occurred during construction, but they are not well documented. Shortly after the roadway opened a local newspaper noted two slides, one involving 250,000 cubic yards of material; the other 180,000 cubic yards. No further details of these slides were given. Even though these figures may be somewhat exaggerated, these slides would have to be considered major.

A series of slides were reported in April of 1972, one in a large colluvial deposit at Milepost One. In March of 1973, another slide at this location temporarily blocked four lanes of the Interstate.

In June of 1973, heavy rains triggered another series of slides, including two major ones. The first, and most serious, could be considered a debris avalanche, and occurred when colluvium in a narrow ravine failed almost instantaneously. The ravine was cleared down to bedrock for a distance of 1500 feet up the mountainside. Debris blocked the roadway for over ten hours and destroyed three automobiles and a large truck. In the second occurrence, approximately 125 feet of the eastbound lanes, supported by an embankment, were lost. This has been the only recorded instance of major embankment failure.

In May of 1978, a slide near Milepost 16 blocked the eastbound lanes for a distance of 600 feet. This failure took nine months to repair and involved removal of 300,000 cubic yards of material. It was this slide that led to the current investigation of cut and fill slopes throughout the North Carolina section of the Pigeon River Gorge.

The incidents described above are only a few of the major rock related events which have occurred over the years. Rockfall within the gorge is almost a daily occurrence. Several small slides are reported each year and a major failure occurs every few years.

THE INVESTIGATION

This investigation was begun in December, 1978 with the purpose of making a survey of stability problems in cut and fill slopes along the Interstate highway system in western North Carolina. As problems were obvious along Interstate 40 through the Pigeon River Gorge, work was concentrated there. The study was modeled after techniques outlined by Piteau (1978) in his course on rock slope engineering.

1. An inspection was made of the first 20 miles of Interstate 40 in order to determine unstable areas. Past, current and potential failures were recorded in order to have an inventory and description of the problems in each slope.

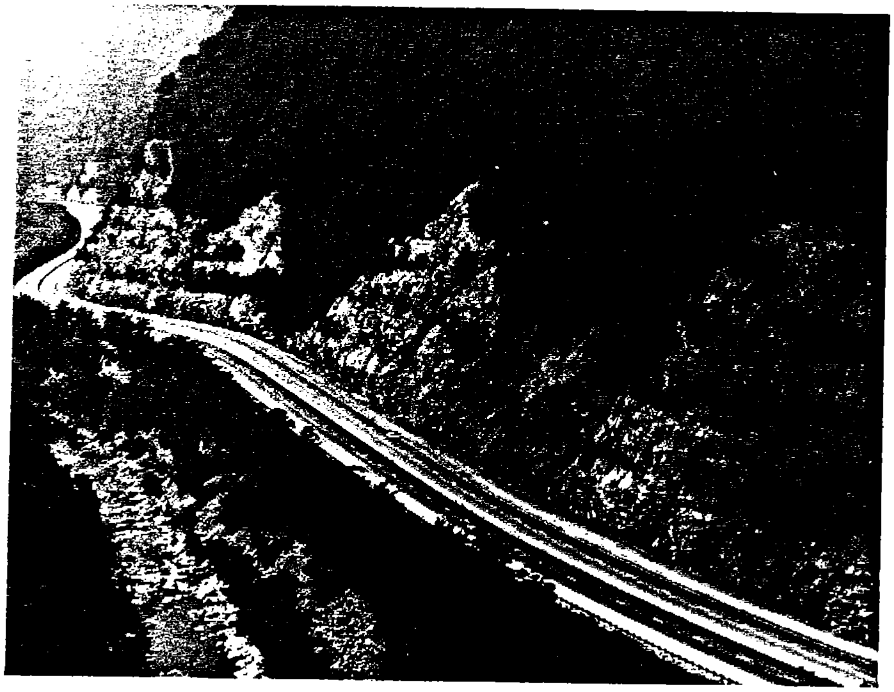

Figure 2. Typical cut and fill sections along Interstate 40 through the Pigeon River Gorge.

Figure 3. An old wedge failure. Unstable material left on the failure planes continues to be a problem.

2. The orientation and characteristics of bedding, foliation, joints and other discontinuities were recorded in the field. This information was processed by computer and major discontinuities in each area were summarized. From this data, failure modes formed by unfavorably orientated discontinuities were identified.

3. Photography was used extensively. A mosiac of each cut was made from photographs taken at grade. These mosiacs proved to be very useful both in the field and in the office. Also, low level oblique photographs were taken from a helicopter. The quality of this photography was such that many stereo-pairs were obtained. Use of a helicopter also allowed inspection of large areas which were relatively inaccessible by foot. Black and white and color aerial photography was also available, from which topographic maps and cross-sections were made.

4. State Highway Patrol reports, newspaper accounts and records or recollections of Department of Transportation personnel who have worked in the area were compiled in a history of rock related incidents occurring over the years. In order to assess the quantity of falling rock and the number of rock related incidents, drivers manning the 24-hour patrol were asked to keep logs of rockfall occurrences and time spent cleaning the roadway of debris.

5. Original plans, cross-sections, maps and reports pertaining to the project were collected from Department of Transportation files.

6. Stability analyses were conducted in several specific areas suspected of being in a state of impending failure. In each case, the boundaries and dimensions of each area were determined; each area was examined for significant features such as tension cracks, shear zones and groundwater seepage; computer selected discontinuities were verified with direct field readings; and simple field tests were performed to determine approximate strength values along the discontinuities. The specific analyses were modeled after Hoek and Bray (1974), Piteau (1978) and the Canadian Pit Slope Manual (1976).

CAUSES OF FAILURE

It would be difficult to assess which factors in the following discussion are the most important relative to stability problems through the Pigeon River Gorge. All factors mentioned contribute to rockfall and slides, and in combination pose some very interesting and difficult problems for engineering geologists.

Discontinuities

Discontinuities within the rock mass have been responsible for failures, both large and small, occurring in roadway cuts along the Pigeon River. Because of the relationship between the regional structure and the roadway alignment, bedding planes dip obliquely out of the cut slopes to form one of the major failure surfaces. When bedding is combined with a prominent, steeply dipping joint set, wedges of unstable rock of varying sizes are formed (Figure 3). Although wedge failures are probably the most common failure mode, the discontinuities are also responsible for rockfall, toppling and planar sliding.

Colluvium

For many thousands of years, extensive deposits of colluvium have collected in the steep, narrow ravines. These deposits were undermined during construction and are now left unsupported, both high around the tops of the cut slopes and at roadway grade. Undermined colluvium has been responsible for failures ranging from minor rockfall to some of the largest slides in the area.

Explosives

At the time Interstate 40 was constructed, blasting was not as well controlled as it is on most projects today. No pre-shear techniques were used and since this was an isolated area, the objective seems to have been to move as much rock as possible with each shot. As a result, rock in the cut faces is shattered, loose and extremely unstable.

Slides

Slides occurring both before and after the Interstate was opened have undermined slopes above many of the cuts. Due to lack of access and adequate equipment, material that did not reach the roadway was left in the slopes. This material now migrates downslope as rockfall and small slides (Figure 4).

Rainfall and the Freeze-Thaw Cycle

Rainfall in the area averages 50 to 55 inches per year, primarily in the winter and spring. As expected, it is during these months that rockfall and slide activity is greatest. Equally important are the freeze-thaw cycles, possibly some of the highest in the United States. Temperatures and precipitation recorded at a climatic station on the North Carolina-Tennessee State Line for the months of October through April are shown in Table 1. In winters occurring between 1976 and 1981, the average number of freeze-thaw cycles was 70, with the greatest number being 78 in 1978-1979.

Table 1. Precipitation and Freeze-Thaw Cycles

Years	Number of Days Temp. 32 F 32 F	Days Temp. 32 F	Precipitation in Inches
1976-1977	77	12	18.65
1977-1978	78	14	24.26
1978-1979	79	3	25.94
1979-1980	53	7	22.78
1980-1981 (Through February)	63	10	13.04

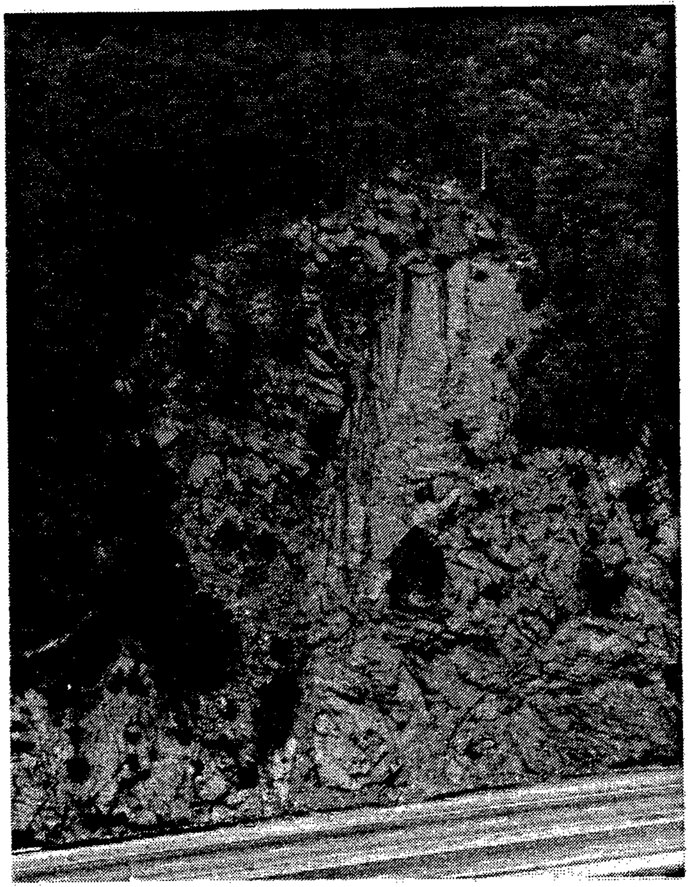

Figure 4. Material undermined by previous failures is a source of rockfall and small slides

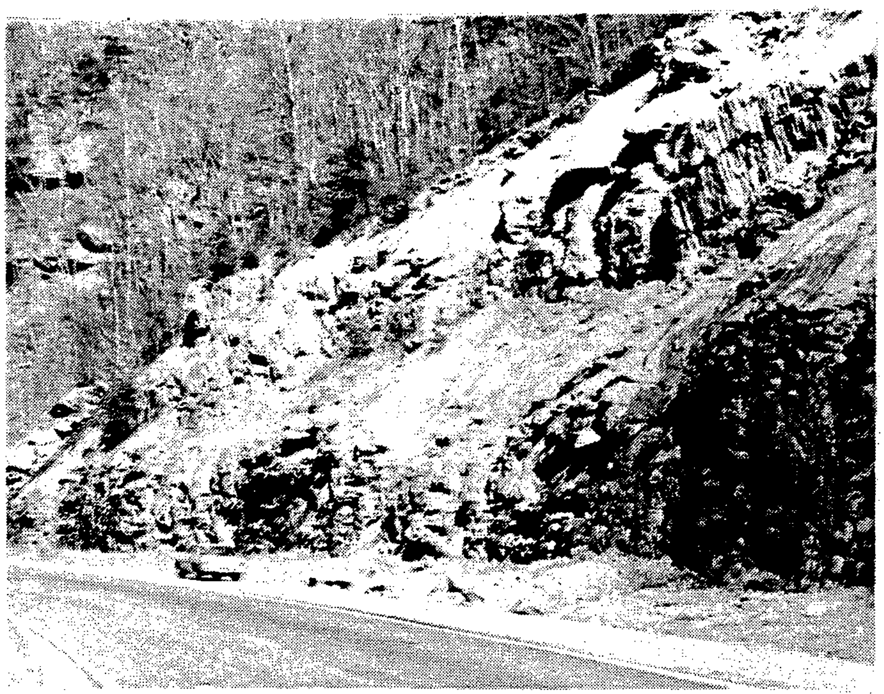

Figure 5. Benches in the original slopes have failed along discontinuities dipping into the roadway.

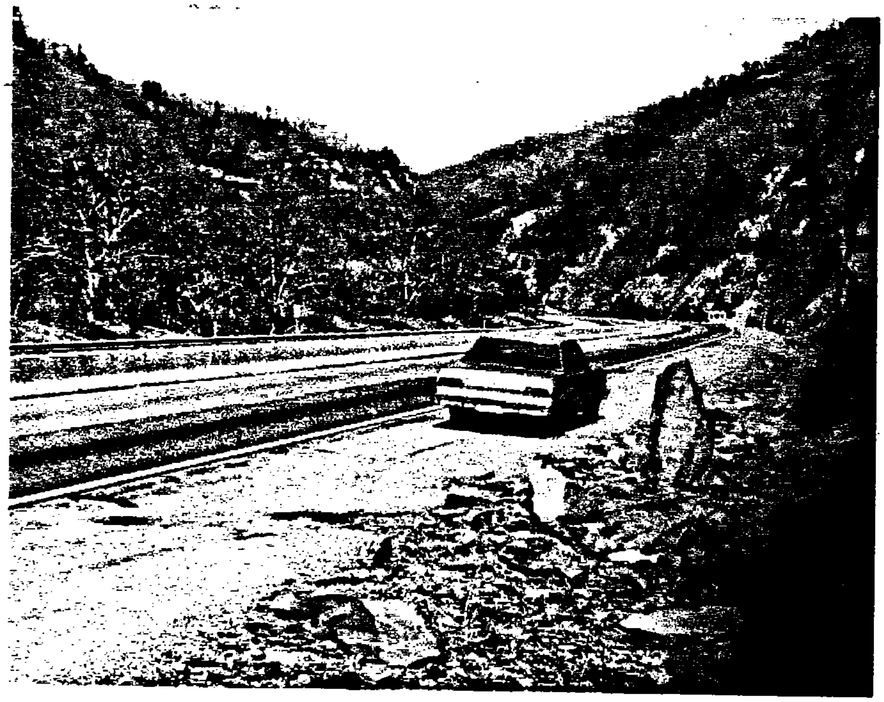

Figure 6. Falling rocks accumulated over a period of about four years.

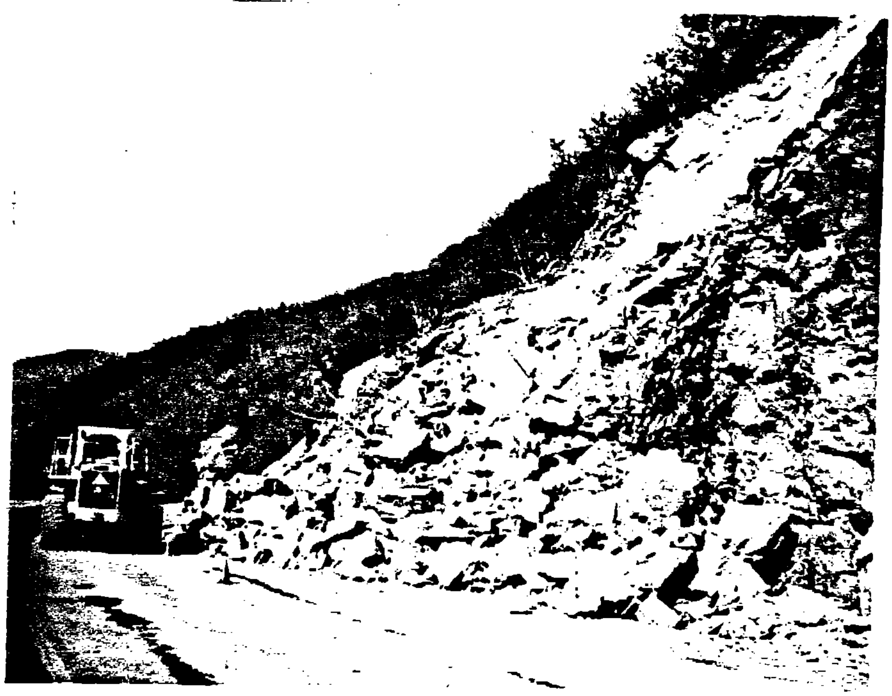

Figure 7. With no catchment area at grade, a relatively small slide can block one or more travel lanes.

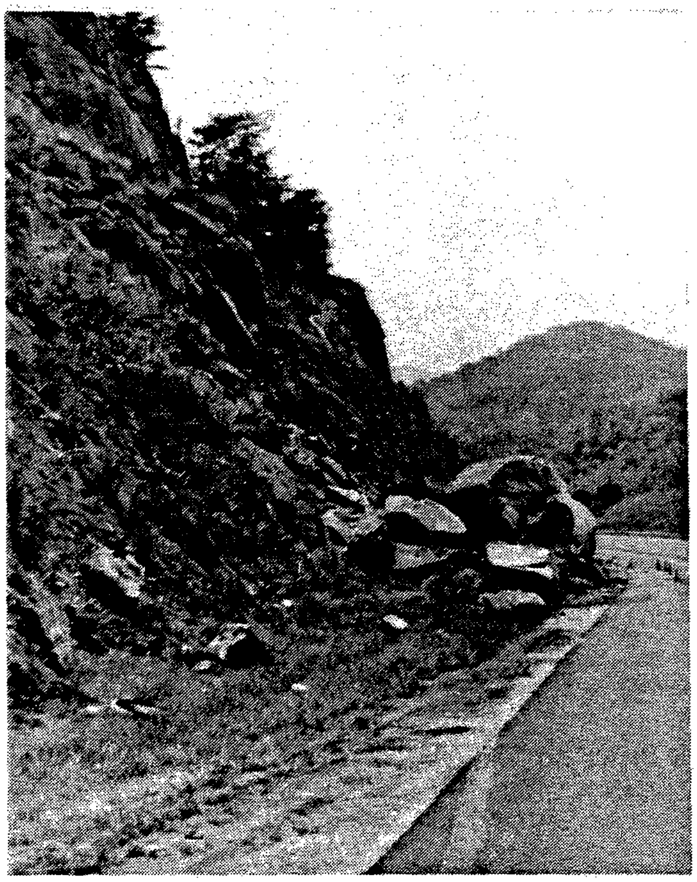

Figure 8. With a catchment area at grade, the majority of material in smaller slides does not reach the travel lanes.

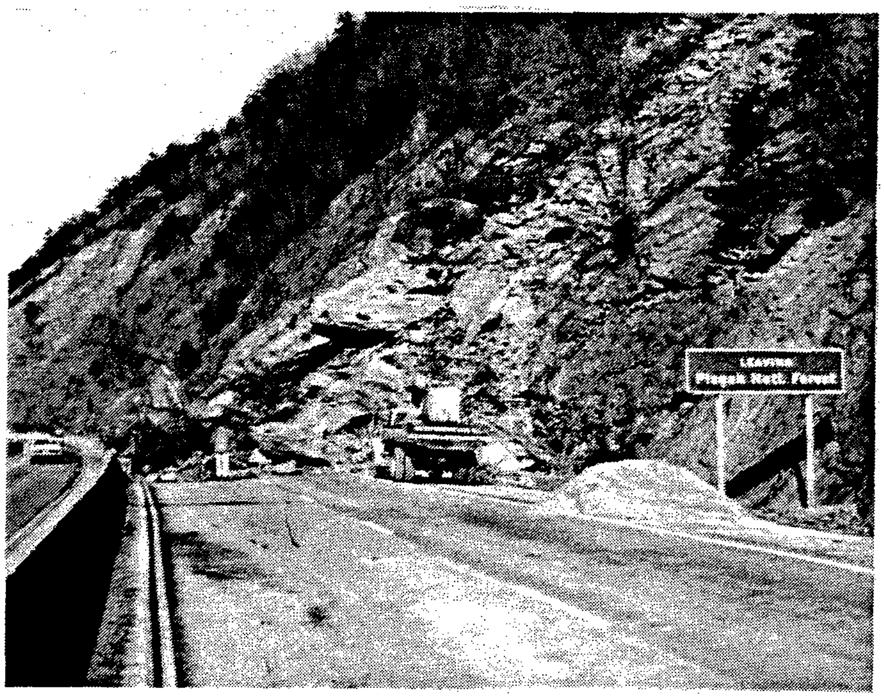

Figure 9. Larger slides can completely block two lanes and spill over the median wall.

Bench Failure

During the planning stages, it was realized that rockfall would be a major problem. In anticipation of this, ten foot wide benches, set at regular intervals, were incorporated into the slope design. No bench at grade was left and the cut slopes started at the ditchline. Unfortunately, most of the bench width was lost due to failures, and that which was left soon filled to overflowing with rubble (Figure 5). Later in the project, the benches were widened to 30 feet, and have proved to be somewhat more adequate.

TYPES OF FAILURE

The most common occurrences are isolated rocks falling from slopes to land on shoulders and travel lanes (Figure 6). Sources of rockfall are the undercut colluvial deposits, loose rock in cut faces and materials remaining at the head of previous slides. Rockfall activity is at the highest during and after rainy periods and when freeze-thaw conditions are prevalent.

Since falling rocks are isolated events, the motorist is often upon one before realizing it, and has no time to react. Damage to vehicles is usually confined to burst tires or damage to the body or undercarriage. However, if the rock is large enough, the driver can lose control, resulting in a serious accident.

Small Slides

These slides may consist of several hundred cubic yards of small to large rock fragments and soil material. The debris generally fills the the ditch and spills onto the roadway, sometimes partially blocking the travel lanes (Figure 7). If, as in some cases, a narrow bench at grade is present, much of the debris is contained on it (Figure 8). Once the travel lanes are cleared, the remaining material is usually pushed aside and left until such time maintenance personnel have the opportunity to remove it.

Large Slides

Massive failures are less frequent, but have in the past closed the Interstate for several hours. Several weeks or months may be required to repair a large slide, while traffic is detoured around the area. Cleanup is generally conducted by State maintenance forces although in one instance the slide was so large the work was contracted out. Slides of this magnitude have been either wedge type failures or failure of large colluvial masses (Figure 9).

STATEMENT OF THE PROBLEM

Once the preliminary study was completed, it was determined problems were twofold. The first concerned the continual rockfall and smaller slides which are usually confined to the lanes adjacent to the cut slopes by the concrete median wall. Second is the potential for large cut slope failures blocking two or more travel lanes. Several areas were identified as having the potential for failures of this magnitude, either as failures in large colluvial deposits or as a wedge of material sliding along two intersecting discontinuities. Although these conditions exist in the cut slopes throughout the length of the gorge, it was decided that problems were the most numerous and of the greatest magnitude beginning at the State Line and continuing generally eastward for four miles. Therefore, it was determined further work would be concentrated in this area.

While planning remedial measures, several factors had to be considered. Access to cut slopes is extremely difficult and very little working room exists at roadway grade. Furthermore, any activity in these cuts would be likely to dislodge unstable debris, presenting a danger to workers and traffic below. Major potential failures could be identified to some degree; however, the slopes are in such poor condition it is impossible to predict in advance the location of the next rockfall incident or slide.

ALTERNATIVE SOLUTIONS

The following alternative remedial measures were considered:

1. Do nothing and accept the maintenance costs. These include the costs of the 24-hour patrol, cleanup of slides and rockfalls, damage to the roadway and damage from rock related accidents. Also included is a yearly average of the cost to clean up major slides occurring since the project was opened. The estimated yearly cost of maintenance approaches $500,000.

2. Redesign of slopes was not seriously considered since it would involve a major construction project, requiring closure of the Interstate. Another drawback to this approach was the enormous quantities of waste material that would be generated with no nearby disposal sites.

3. Scaling, trimming and general slope repair through the use of rock bolts, cables, wire mesh, shotcrete or other methods was also considered. Although these measures could be applied in some localized areas, their use throughout the four mile project is not believed to be economically feasible.

4. As the quantity of material falling from the cut slopes into the travel lanes is so great, relocation of the roadway away from these slopes would seem a reasonable solution. However, any major relocation is restricted by the narrow confines of the Pigeon River Gorge. Some relocation onto the existing fills is possible, but the amount would not be adequate in the event of a major slide.

With these constraints in mind, the tentative recommendations for the first four miles are a combination of slope repair and relocation.

1. Relocate the roadway laterally 24 feet onto the existing embankments to provide a catchment area 30 to 35 feet wide at the toe of the cut slopes for falling rocks and smaller slides. This catchment area should be shaped to contain rockfall and an energy absorbing barrier should be

installed along the shoulder's edge. The relocation would involve about three of the four miles within the study area.

2. Scale and trim obvious problem areas in the cut slopes.

3. Investigate in more detail, those areas where the potential for massive sliding is suspected. Develop remedial measures for these areas while the relocation is underway. Corrections would be implemented once the roadway shift is completed.

CURRENT WORK

The success of these proposals depends on the ability of the existing fills to support the relocated roadway. The fills along the proposed relocation are 80 to 100 feet high, with slope angles of 40 to 45 degrees. In order to sample the fill material, core holes were drilled at selected locations. These holes indicate the fills consist of small to large rock fragments, with little soil infilling. At most locations, a zone of colluvium is present between the bottom of the fill and the top of bedrock, indicating that no undercutting was done before emplacement of fill material (Figure 10).

In order to monitor stability of the fills, slope inclinometers were installed in the drill holes. Furthermore, surveying techniques using a series of permanent monuments and hubs or iron pins, were utilized to gain additional information between the inclinometers.

The fills were inspected for signs of distress, and although some tension cracks were found on the surface, there are no indications of serious impending failures. Along much of the proposed relocation, excess material was wasted on the fill side by constructing berms ten to twelve feet high. Removal of this material will probably improve the stability of the fills. It was also determined that the majority of the cross pipes will have to be lengthened and additional fill emplaced around them to accommodate the relocation. A few of these locations may require some type of retaining wall.

On the cut side, targets were set in some of the obviously unstable areas. These targets are monitored periodically for changes indicating movement, by triangulation and electronic distance measuring techniques. The condition of the cut slopes is such that all areas of suspected instability could not be covered in this manner, however, data available to date has been interesting. At one location indicating a cummulative change, the target and the rock it was attached to eventually fell.

To assess the potential for massive cut slope failures, the State is now initiating a drilling program to sample and place instruments in some of the critical wedges and colluvial deposits. The objective of this program is to more clearly define the problems in these areas and determine if and what type of remedial measures are necessary. This work will be expensive, difficult and dangerous, but worth the effort if a major slide can be prevented.

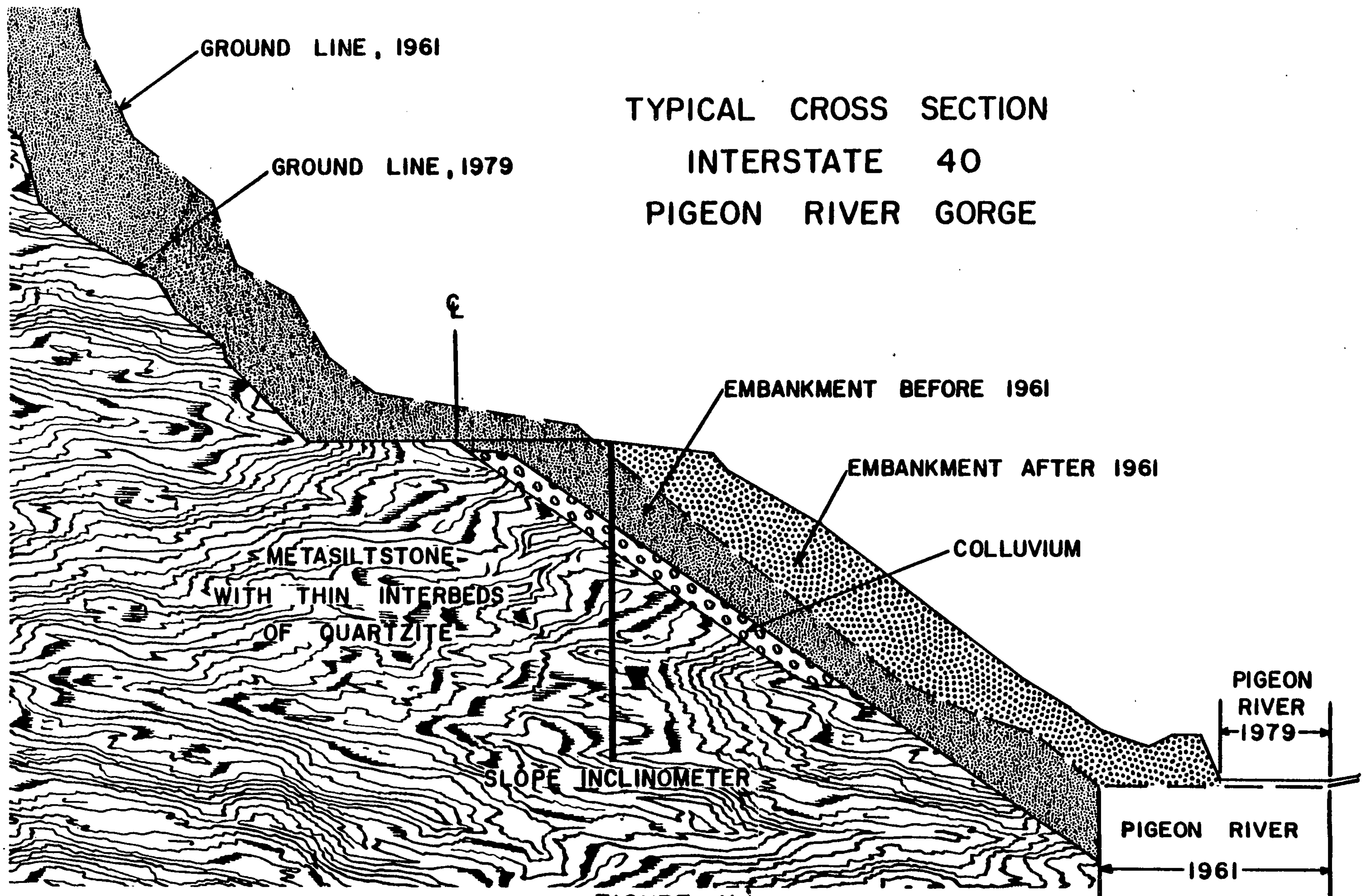
TYPICAL CROSS SECTION
INTERSTATE 40
PIGEON RIVER GORGE
GROUND LINE, 1961
GROUND LINE, 1979
℄
EMBANKMENT BEFORE 1961
EMBANKMENT AFTER 1961
COLLUVIUM
METASILTSTONE
WITH THIN INTERBEDS
OF QUARTZITE
SLOPE INCLINOMETER
PIGEON
RIVER
1979
PIGEON RIVER
1961

FIGURE II

SUMMARY

Interstate 40 through the Pigeon River Gorge has had a long history of stability problems. Due to the difficulty involved in defining and correcting problems, policy in the past has been to deal with each event after it occurs. This study has been an effort to recognize potentially dangerous situations within the area and to recommend remedial measures. This objective was approached by utilizing techniques recently developed in the field of rock slope engineering to assess and improve the stability of a particular slope. It is hoped this systematic approach to stability problems, both on this project and along other North Carolina highways, will be continued in the future.

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THE HARTFORD SLIDE - A CASE HISTORY

BY

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The Great Smoky Mountains have long formed a physical barrier to east-west travel between Tennessee and North Carolina. The mountains are very rugged with steep slopes and many narrow gorges. Construction has been difficult for the few roads that have been constructed, and maintenance has been a continuing problem particularly with respect to rock slides. This paper discusses a slide near the western edge of the Smokies that has occupied the Tennessee Department of Transportation for about 15 years in maintenance and repairs.

The slide location is in Cocke County, Tennessee just east of the Hartford Interchange on Interstate 40 (Figure 1). Through this section of the Smoky Mountains, I-40 follows the Pigeon River gorge. Original ground slopes in the area of the slide were approximately 1½ (H) : 1 (V) or steeper from the river elevation of 1255 up to about elevation 1520. A flatter section then occurred to about 1600, then very steep slopes rose to the ridge crest elevation of about 1985. In all, approximately 730 ft. of relief occur in a horizontal distance of about 1500 ft. in this area.

HISTORICAL BACKGROUND

Prior to construction of Interstate 40, the Tennessee and North Carolina railroad followed the Pigeon River along the same general route as the highway later took. This railroad was constructed in the early 1920's and had numerous rock slides. The route was abandoned in the 1930's.

The construction of I-40 began in 1962. The design called for a combination of cut slopes into the mountain and fill slopes down to the river. Cut slopes were designed as 1/4 : 1 rock cuts. Just after grading reached approximately the final subgrade elevation, a cut slope failure occurred in the general vicinity of the landslide. Examination of the failure by the Tennessee Department of Transportion revealed that materials through this area were not rock, and the design was changed to 1 : 1 single benched slope through the slide zone.

Shortly after the highway was opened to traffic in 1964, problems began occurring in the west bound lane. The pavement section rose upward and had to be graded down and repaved three to five times a year. No problems were noted with the east bound lane through the slide section. Throughout this section, the east bound lane was constructed entirely on fill material with the west bound lane being entirely on original ground.

In 1969, the maintenance problems had become severe enough to warrant investigation by the Tennessee DOT and by the Federal Highway Administration (then Bureau of Public Roads). The FHWA study was done by R. Woodward Moore who used constant depth electrical resistivity traverses. Figure 2 illustrates locations of Mr. Moore's resistivity traverses with data interpretations which were made by Law Engineering Testing Company during our study of the slide area. Resistivity contours are shown at 70 to 80 ft. depth intervals. Of interest is the zone of low resistivity shown by the contours. In a report prepared by Moore (4) for the Tennessee DOT, the data were interpreted to show a series of relatively shallow planar slip surfaces rather than a single deep-lying slip circle or large continuous mass movement.

The DOT study was directed by Jim Aycock, Regional Engineering Geologist, and included a site reconnaissance and borings at the road level. Results of the borings are shown on Figure 3. The reconnaissance of the area uphill from the highway found signs indicating large mass movements such as scarps, ground cracks, and a crack in the bench. These observations, suggesting that a large mass was involved in some type of movement, were contrary to the small observed effect at the road and the FHWA conclusion. In the report, Aycock (1) reached the following tentative conclusions based on the results of the DOT borings, the site reconnaissance, and general geologic information:

1. The area involved in the problem zone is an ancient slide zone;
2. The material involved in the moving mass is weathered, highly fractured rock, soil, and detritus;
3. Movements are essentially planar;
4. There must be some type of confining layer near the Pigeon River that restricts movement of the slide mass; and
5. There was a definite need for more information.

Law Engineering Testing Company was retained by the Tennessee Department of Transportation in 1971 to provide detailed study of the slide zone and to recommend possible remedial measures for consideration by the DOT. The remainder of this paper describes the findings of the further study and discusses the remedial measures finally used.

EXPLORATION ACTIVITIES

Information about the slide area was obtained by reconnaissance by a geologist and engineer, exploratory soil test and rock core borings, establishment of inclinometers in selected borings, setting of piezometers in selected borings, and by utilizing a survey grid system for estimating rates of slide movement. The grid system was established by the DOT in 1969 and had been periodically monitored since that date, giving about 2 years of movement data. Figure 4 shows locations of borings along with original contours.

Surface features were mapped using the grid system for reference. Cracks, scarps, and seepage zones were particularly noted and the major features are shown on Figure 5. Well above the cut slope intersection with original ground, several broad depressions were found in a flatter portion of the topography. Ponded rain water was collected in these depressions and was assumed to be feeding some of the seepage zones noted on the face of the cut.

AREA AND SITE GEOLOGY

Regional geologic information was obtained from the published geologic map of Tennessee and from Hadley and Goldsmith (2) as well as by reconnaissance along I-40. The area geology is shown on Figure 1. The rock units in the vicinity of the site are an assemblage of slightly metamophosed siltstones, sandstones, and shales of the Ocoee series. The rock sampled in core borings and observed in outcrops appeared to best match published descriptions for the Pigeon siltstone and Roaring Fork sandstone formations. The unweathered rock at the site is hard, bluish gray to bluish green, siltstone which contains a few thin layers of phyllite and fine sandstone.

The slide area is located within a section of Tennessee that has numerous ancient faults which are now inactive. The major faults trend in a northeast-southwest direction, and it is generally accepted that most of these faults are inclined down to the southeast. Rocks on the southeastern side of the faults typically were moved up and toward the northwest. Movements along a fault usually occurred along a fault zone as opposed to a single plane of displacement. From data obtained during our study of the area, we concluded that a fault zone is located either within the landslide area or just northeast of the slide area.

In the landslide area, faulting is manifested by an enhanced joint system and an apparent disruption of the bedding orientation. Vertical jointing is well developed along three directions. The best developed joint set is oriented perpendicular to the roadway, and the other two sets are aligned at about 45 degrees to the roadway as shown on Figure 6. The spacings between joints observed in the road cut at the landslide ranged from 2 inches to 8 feet, but most joints were 1 to 3 feet apart.

The slight metamorphism of the rock tended to obscure the bedding. Bedding orientation data obtained indicates that a change in the rock dip occurs in the slide area. The differences in orientation of the bedding can be attributed either to tight folding, faulting, or both. Figure 6 shows bedding observations and the interpreted axis of a probable fold or fault.

At the east end of the slide area where the 1 : 1 soil cut transitions to a 1/4 : 1 relatively stable rock cut area, slickensides were observed along the rock faces supporting earlier observations of past movements.

SUBSURFACE CONDITIONS

A typical geologic section prepared from the boring information is shown in Figure 7. Above the cut section, the borings encountered a sandy SILT residual soil with sandstone fragments. Below the residual soil, a zone of slightly fractured rock alternating between soft to moderately hard, tan, weathered, fine-grained sandstone and hard, bluish-green siltstone was encountered. Breakage of the core along iron-stained joints was common with fracture spacings ranging from 1 inch or less in severe breakage zones to 1 foot or more in less broken zones. The thickness of the broken zone ranged from about 30 feet near the head of the slide area to about 75 feet at the bench level. The material exposed in the cut was typical of the broken rock zones.

With increasing depth, the broken rock zone graded into a zone of badly weathered and fractured rock containing soil seams and rock fragments in a soil matrix. Open zones of 12 to 18 inches thick were encountered in two borings. The thickness of the badly weathered zone ranged from about 10 feet in the borings near the head of slide to more than 95 feet in borings at the bench level.

Most of the borings penetrated through the badly weathered zone into very hard, unweathered, bluish-green to bluish-gray siltstone. The characteristic feature of this rock was the presence of thin, wavy carbonate quartz veins which were rarely present in any other rock. The hard, unweathered rock was encountered at depths ranging from 16 feet at the road level to 170 feet at the bench level, and at a depth greater than 100 feet in all borings above the top of the man-made cut. Note on Figure 7 how the hard, unweathered rock rises toward the highway median.

Water levels shown on Figure 7 were obtained from interpretation of levels in piezometers. Locally higher water levels were indicated in the slide zone as shown by seepage from the slope faces. Most of the higher water was interpreted as rainfall travel through joint patterns and seepage from the ponded water in the depressions above the cut slope area.

SLIDE MOVEMENTS

Figure 8 shows the pattern and magnitude of surface movements obtained from monitoring survey grids. As much as 14 feet of movement occurred during the monitoring period with 4 to 8 feet of movement common. Rates of movement correlated well with rainfall records with a rate of about 12 feet per year observed in the late winter and early spring. This rate decreased to about 2 to 3 feet per year in the drier summer weather.

Inclinometers established in the borings showed definite signs of movement occurring in the badly weathered zone. Depths of recorded movement ranged from 16 feet at the ditch line of the west bound lane to 140 feet at the bench level. Movements of magnitude sufficient to block the inclinometer casing occurred in several borings.

From the pattern of movement observed in the inclinometers as indicated on Figure 7, it was apparent that a large mass of material was moving as a unit with movement occurring throughout a broad weak zone - the badly weathered rock mass. The moving mass is forced upwards by the competent underlying rock which rises near the Pigeon River forming in effect, a large "sliding board". The coincidental location of the median center line above the highest sound rock location restricted the observed problems to the west bound lane only.

STABILITY ANALYSIS

In order to evaluate remedial measures, it was necessary to quantify the apparent stability of the moving mass so that the relative effects of different remedial measures could be compared. Due to the heterogeneity of the materials in the badly weathered rock zone where movements were occurring, it was not possible to obtain undisturbed samples for conventional laboratory testing. A relatively intact sample of the badly weathered zone was recovered by coring and was tested in the laboratory in triaxial shear. The Menard Pressuremeter was used in one boring in an attempt to measure strength in-situ. Menard (3) has established a method of computing soil strength from the measured pressuremeter modulus. Strength values computed from the field test data were higher than the laboratory tests on the weak soil seam tested.

After review of all data obtained, strength parameters were selected as shown on Figure 7. Stability analyses were run using the modified Bishop method and the basic stratrigraphy and water level information shown on Figure 7. Although it was recognized that the actual failure surface was probably a combination of curved and planar surfaces, large radius circles were used as an approximation. Figure 7 shows a circular surface which was calculated to have a factor of safety of 1.09. This surface was used as a basis for comparing the effect of various remedial alternates.

REMEDIAL ALTERNATES

A number of alternates were considered and evaluated for their effect on stability and their probable construction costs. The alternates considered and their features are discussed briefly in the following subsections:

Drainage: Lowering of the water level was expected to improve the computed factor of safety and, if nothing else, reduce the rate of slide movement. Because there were no horizontal drain contractors in the east in 1971, a system of vertical wells intersected by a few horizontally drilled outlets was considered as shown on Figure 9. For this system, only a minor reduction in water level would be possible. As shown on Figure 9, the factor of safety was increased by only about 10 percent at an estimated cost of about $230,000.00.

Unloading: Removing weight from the top of the slide to reduce the driving force was evaluated for both a minimum acceptable removal and a maximum feasible removal. Major drawbacks to unloading were locating a disposal area for the material and the detrimental environmental effects. Figures 10 and 11 show the two removal options. Minimum removal, about 200,000 cubic yards, increased the factor of safety about 10 percent at an estimated cost of $500,000.00. Combining minimum removal with drainage increased the factor of safety by about 15 percent. Maximum removal would have required about 650,000 cubic yards at an estimated cost of $1,300.000,00 and increased the computed factor of safety by approximately 15 percent. Combining drainage with the maximum removal resulted in about a 20 percent increase in the factor of safety.

Road Relocation: Bridge and crib retaining wall alternates to move the highway into the Pigeon River, thus, bypassing the effects of the slide were considered. Both of these options would move the west bound lane to stable ground but at relatively high costs.

Summary: Figure 12 illustrates the factor of safety increase and cost associated with the various remedial measures considered. After discussing the various alternates with the Tennessee Department of Transportion, the decision was made by the DOT to continue maintaining the highway while further studying possible drainage solutions.

REMEDIAL HISTORY

In 1972, the DOT installed three vertical wells with submersible pumps along the west bound lane ditch line. The pumps lowered the groundwater from about 7 feet before pumping to about 14 to 20 feet below ground after pumping. Over the next year, only one uplift of the west bound lane pavement occurred requiring maintenance work. The observed uplift occurred after 3.5 inches of rainfall.

Encouraged by the effect water control had on the slide movement rate, the DOT explored use of horizontal drains. In 1973, a horizontal drain contractor had come to Tennessee for work on landslide corrections on I-75. The contractor attempted to drill some trial horizontal drains from the road level while on his way to a project in North Carolina. With conventional equipment, the contractor could only advance drains 55 feet into the badly weathered and broken rock zone. Even so, at 55 feet of penetration, a 10 gallon per minute flow was obtained.

The results of the horizontal drain trial indicated that conventional equipment then available would not allow adequate penetration of drains through the badly broken and weathered rock zone to reach depths that would result in effective lowering of the water table. Therefore, eight new vertical wells were added during the winter of 1975-76. The vertical well pumping continued to show a reduction in rate of slide movement from that historically noted.

In 1976, new types of drilling equipment became available to horizontal drain contractors. The equipment was heavier and contained a downhole hammer which the contractors felt would enable better penetration of the weathered and broken rock zone. A trial made at road level was able to penetrate 350 feet into the cut and obtain flows of 15 to 60 gallons per minute. Based upon the success of this trial, the Tennessee DOT established a horizontal drainage plan as shown in Figure 13. The plan required three rows of drains, one advanced from the road level, one from the bench level, and one advanced from drilling platforms near the level of the Pigeon River.

The drain installation was let to contract and the work was completed by Jensen Drilling Company in August 1979. Seventy-six drains totalling 19,555 linear feet were installed at a contract cost of $550,000.00. Based on favorable flow rates from the drains at the river and road levels, the drains at the bench level were deleted from the work. No significant problems have been observed in the slide area since completion of the drains and most of the drains are still functioning.

CONCLUSION

The Hartford Slide illustrates a slow, creep type movement which can often be found in weathered rock and soil zones. Sensitivity of the slide mass to fluctuations in the water level is also common. In contrast to the slow creep type movements of the Hartford Slide, many sections of I-40 through the Smoky Mountains have experienced sudden large rock falls and slides along joint planes.

The Hartford Slide also illustrates that relatively inexpensive means to control water level can reduce maintenance costs and obtain time for study of alternate solutions. Improvements in available technology ultimately allowed construction of a cost effective remedial scheme and one which resulted in minimal disruption to the environment.

ACKNOWLEDGEMENTS

Numerous individuals and agencies have worked on the Hartford Slide over a number of years. The reports, pictures, and discussions which these people have had with the writer during his association with the project have been most helpful. In particular, the assistance of Jim Aycock, Regional Engineering Geologist, Tennessee DOT, and David Royster, Director of the Soil and Geology Division, Tennessee DOT have been extremely useful. The permission of Royster to utilize some statistics and data taken from his files, and the loan of slides taken by Aycock and Royster is gratefully acknowledged.

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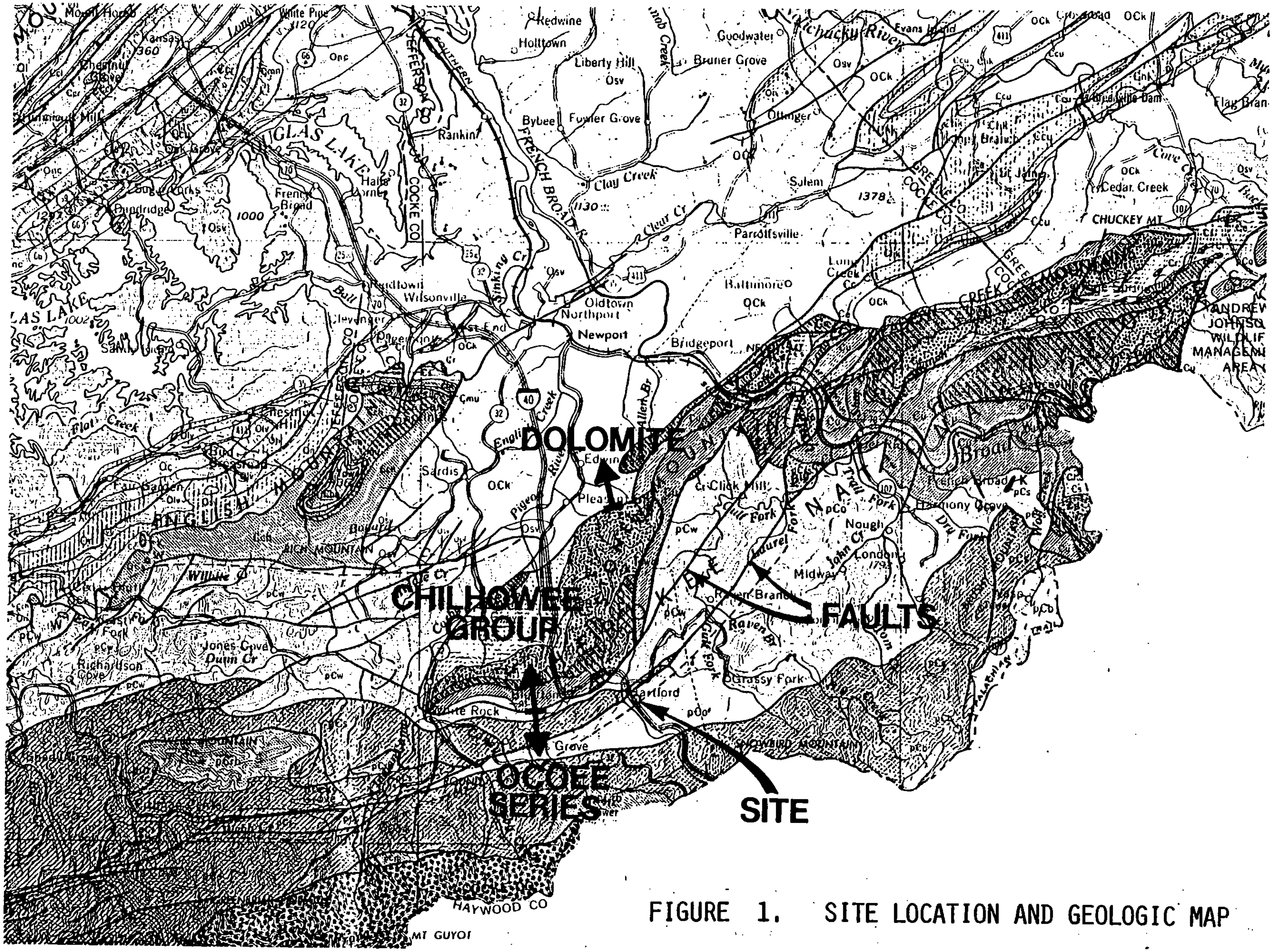


FIGURE 1. SITE LOCATION AND GEOLOGIC MAP

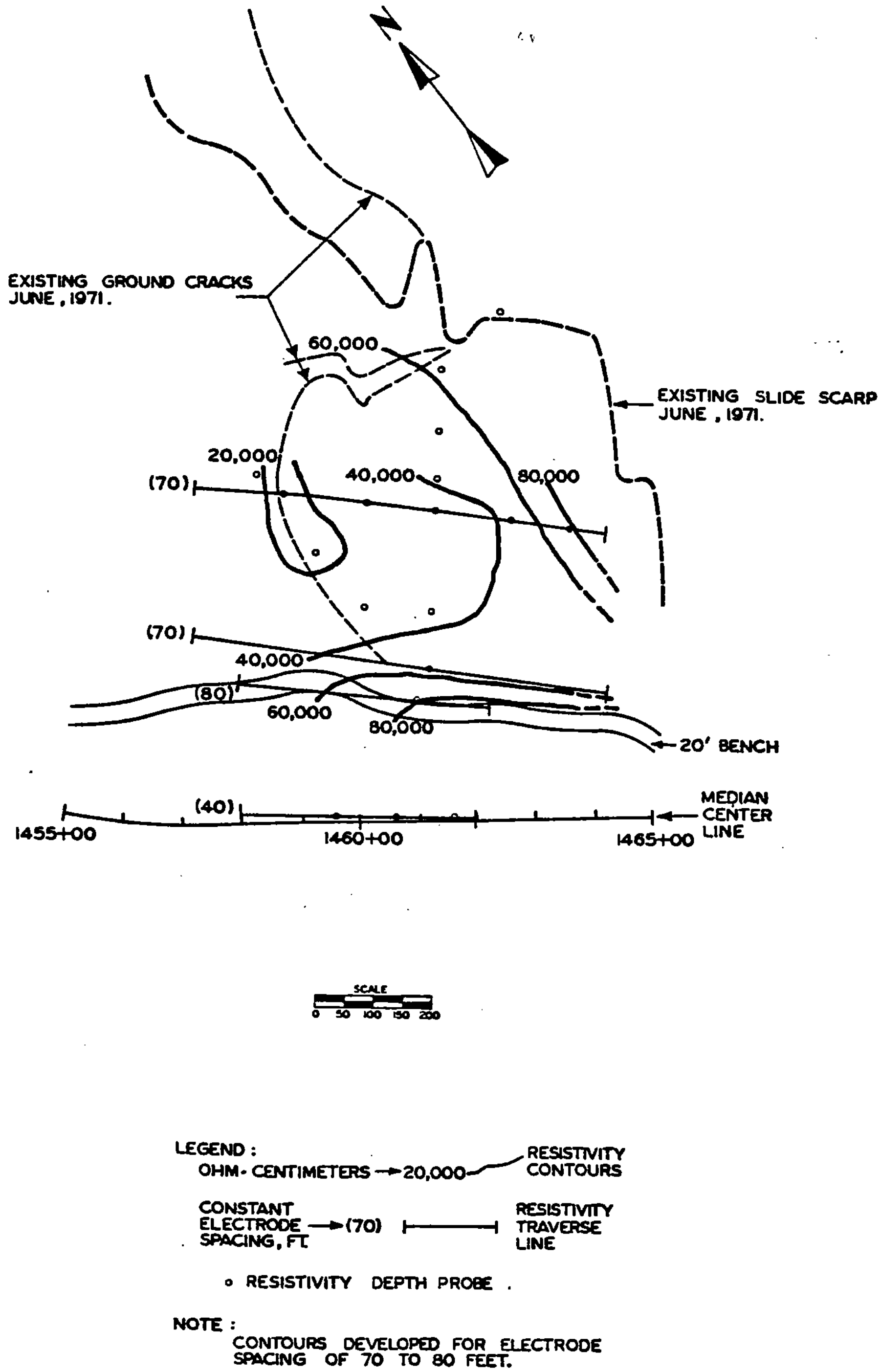


FIGURE 2. ELECTRICAL RESISTIVITY CONTOURS

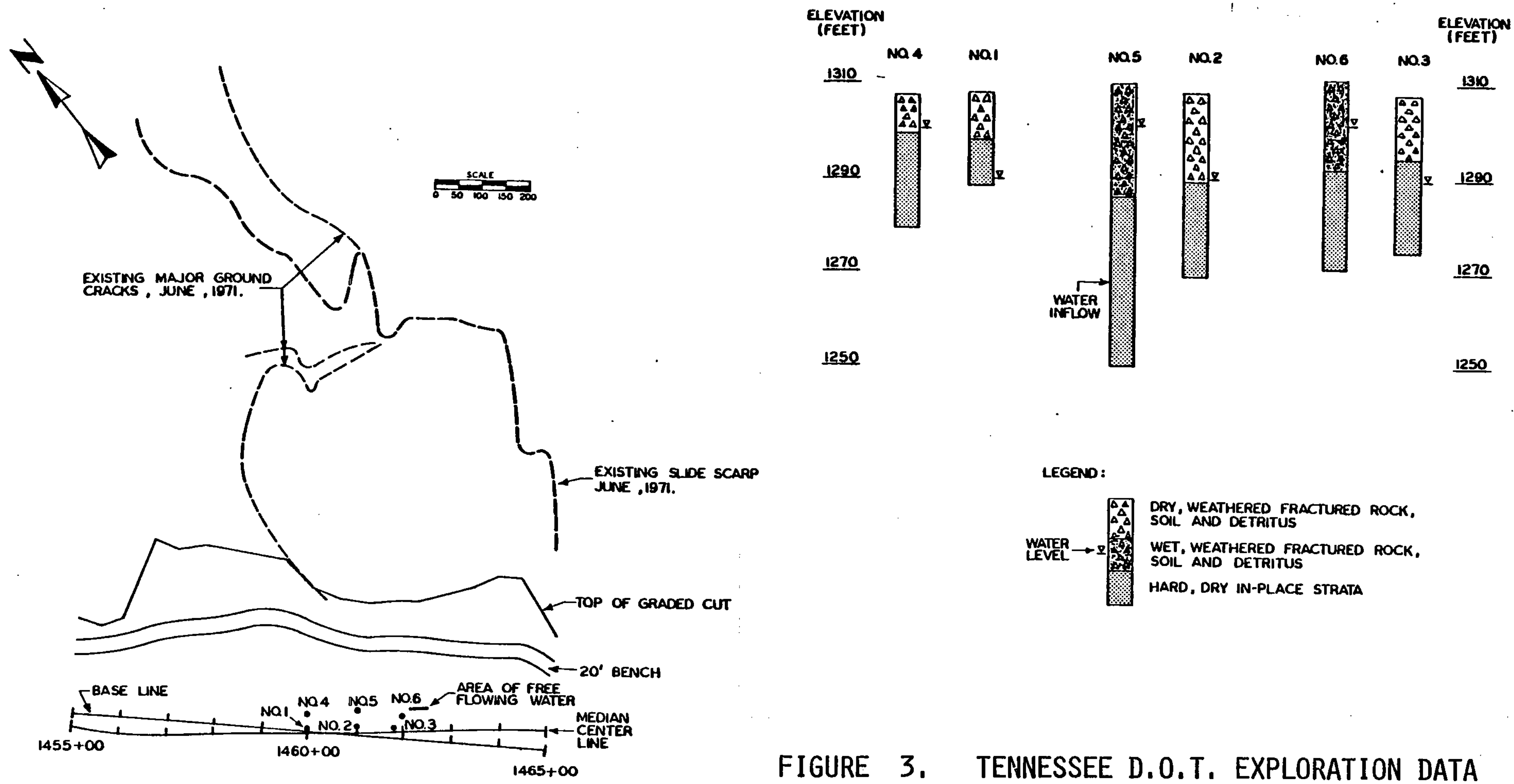
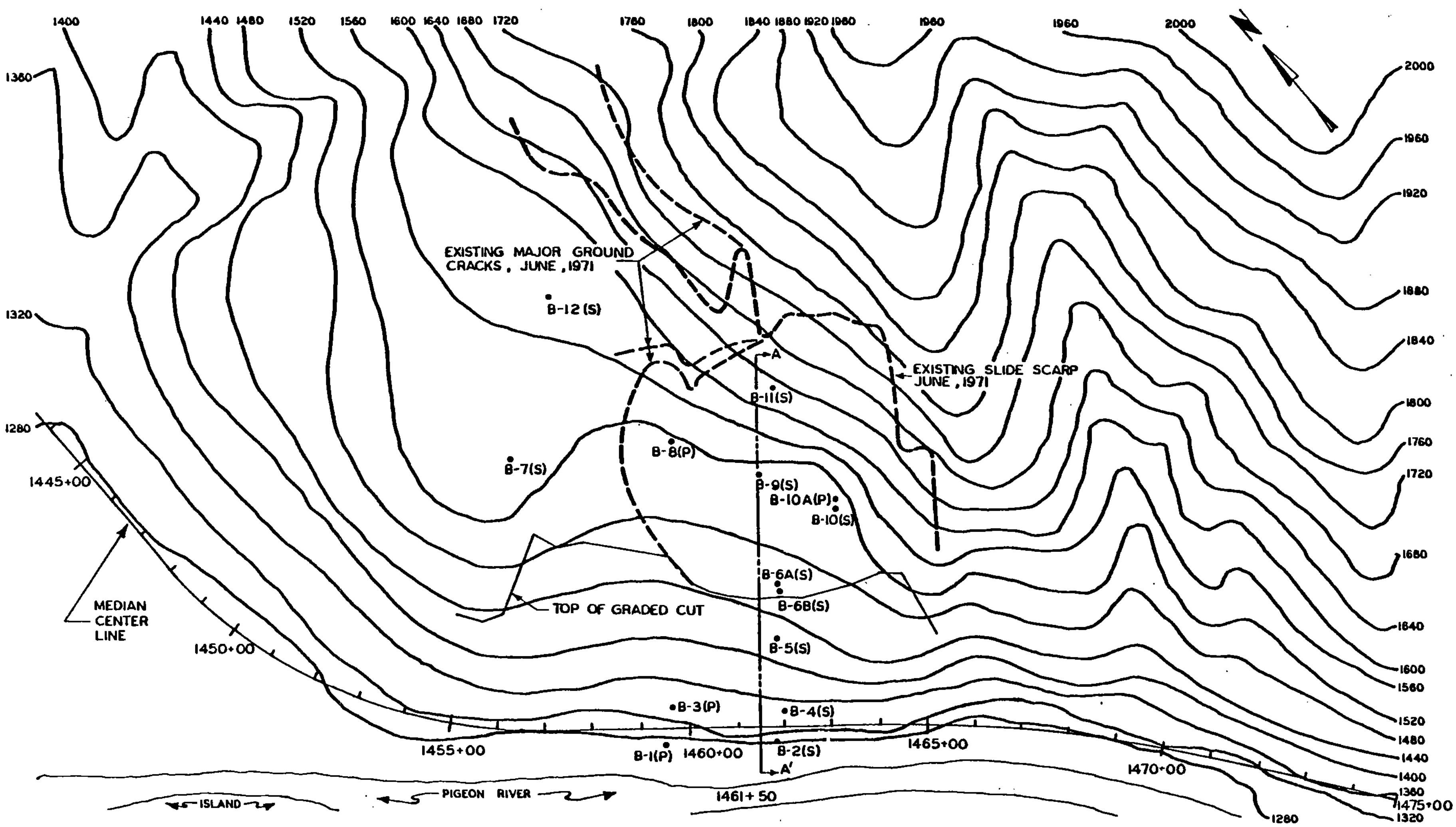


FIGURE 3. TENNESSEE D.O.T. EXPLORATION DATA



NOTE:

(1) THE GROUND CONTOURS SHOWN REPRESENT THE TERRAIN BEFORE CONSTRUCTION AND WERE TAKEN FROM MAPS SUPPLIED BY THE STATE OF TENNESSEE DEPARTMENT OF HIGHWAYS.

LEGEND:

B SERIES - LAWAUGER - CORE BORINGS (LOCATED BY SURVEY, SUMMER, 1971)

(S) SLOPE INDICATOR INSTALLATION

(P) PIEZOMETER INSTALLATION

FIGURE 4. ORIGINAL CONTOURS AND BORING LOCATIONS

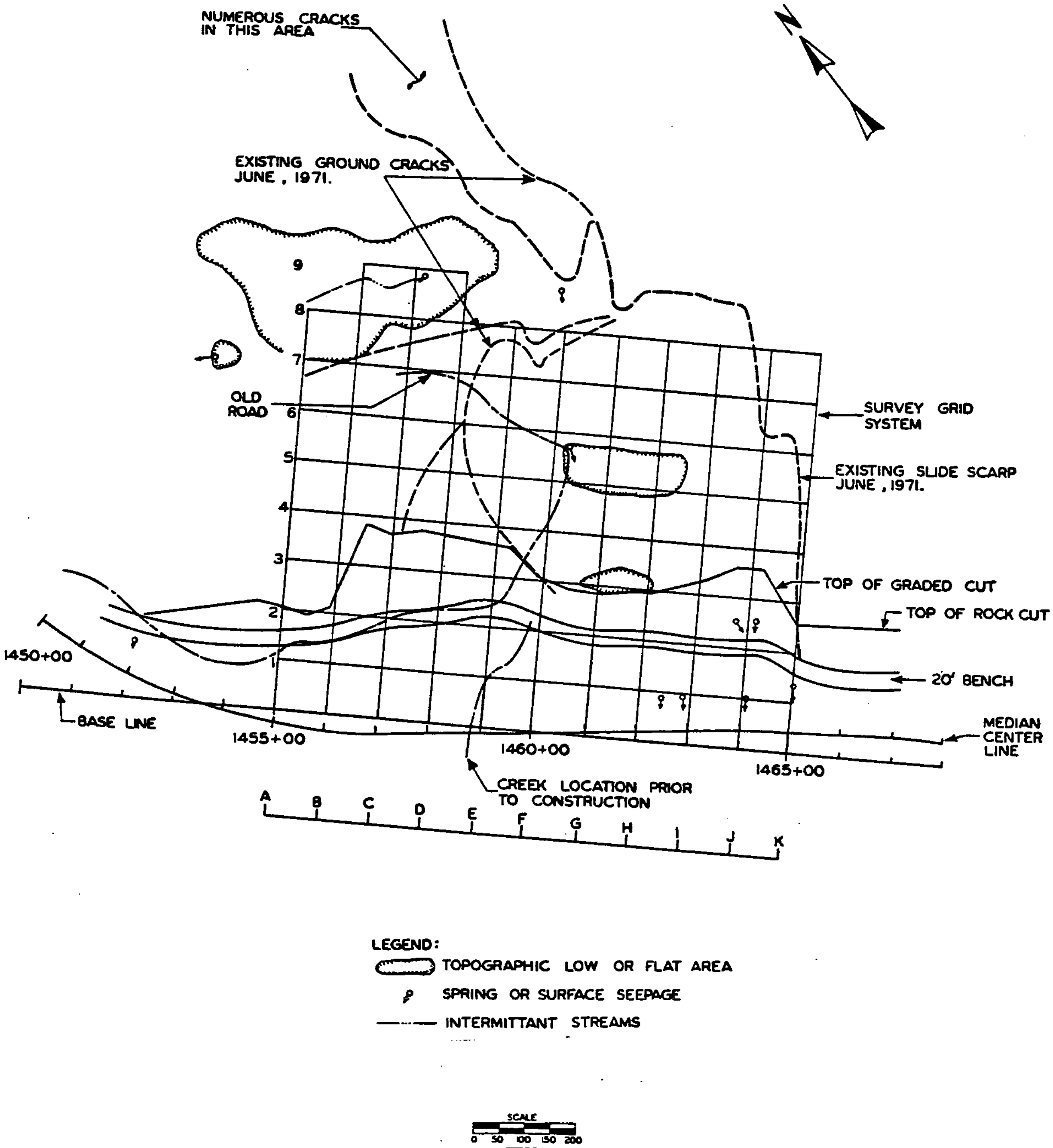


FIGURE 5. SLIDE SURFACE FEATURES

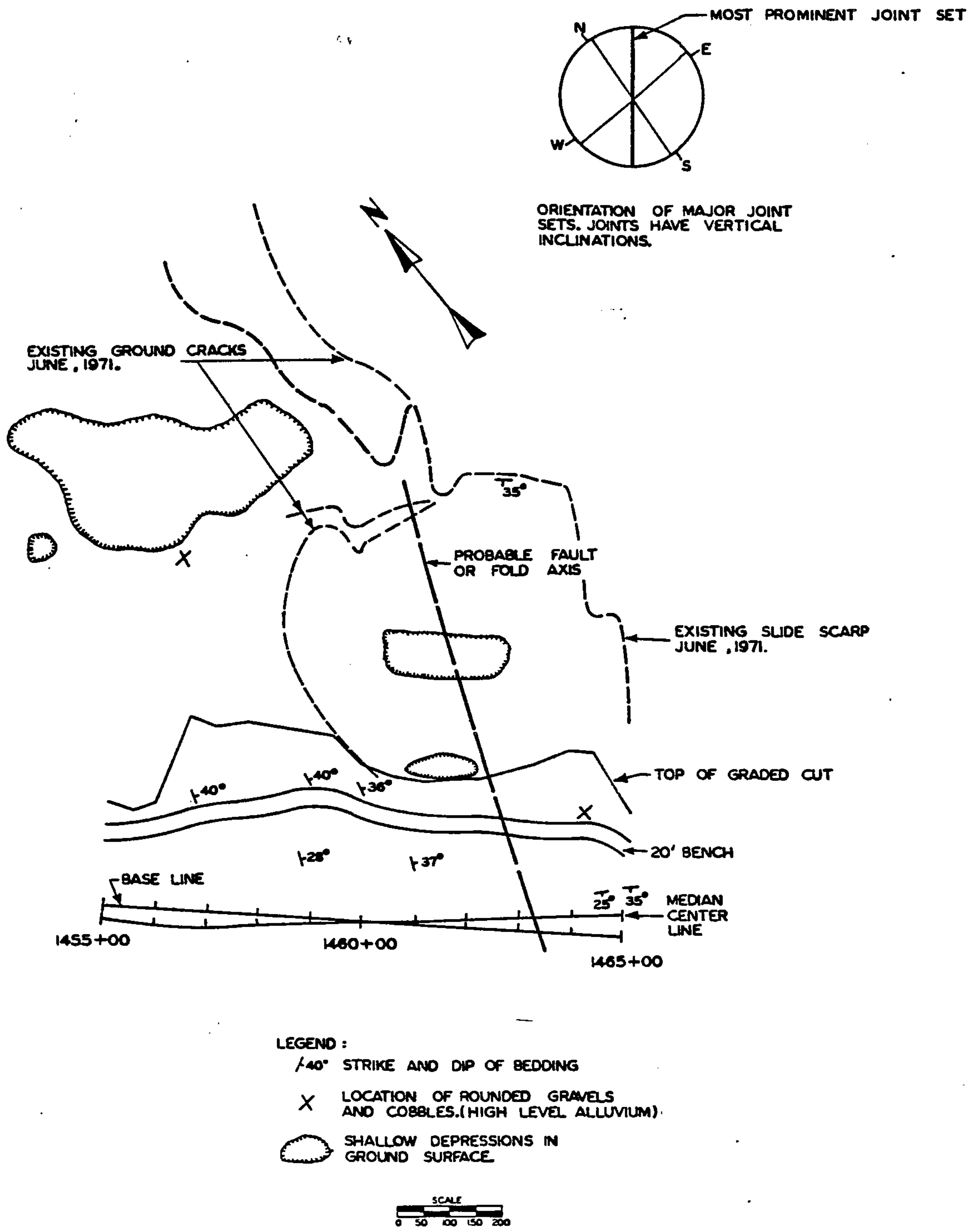
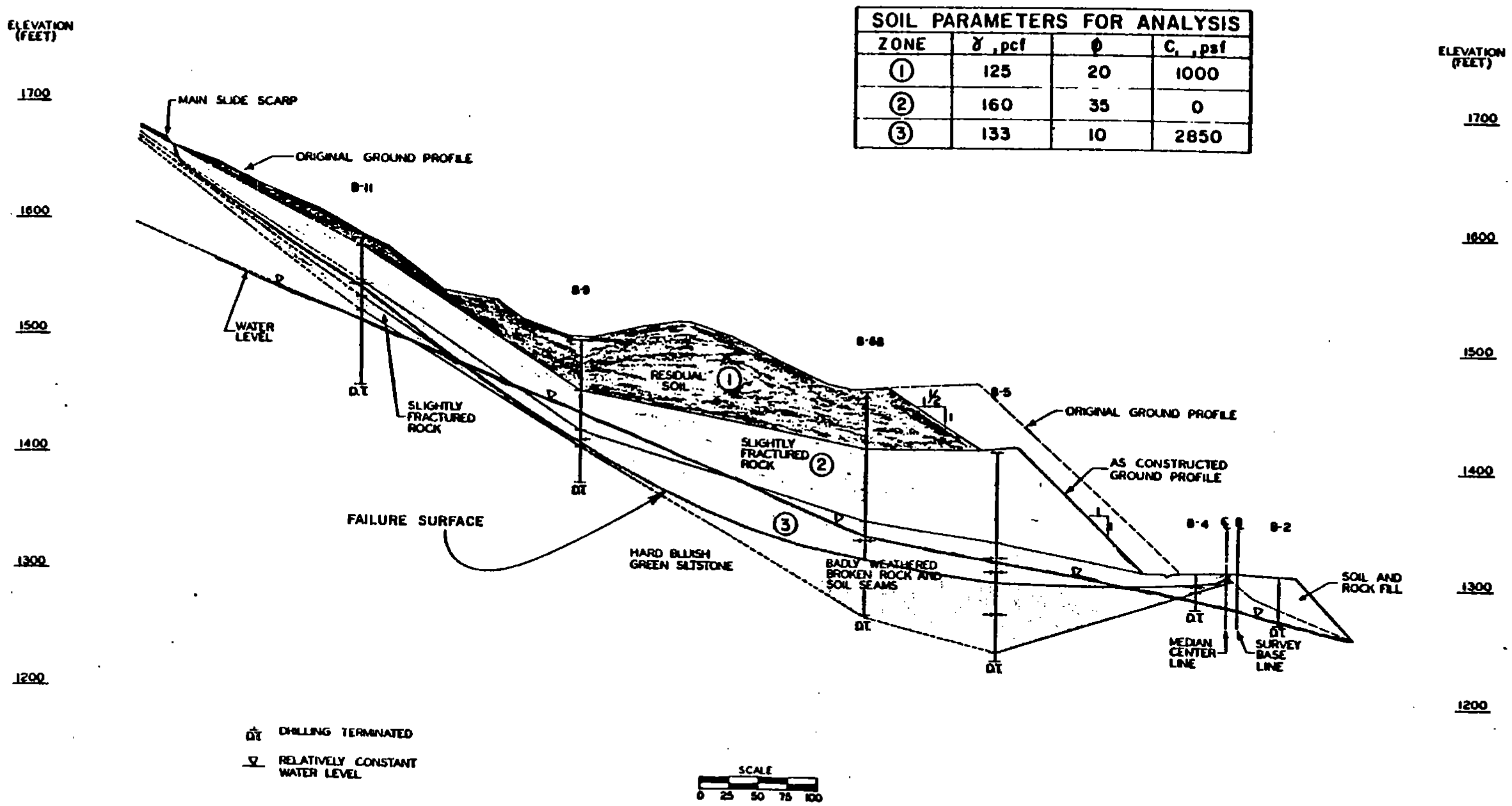


FIGURE 6. STRUCTURAL GEOLOGIC FEATURES OF SLIDE



SOIL PARAMETERS FOR ANALYSIS			
ZONE	γ, pcf	ϕ	C, psf
1	125	20	1000
2	160	35	0
3	133	10	2850

FIGURE 7. TYPICAL SUBSURFACE PROFILE - ANALYSIS SECTION

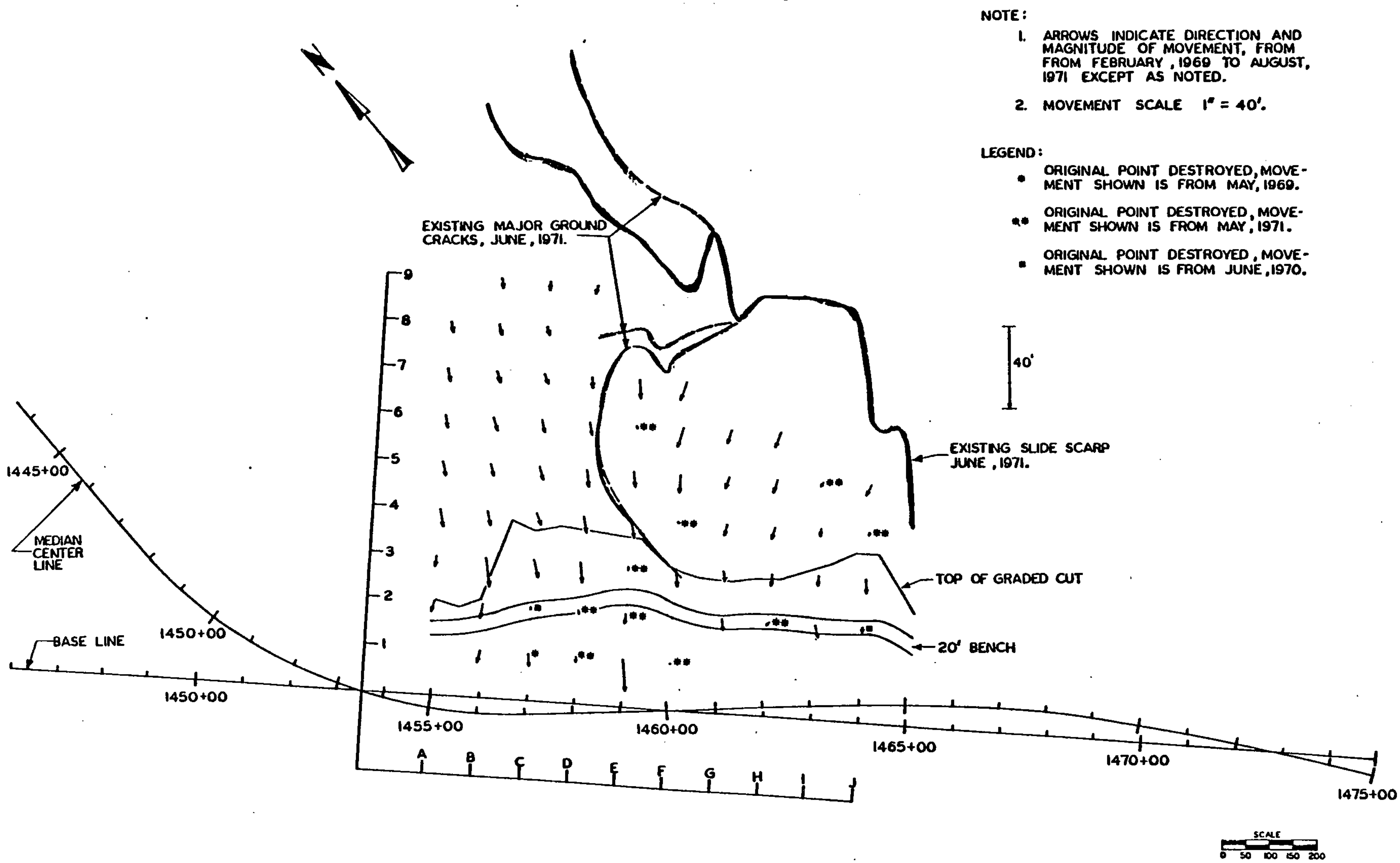


FIGURE 8. SURFACE MOVEMENTS

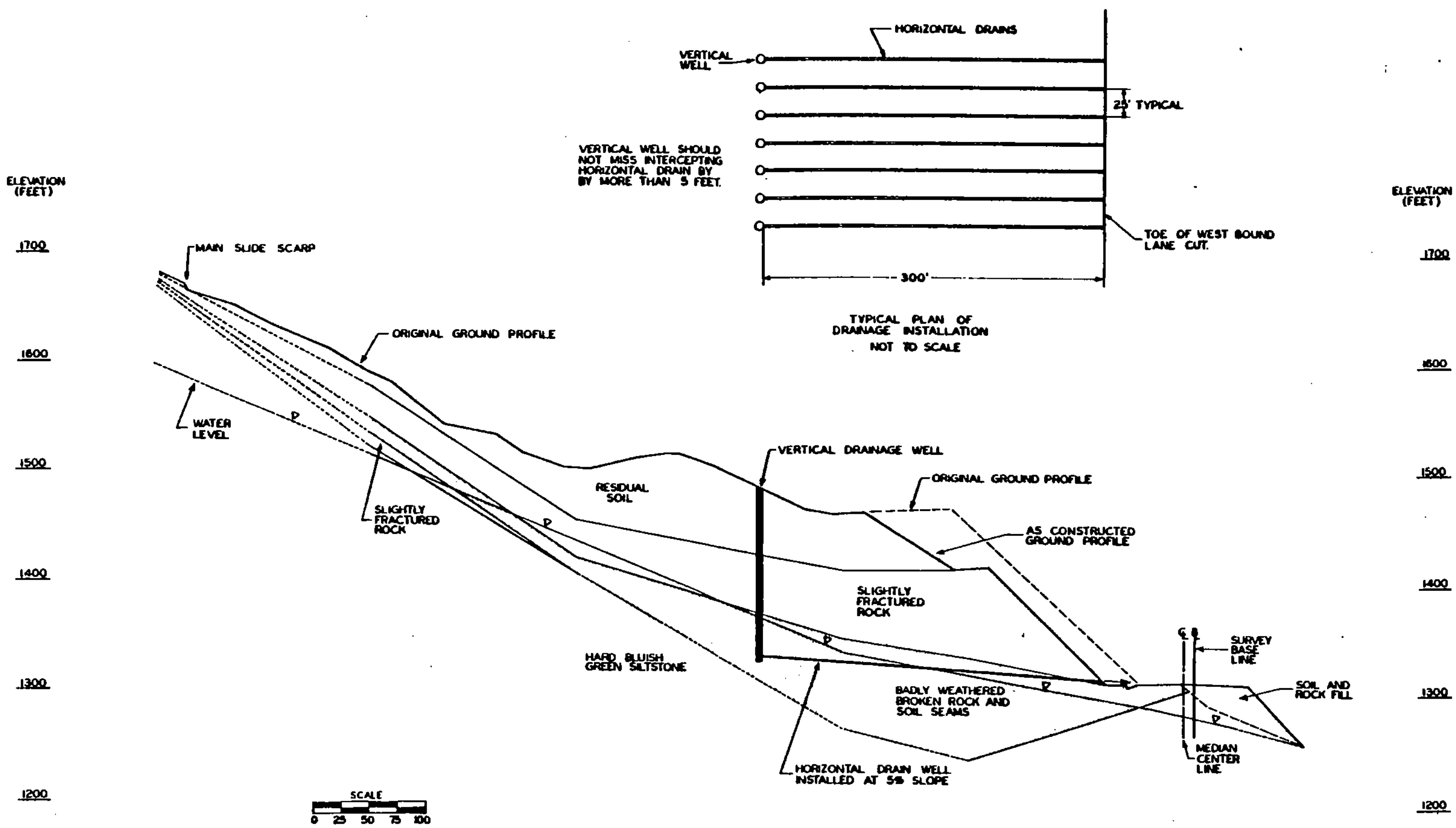


FIGURE 9. REMEDIAL DRAINAGE CONCEPT

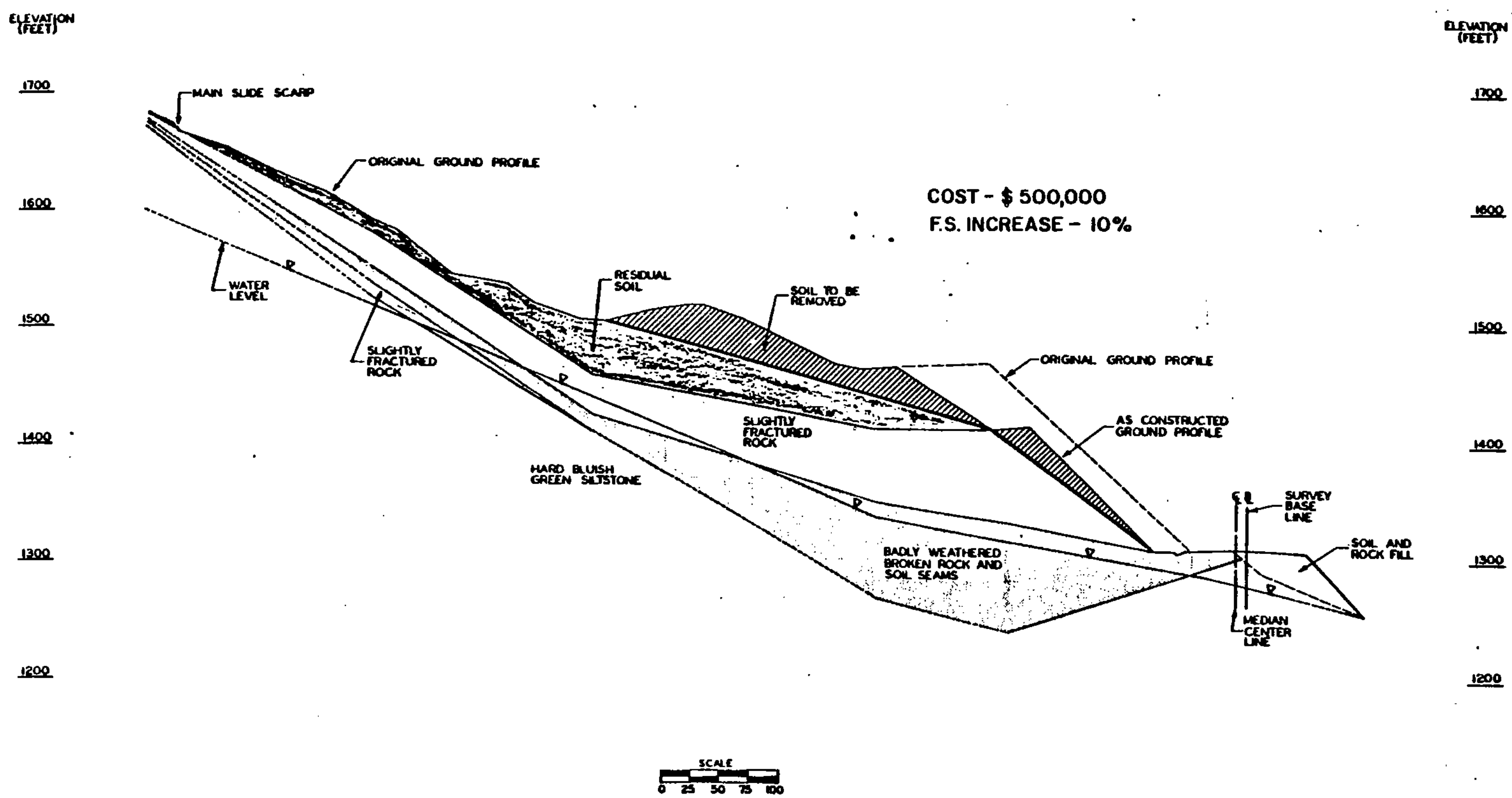


FIGURE 10. ' MINIMUM UNLOADING CONCEPT

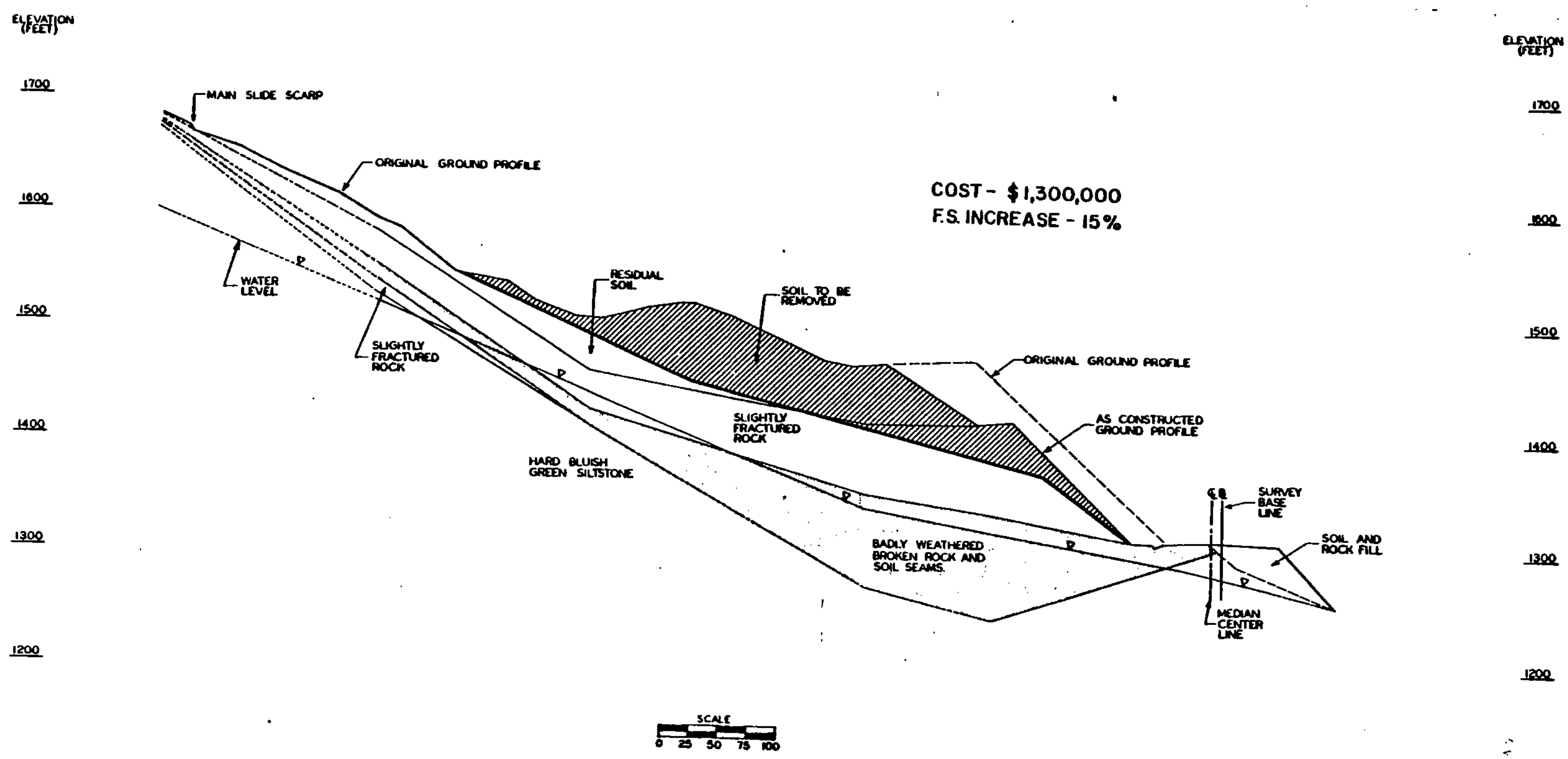


FIGURE 11. MAXIMUM UNLOADING CONCEPT

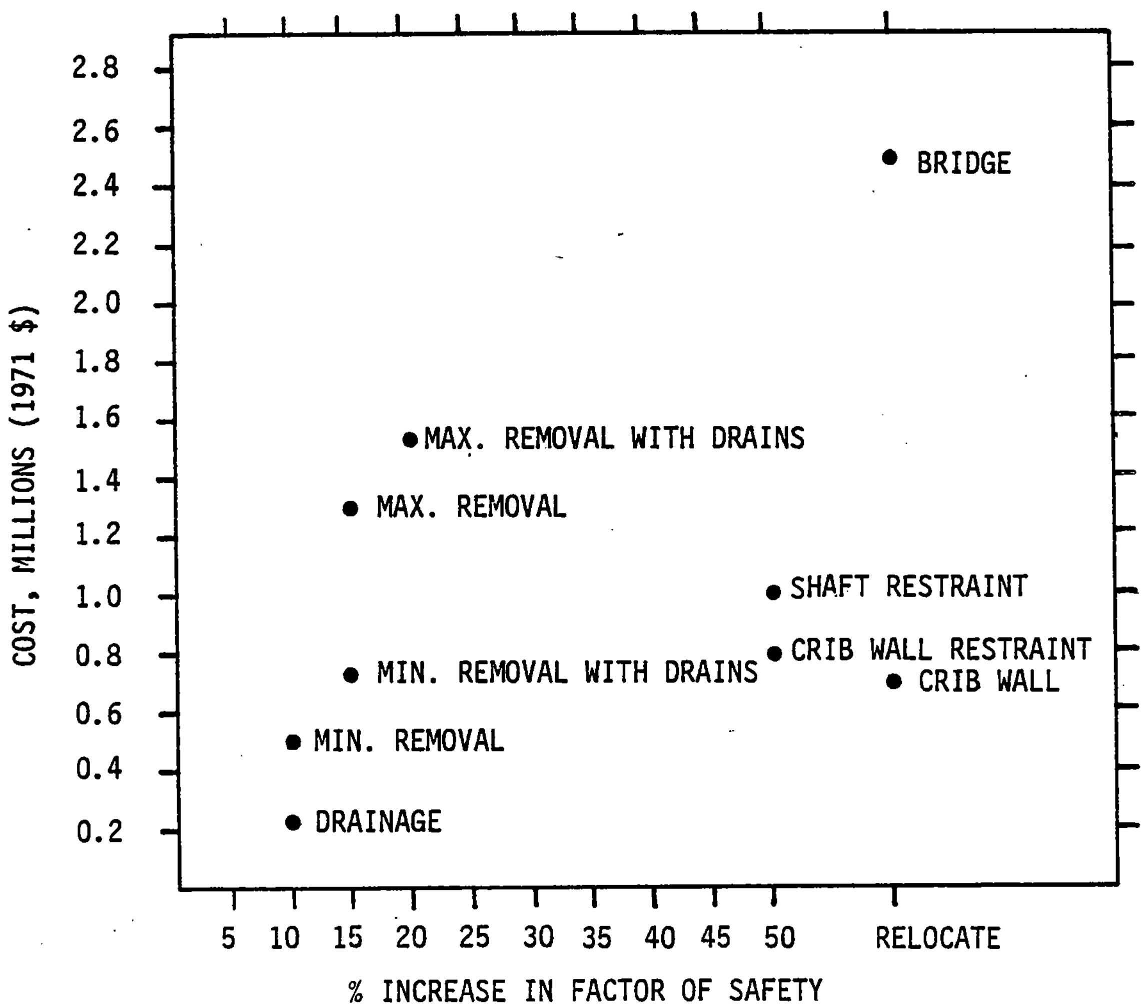


FIGURE 12. SUMMARY OF REMEDIAL CONCEPTS

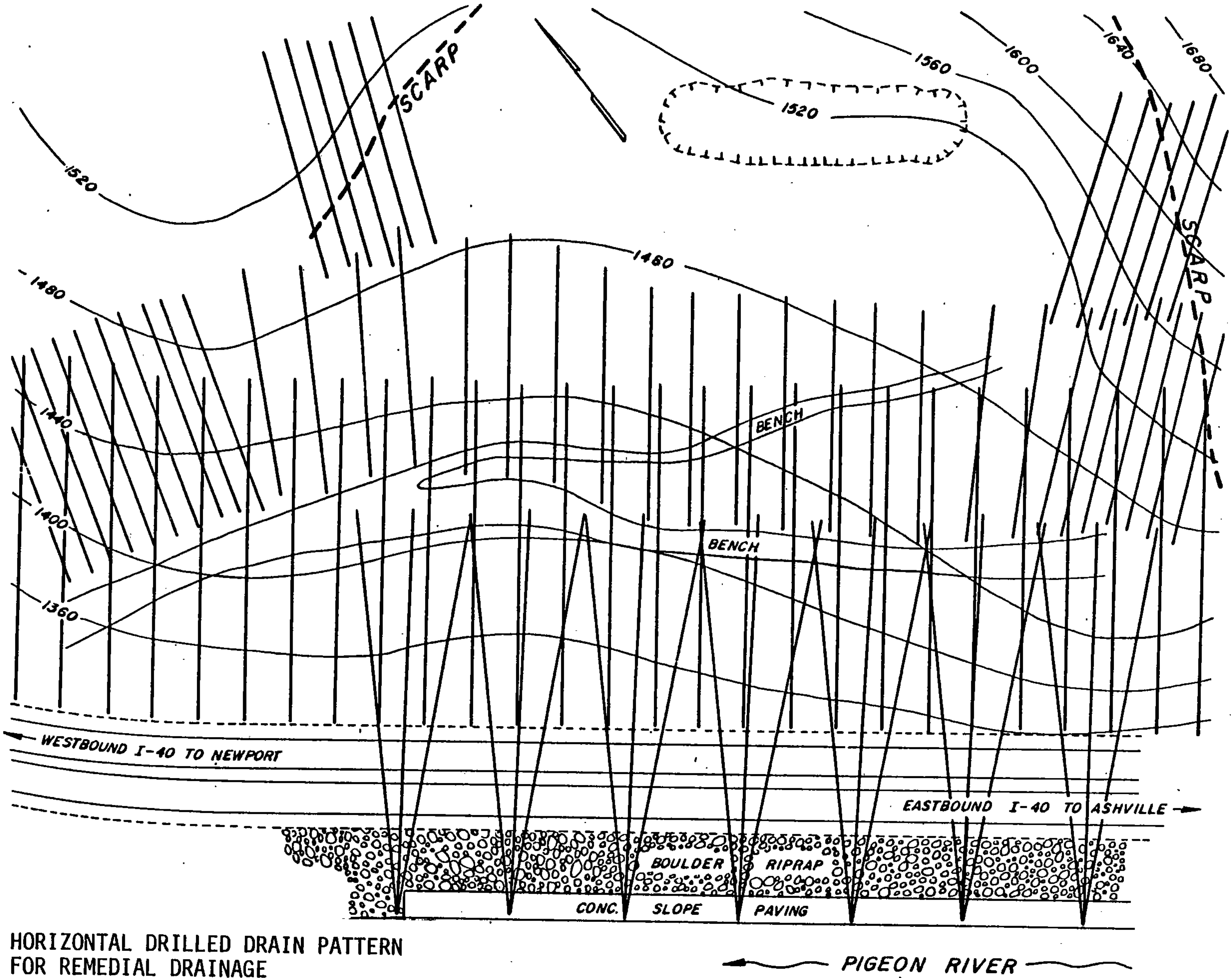


FIGURE 13. HORIZONTAL DRILLED DRAIN PATTERN FOR REMEDIAL DRAINAGE (ROYSTER (5), BY PERMISSION)

CONSTRUCTION PROBLEMS INVOLVING SHALE

IN A GEOLOGICALLY COMPLEX ENVIRONMENT

STATE ROUTE 32 - APPALACHIAN CORRIDOR "S"

GRAINGER COUNTY, TENNESSEE

By

James H. Aycock
Regional Engineering Geologist
Division of Soils and Geological Engineering
Tennessee Department of Transportation

INTRODUCTION

A recent survey conducted by the Federal Highway Administration (FHWA) revealed that we are presently spending $50 million annually to repair major landslides on the Federal Aid Highway System. The survey concludes that the total cost for landslide repair may be as high as $100 million if all failures, including those repaired by State Maintenance Forces, are included. The January 1976 issue of Highway Focus published by the U. S. Department of Transportation was devoted entirely to stressing the need for the utilization of geotechnical techniques during the design, location, and construction of modern highways as a means of reducing this astronomical figure.

The FHWA survey also revealed that the cost for landslide repair and related remedial work in Region IV (Figure 1), which includes the 8 southeastern states, was $12 million annually--a rate almost double that of any other region. Several of these states included in Region IV lie within the Appalachian Highlands Physiographic Division. East Tennessee, located in these highlands, includes parts of the Blue Ridge, or Unaka Mountains, the Valley and Ridge Province, and the Cumberland Plateau Section. The preponderance of problems in Region IV are, in part, a direct reflection of the severe deformation that occurred on a regional level during the mountain building period near the end of Paleozoic time. The inherent structural weaknesses of the faulted, folded, and severely fractured rock strata have been magnified by a very long period of weathering and, during recent geologic time, a very high annual precipitation rate.

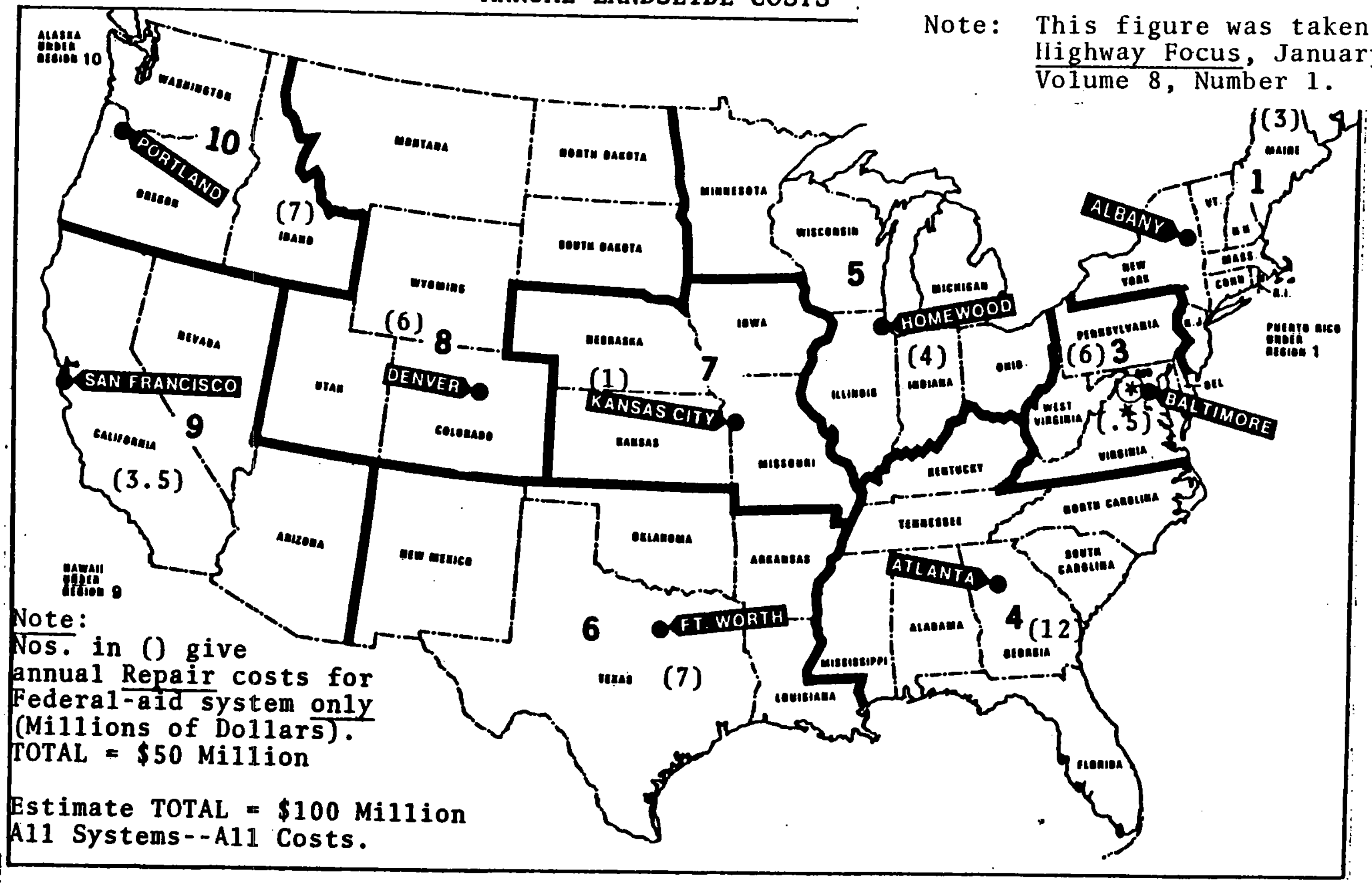
The high annual rate of precipitation has been the determining factor in most of the major failures. For example, some of the most costly repairs have been necessitated because of the deterioration of large embankments constructed from slaking shales where infiltrating groundwater has reduced the shear strength to the point of failure.

In East Tennessee the cost of landslide and related repair on Interstate 75 in Campbell County and Interstate 40 in Cocke and Roane Counties alone is near the $30 million mark. Except during the remedial work, there was very little geotechnical input into these projects.

Field Regions of the Federal Highway Administration

ANNUAL LANDSLIDE COSTS

Note: This figure was taken from Highway Focus, January 1976, Volume 8, Number 1.



Washington, D.C. Headquarters

* Region 15 (Arlington, Va.), Eastern Federal Highway Projects Office

Field Region Headquarters

NOTE: FHWA Region 1 Conforms to Standard Regions 1 and 2

FIGURE 1

Subsequent to these events, two major roadways located in East Tennessee in Region I of the Tennessee Department of Transportation have been planned in corridors where severe geotechnical problems are largely unavoidable. Acute attention has been focused on the geotechnical problems during the location, design, and construction phases. A section of the first project, State Route 63 in Scott and Campbell Counties, is now complete. The second project, State Route 32 - Appalachian Corridor "S" which crosses Clinch Mountain in Grainger County, is presently under construction. The Division of Soils and Geological Engineering of the Tennessee Department of Transportation was and is very responsibly involved in the construction of these projects. This paper covers some of the design concepts and field decisions made during construction on that specific section of State Route 32 - Appalachian Corridor "S" which traverses the slopes of Clinch Mountain. Relative to geotechnical complexity, this is the most difficult roadway yet attempted in this part of the Valley and Ridge Province of East Tennessee. Present construction is divided into two projects which are under separate contracts. The two projects meet at Beans Gap at the top of the mountain.

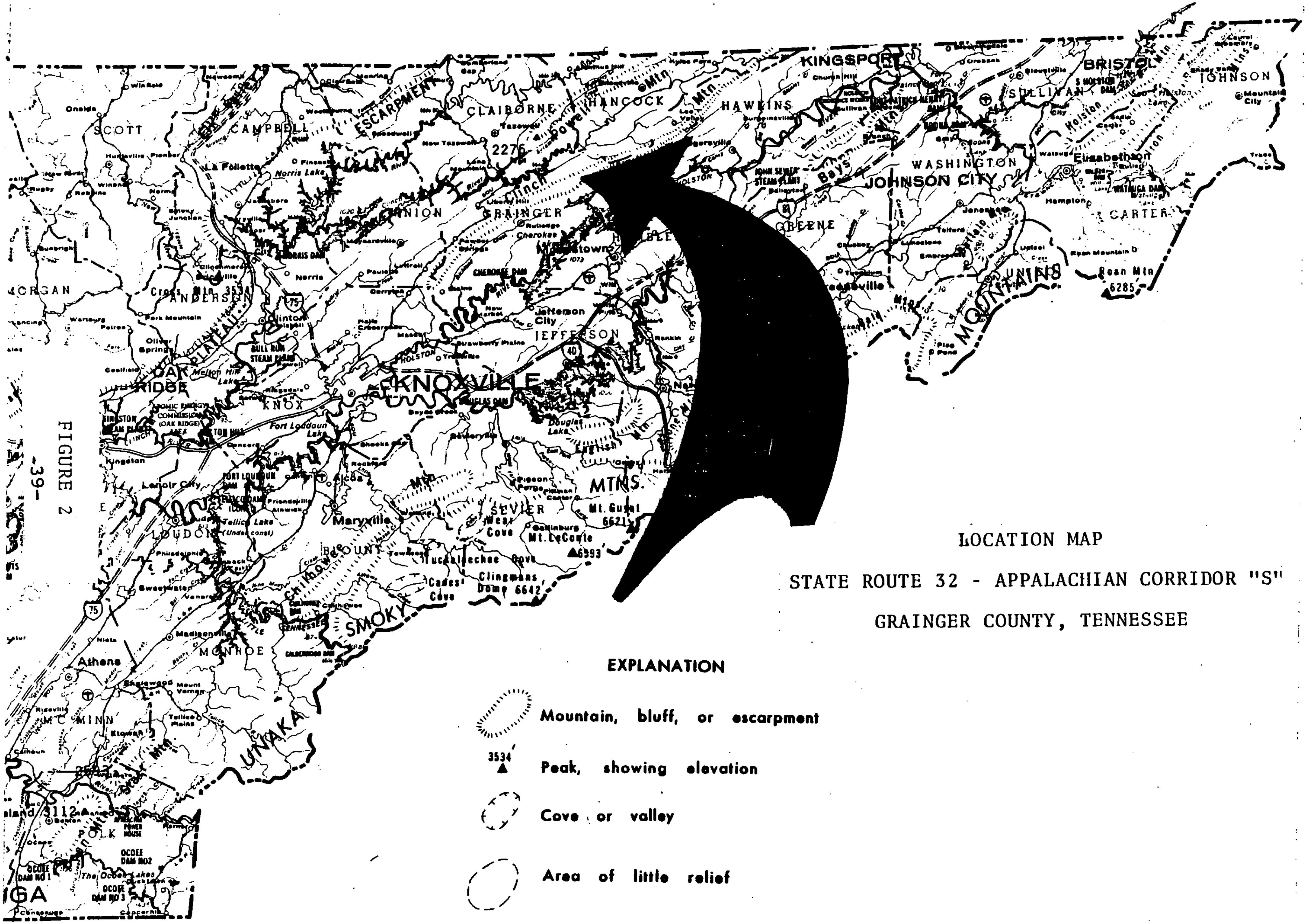
GEOGRAPHIC SETTING

State Route 32 - Appalachian Corridor "S" is located in northeastern Grainger County and lies well within the Valley and Ridge Physiographic Province. This province is characterized by long parallel valleys and ridges that trend northeast-southwest. Clinch Mountain is the most prominent ridge in this province and extends from northeast of Knoxville into Virginia, a total distance of some 402.34km (250 miles). In Tennessee this mountain varies from 701m to 731m (2300' to 2400') in elevation and divides the Clinch and Holston River Valleys. The scarp side of the mountain is to the northwest and the slope side to the southeast. Because of the resistant sandstone cap rock there are no deep gaps in the mountain that might permit easy passage.

The project is 13.2km (8.2 miles) in length. It begins near Indian Creek on the northwest side, crosses the mountain at Beans Gap, and terminates near Briar Fork Creek at the base of the mountain on the southeast side. The low point of the project occurs in the bottoms of Indian Creek near Station 29+50 at an elevation of 1002 feet. The point of highest elevation is 626m (2055') at Beans Gap. Partly because there are no natural passes, this is the first roadway designed to modern grade and alignment specifications across this mountain in Tennessee. Only that part of the alignment which crosses the mountain proper between Stations 161+50 and 451+00 is of concern in this paper (Figure 2).

GEOLOGY: HISTORY, STRUCTURE, AND STRATIGRAPHY

The great mountain building uplift, which occurred near the end of Paleozoic time and formed the Appalachian Highlands, severely deformed the rock strata. The thrusting movement from



LOCATION MAP

STATE ROUTE 32 - APPALACHIAN CORRIDOR "S"

GRAINGER COUNTY, TENNESSEE

FIGURE 2

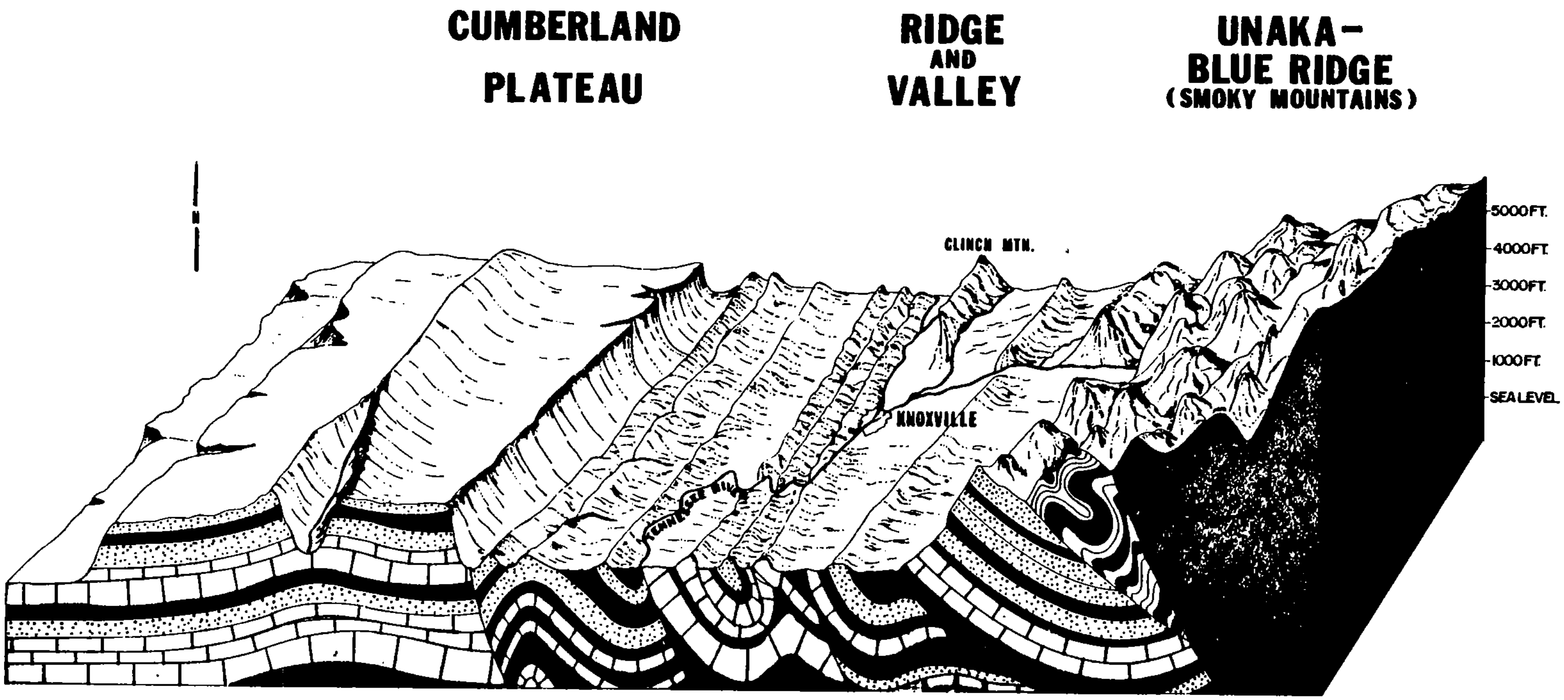
the southeast toward the northwest shoved large rock masses inland. Rock strata which had once been horizontal were tilted at steep angles, folded, faulted, and fractured. A pronounced joint system was formed. All of these features can be seen in the exposed rock on Clinch Mountain.

The subsequent long period of weathering developed deep colluvial or talus deposits on the scarp side of many ridges and mountains. Colluvial deposits up to 12.2m (40') deep were encountered on the northwest slope of Clinch Mountain during the exploratory drilling for the project. Joints and fractures in the rock strata were infiltrated by water which widened the joints, particularly in the soluble carbonate rocks. Deep weathering in slaking shales occurred, often reducing the shale to the clay state throughout the zone of weathering. Abrupt topographic relief was formed as softer materials were weathered away leaving the more resistive strata. Clinch Mountain exists as it does today because of the less erodible sandstone cap rock (Figure 3).

Relative to the geologic systems, the project begins in the Lower Cambrian Rome Formation on the northwest side of the mountain, crosses progressively younger strata of Cambrian, Ordovician and Silurian ages, and terminates in the Upper Devonian-Mississippian Chattanooga Shale on the southeast side.

The interval requiring compensatory design measures begins in the foothills at the base of the mountain at Station 161+50 on the northwest side, crosses the mountain at Beans Gap at Station 267+00 and continues to the end of the project at Station 451+00. Most of the design and construction problems relate to the geologic processes defined earlier. The northwest, or scarp side, of the mountain is underlain by deeply weathered clay shales with high moisture contents which are, in turn, overlain by colluvial or talus deposits up to 12.2m (40') deep. The roadway traverses this unstable material in a steep side-hill location. As would be expected, the depth of colluvium diminishes near the top of the mountain. The subsurface materials change to harder shale, claystone, and siltstone; however, the well developed joint system poses a serious rockfall condition in the deep cut slopes in this interval. Yet, the strata dip into the mountain so that bedding plane failures are not a threat. From the crest of the mountain to the end of the project on the southeast side, the roadway drops approximately 244m (800') in elevation. The roadway is generally in side-hill cut and fill for 5.46km (3.39 miles). On the uphill side or to the left of centerline, the strata dip into the roadway at angles ranging from 20 to 30 degrees, thus creating stability problems in both cut and fill.

Beginning at the crest of the mountain and continuing for a distance of some 3.07km (1.91 miles), the project traverses the massive Clinch Sandstone. Thin clay and shale seams interbedded between thick sandstone strata create severe stability problems in this interval. The well developed system of high angle joints in the sandstone has permitted the infiltration of water to the extent that the clay and shale seams are typically saturated. These seams are widely



PHYSIOGRAPHIC BLOCK DIAGRAM OF EAST TENNESSEE
Showing the Relationship of Major Geologic
Structures of the Physiographic Expression
(Harry Moore, 1978)

FIGURE 3

spaced in the higher elevations but become thicker and more numerous near the base of the mountain. Shale predominates in the final 2.38km (1.48 miles) of the project (Figure 4).

COMPENSATORY DESIGN CONCEPTS-NORTHWEST SLOPE

Colluvial or talus deposits overlying deeply weathered clay shale and shale residuum combined with steep natural slopes present the greatest stability problems on the northwest side of the mountain. The most severe problems exist in the interval Station 161+50 to 245+00 where soft layers in the clay shale were encountered. Water contents in the 50's, liquid limits in the 60's and compressive strengths as low as 225kg/m^2 (500 pounds/ft.^{2}) were recorded in the softer material. The overlying colluvium in this interval ranges from 3.05 to 9.15m (10' to 30') in depth. This same juxtaposition of materials in combination with the high annual rate of precipitation resulted in most of the more that 30 failures that occurred in Interstate 40 in Roane County.

The colluvium is a very permeable mixture of loosely consolidated clay, silt, sand and sandstone boulders. The underlying clay shale residuum is relatively impervious. Water percolating through the colluvium encounters the underlying shale and flows downslope along the colluvium-shale interface. Because of the concentration of water along the shale surface, shear strength in the upper few feet is often drastically reduced. This condition is particularly marked where shale residuum underlies the colluvium. Seepage pressures become very high along the interface.

Both cuts and fills can be very unstable where this condition exists in a side-hill location. Where loads are added, such as in the construction of large embankments, consolidation occurs in the colluvium and underlying soft shale. Established drainage channels are disrupted and either blocked or restricted. Seepage pressures increase and the overlying embankment is infiltrated by a rising water table. The increased moisture contents in the fill material reduces the shear strength and failure results. Where cut excavation removes downslope support and exposes the colluvium-shale contact, failure often happens along the contact resulting in a colluvial flow downslope and into the open excavation. Unless the flow is restrained in some way, such as by replacing the support with a free draining gravity structure, very large failures may result as the colluvium continues to flow downslope. One such failure involving approximately 608,000m^3 (800,000 yd.^{3}) occurred on Interstate 40 in Roane County (Figures 5 and 6).

The compensatory design evolved for embankments in the colluvium-shale interval Station 161+00 to 245+00 utilizes a rock filled stability bench or shear key beneath the fill. When constructing the stability bench, excavation is made through the colluvium and soft shale or shale residuum until the unweathered, stratified material is encountered. Select nondegradable rock backfill is then used to fill the excavated area. The rock backfill is brought back to the elevation of

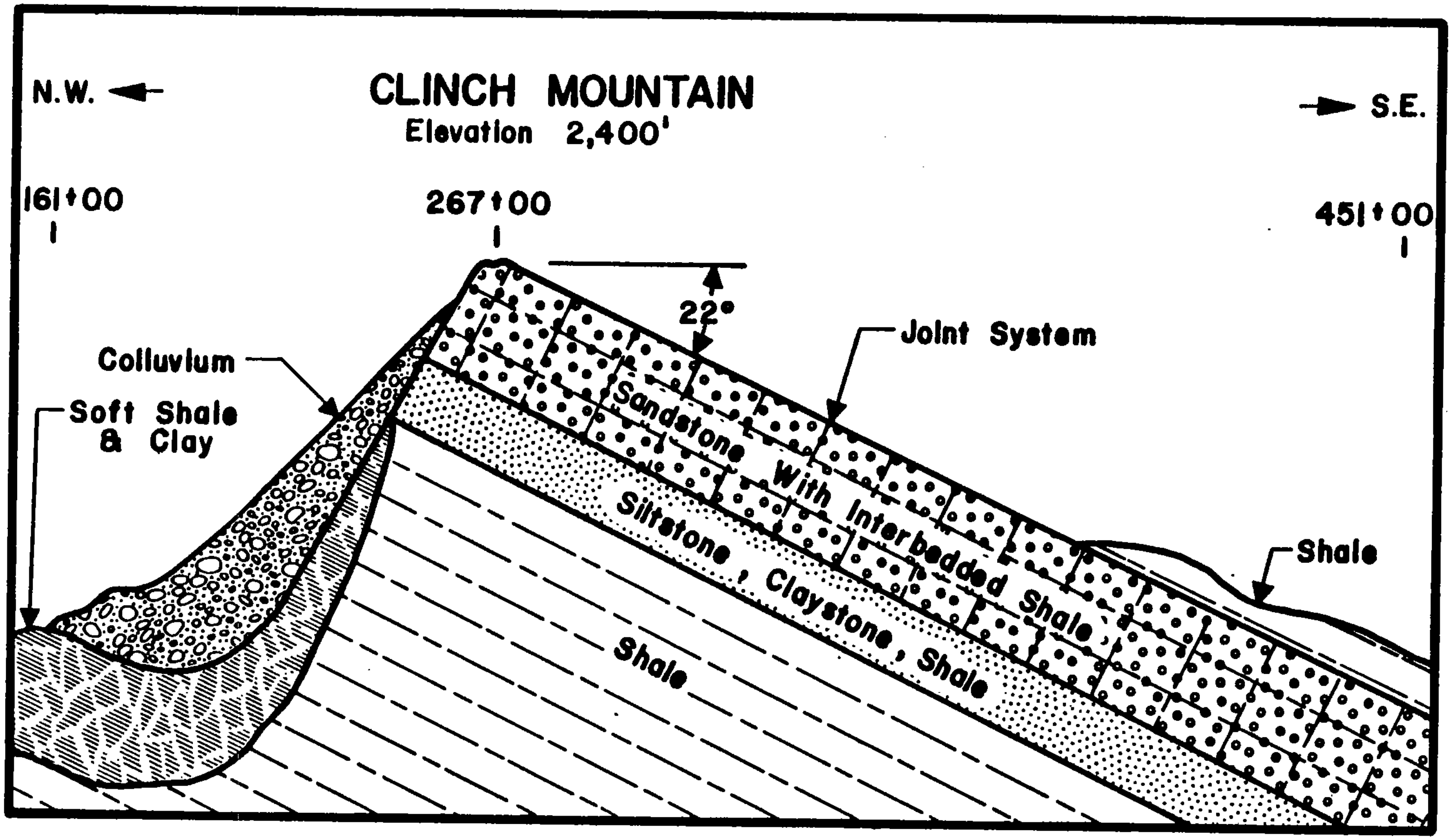
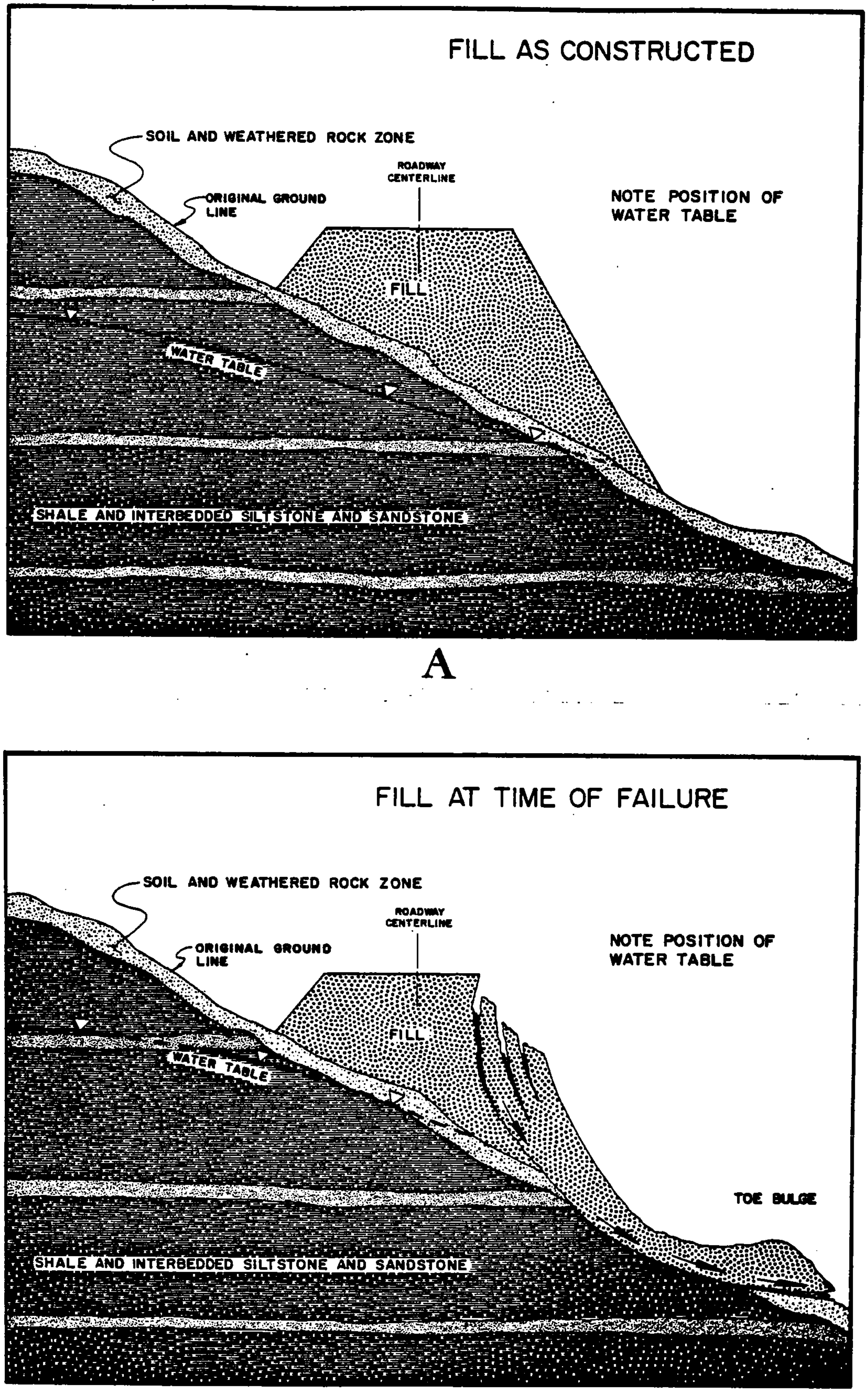


Figure 4. Schematic drawing through subject section of State Route 32 - Appalachian Corridor "S" across Clinch Mountain showing conditions requiring compensatory design for unstable conditions - colluvium overlying soft shale, steep slopes, adverse dips, high angle joint systems, weak shale seams interbedded with thick more competent sandstone strata.

FIGURE 4

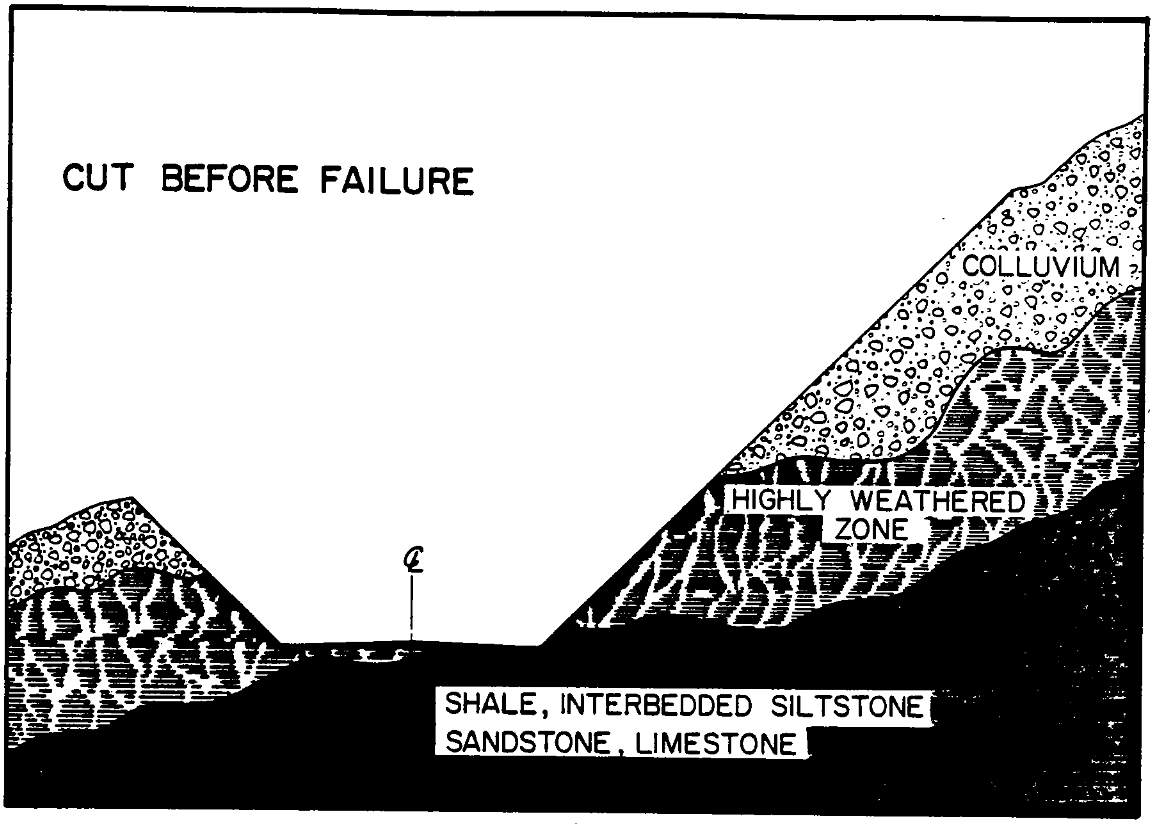


A

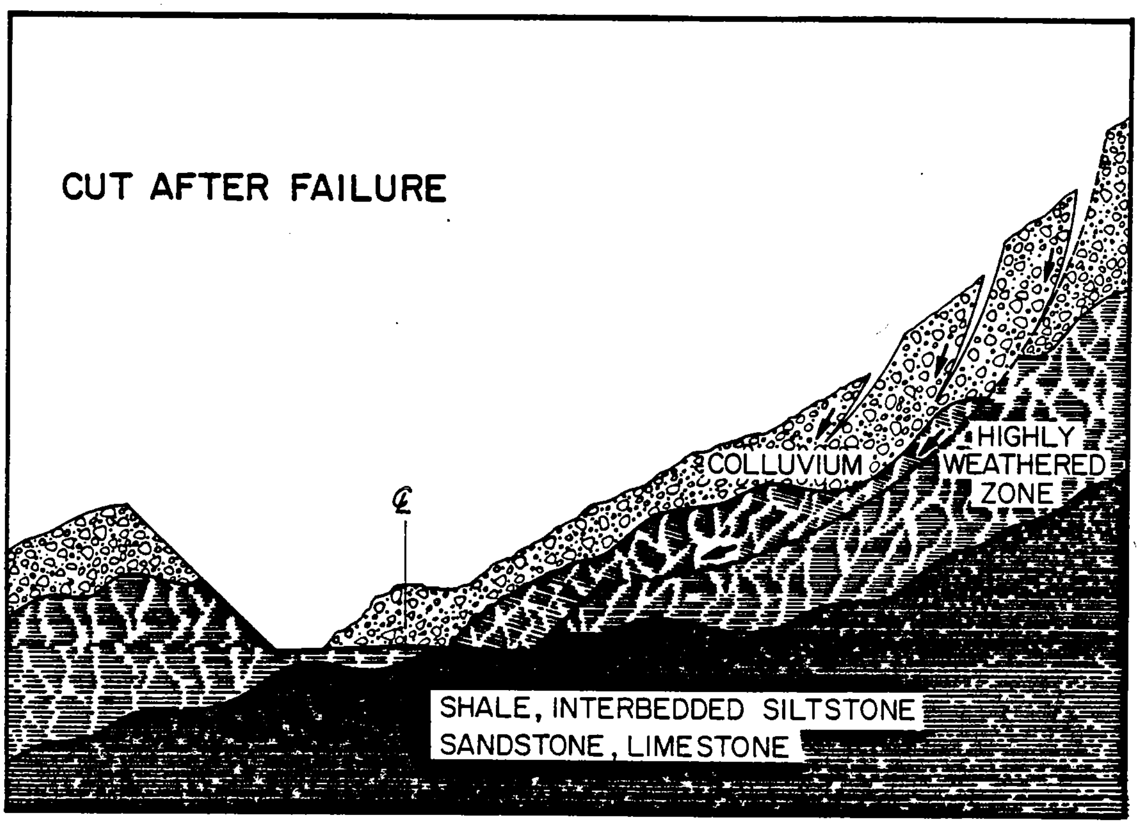
B

EMBANKMENT BEFORE AND AFTER FAILURE

FIGURE 5



A



B

EXCAVATION SLOPE BEFORE AND AFTER FAILURE

FIGURE 6

the natural ground line which existed prior to excavation. Drainage of the stability bench is effected by a perforated cm pipe which is placed in the lowest part of the excavation. The stability bench serves several functions: it forms a free draining foundation with inherent high shear strength for the large embankments, acts as a butress for the upslope colluvium and prevents pore pressure buildup by virtue of its free draining characteristics (Figure 7).

A rock buttress founded on stable material below the colluvium-shale interface is the compensatory design whereever colluvium is present in cut intervals. The buttress forms a free draining gravity structure which effectively restrains the unstable interface and prevents the colluvium and wet shale material from flowing into the open excavation (Figure 8).

The select rock material designated for construction of the stability benches and rock buttresses conforms to the following description:

1. The rock shall consist of sound limestone or sandstone with a maximum size of 0.91m (3') measured along the longest dimension.

2. At least 50 per cent (by volume) of the rock shall be 0.30m (1') or larger in the minimum dimension.

3. No greater than 10 per cent (by volume) shall be less that 5.08cm (2") in diameter.

4. The rock shall be free of shale or clay.

This specification has proven to be the most workable in the construction of free form gravity structures in Tennessee. Sound interlock of the rock fragments is obtained, good internal drainage is provided and a steep frontslope ranging from 1.25:1 to 1.5:1 can be constructed.

Because there were large quantities of sound limestone and sandstone on the project it was specified in the plans that the select rock be developed from planned excavation. The rock did in fact either come from the project or was developed from adjacent sources.

One other notable design concept in the interval underlain by soft shale, but where colluvium is absent, is the use of serrated backslopes. The serrated steps are 0.91m (3') in both the horizontal and vertical dimensions. The maximum height of the serrated lift is 9.15m (30') on a 1:1 slope ratio. The serrated slopes are separated by in-slope benches 6.1m (20') wide (Figure8).

Beyond Station 245+00 to the end of the project at Station 272+00, where the alignment crests the mountain at Beans Gap, overburden becomes thin and more stable claystone, siltstone, shale and sandstone are encountered. Backslopes in this interval range to 48.8m (160') in height. To the right of centerline rock strata dip away from the roadway thus enhancing slope

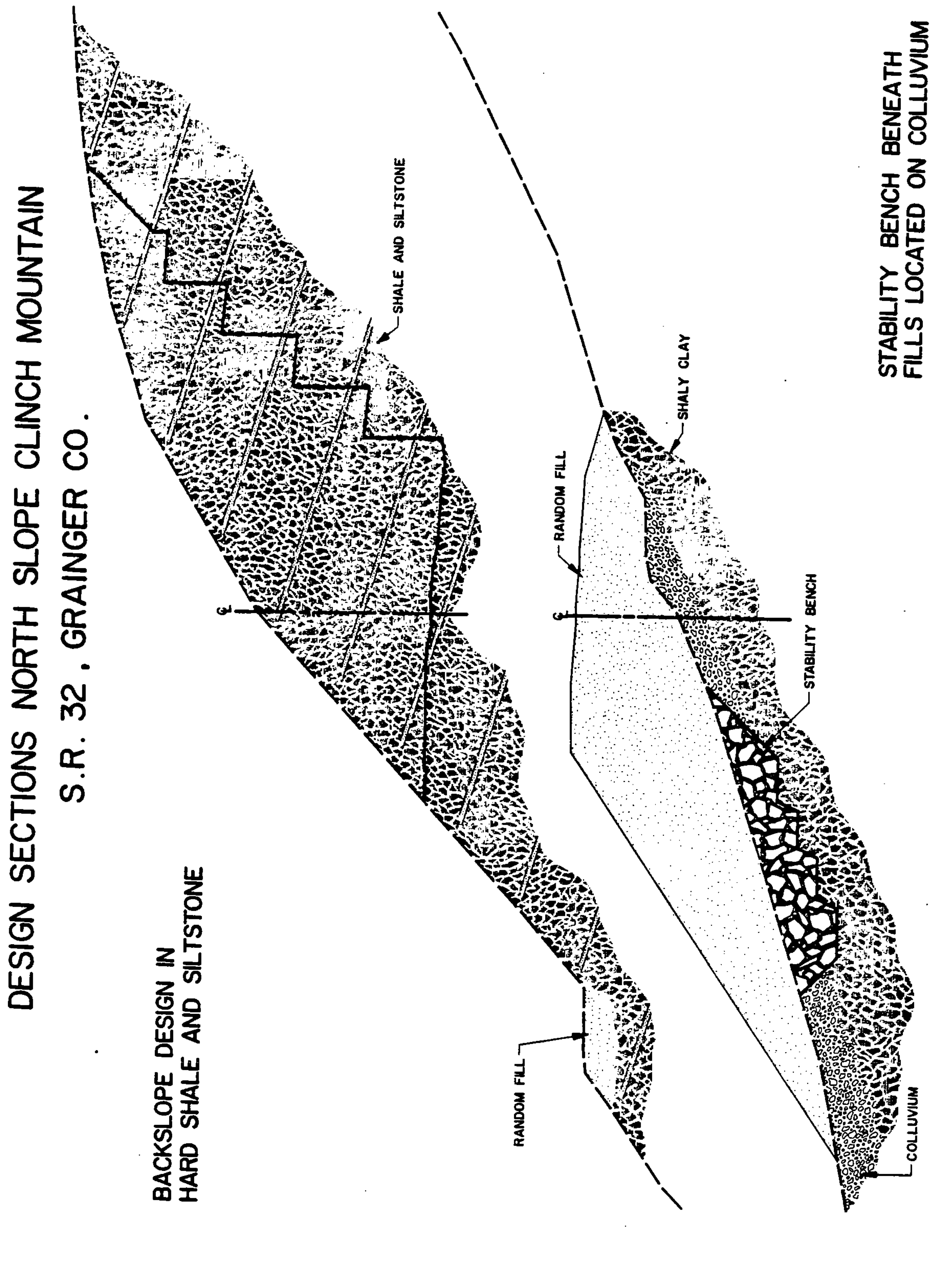

DESIGN SECTIONS NORTH SLOPE CLINCH MOUNTAIN
S.R. 32, GRAINGER CO.
BACKSLOPE DESIGN IN
HARD SHALE AND SILTSTONE
SHALE AND SILTSTONE
RANDOM FILL
RANDOM FILL
SHALY CLAY
STABILITY BENCH
COLLUVIUM
STABILITY BENCH BENEATH
FILLS LOCATED ON COLLUVIUM

FIGURE 7

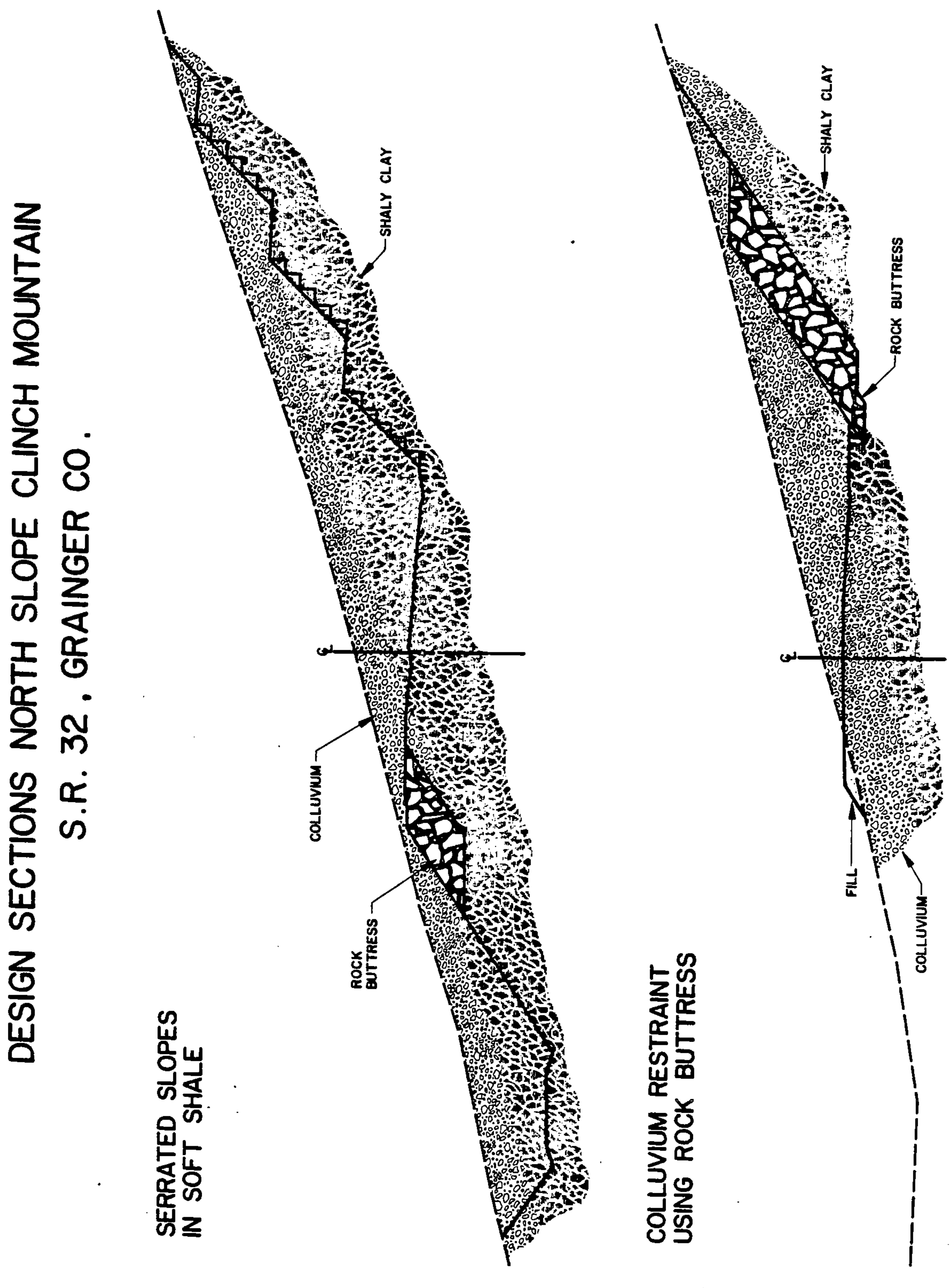

DESIGN SECTIONS NORTH SLOPE CLINCH MOUNTAIN
S.R. 32, GRAINGER CO.
SERRATED SLOPES IN SOFT SHALE
SHALY CLAY
COLLUVIUM
ROCK BUTTRESS
COLLUVIUM RESTRAINT USING ROCK BUTTRESS
SHALY CLAY
ROCK BUTTRESS
FILL
COLLUVIUM

FIGURE 8

stability, but to the left of centerline from Station 258+00 to the end of the project at Station 272+00, the curvature of the roadway just begins to undercut dip. While this slope does not present an "en masse" stability problem, a number of wedge failures have occurred. Vertical backslopes incorporating 9.15m (30') vertical lifts separated by in-slope benches 6.1m (20') wide are specified from Station 245+00 to the end of the northwest slope project at Station 272+00.

EXPERIENCES DURING CONSTRUCTION - NORTHWEST SLOPE

Construction on the project on the northwest slope was initiated on February 14, 1977. The most significant construction problems to date have involved the following:

1. Failure of excavation slopes both during preparation and following preparation of stability bench areas prior to backfilling;

2. Difficulty in obtaining the select rock specification during early stages of construction;

3. Wedge failures in the cut slope to the left of centerline Stations 258+00 to 272+00.

An inherent problem with the construction of stability benches and rock buttresses in the colluvium-soft shale complex is the almost certain failure of excavation slopes before the rock backfill is in place. Because of the danger of failure, it is usually specified that such construction be done only during the dry months of the year, that excavation be carried out in specific sections - usually 15.25 to 30.5m (50' to 100') long - and that sufficient rock be on hand to backfill the sections upon completion. Perversely, construction is usually carried out in the wettest months, the entire construction interval is opened to the elements and rock production is some several days or weeks away. In the case of the northwest slope project several of the stability bench intervals were opened to foundation elevations early in construction and left open to weathering through the winter months. In the stability bench area at Station 189+00 where the colluvium was the deepest, where the shale was weathered to a wet plastic clay and hydrostatic pressures the greatest, the inherent instability was irrevocably compounded and failure assured when waste material from the excavation was stored upslope adjacent to the open pit. Subsequent failure deposited several thousand cubic yards of colluvium and stored waste into the open excavation. The material, of course, had to be excavated again and removed from the site. Several less significant slumps into open excavation areas have occurred.

During the early stages of construction, it was most difficult to obtain the required specification for select rock backfill. There was an inclination to use material that was either too large - up to 3.66m (12') along the longest dimension on the largest fragments - or too small, consisting largely of undersize rock fragments in a sand matrix. This conflict was resolved eventually and the select rock that has been used to date is very good.

A third problem encountered during construction involved the cut slope to the left of centerline Station 258+00 to 272+00. As noted earlier, the curvature of the roadway just begins to undercut the dip slope in this area. The original design called for vertical lifts 9.15m (30') high separated by in-slope benches 20 feet wide. The upper 15.25m (50') of the slope passes through weathered shale and thin sandstone layers. A 1:1 slope was constructed through the upper interval. The joining of several high angle joint sets with the slightly undercut dip has precipitated a number of large wedges into the roadway. The wedge failures have completely removed large sections of the low benches rendering them useless as catchment areas. The slope has been redesigned utilizing a 1:1 slope ratio throughout and a 12.2m (40') wide fallout area at grade. The 1:1 slope ratio is not expected to eliminate all the wedge failures, but is expected to reduce them significantly in size. The fallout area at grade should be wide enough to prevent migration of material into the roadway. Because of the steep natural slopes, a flatter cut slope is prohibitive. Consideration was given to removal of the lower benches to obtain a wider catchment area at grade; however, the wedge failures had rendered the benches inaccessible to the drilling equipment.

COMPENSATORY DESIGN CONCEPTS - SOUTHEAST SLOPE

From the beginning of the project on the southeast slope of the mountain at Station 272+00 to the end at Station 451+00, the roadway is most typically in side hill cut and fill. For the entire 5.46km (3.39 miles) the rock strata dip into the roadway on the uphill side of the alignment, an unprecedented condition for a roadway in East Tennessee. This condition poses a serious stability problem in both cut and fill throughout the southeast slope. This project may well be the definitive experience with respect to dip slope problems in roadway construction. The massive Clinch Sandstone predominates in the interval Station 272+00 to 373+00, but interbedded shale and clay seams present planes of instability throughout this interval. The percentage of interbedded shale increases progressively toward the base of the mountain; beyond Station 373+00 shale predominates. The dip slope ranges from 22 degrees± to 30 degrees±. The surface expression of the south slope roughly parallels the dip slope of the underlying strata.

During the exploration phase, it was determined that there was no practical means of avoiding at least some cut slides in the very deep excavation required for the prescribed grade and alignment on the southeast slope. The maximum cut height exceeds 33.55m (110') in one backslope to the left of centerline with slopes 9.15m to 21.35m (30' to 70') in height being common. During the location phase, a preliminary report by the Division of Soils and Geological Engineering indicated planar failures along dipping strata would occur in backslopes which exceeded 6.1m (20') in height. The threat of planar failure exists in any planned construction from the crest of the mountain to the end of the project. The angle of dip is optimum for sliding. Had the dip been a few degrees steeper, it would have been possible to preclude sliding by constructing the backslope on a slope ratio equivalent to the angle of dip. Individual strata would not have been undercut and would have remained supported at the toe. If the dip had been a few degrees flatter, sliding may not have occurred or may not have been so severe.

Since it was concluded no practical way could be found to avoid at least some large cut slides, a backslope was designed to retain slide debris so that the traveled roadway would not be affected. In deep excavations the backslope design utilized vertical lifts and wide benches. The first in-slope bench is located 5.18m (17') above grade and is 12.2m (40') wide. A maximum 15.25m (50') lift with a 9.15m (30') wide bench at the top continues above the first bench. Above the 9.15m (30') bench a vertical lift is carried through the natural slope intersect. A 3.05m (10') wide catchment area at grade is incorporated into the design (Figure 9).

In the shale section beyond Station 373+00 a vertical slope to 5.18m (17') above grade is employed. An in-slope bench 9.15m (30') wide, where still in rock, at 5.18m (17') above grade is designated. A 1.5:1 slope ratio with a 6.1m (20') wide bench on the natural dip at the top of the rock is continued to the ground line (Figure 9).

Unlike the northwest slope, colluvium on the southeast slope is not widespread but rather is confined principally to the ravines where it ranges up to 3.05m (10') deep. Nevertheless, the colluvium presents a real stability threat in that it masks a continuous flow of water from both active and wet weather springs that exit along the contact between the downsloping strata and the base of the colluvium. It is very important that this condition be recognized, especially in the shale interval. Filling over the colluvium without providing adequate drainage will invariably result in water infiltrating the overlying shale embankment. Subsequent reduction of shear strength and ultimate failure is a certainty, if infiltration occurs.

In embankment intervals compensatory design measures (Figure 10) include the following:

1. Undercutting of loose, wet material and construction of the lowermost 1.53m (5') of all embankments with the best available rock (sandstone section Station 272+00 to 373+00);

2. Continuous benching of all existing slopes that are steeper than 4:1 prior to construction of fills;

3. Construction of toe benches to be backfilled with select rock (the toe bench is designed primarily to prevent toe saturation in embankments and secondarily for mechanically securing the toe);

DESIGN SECTIONS SOUTH SLOPE CLINCH MOUNTAIN

S.R. 32, GRAINGER CO.

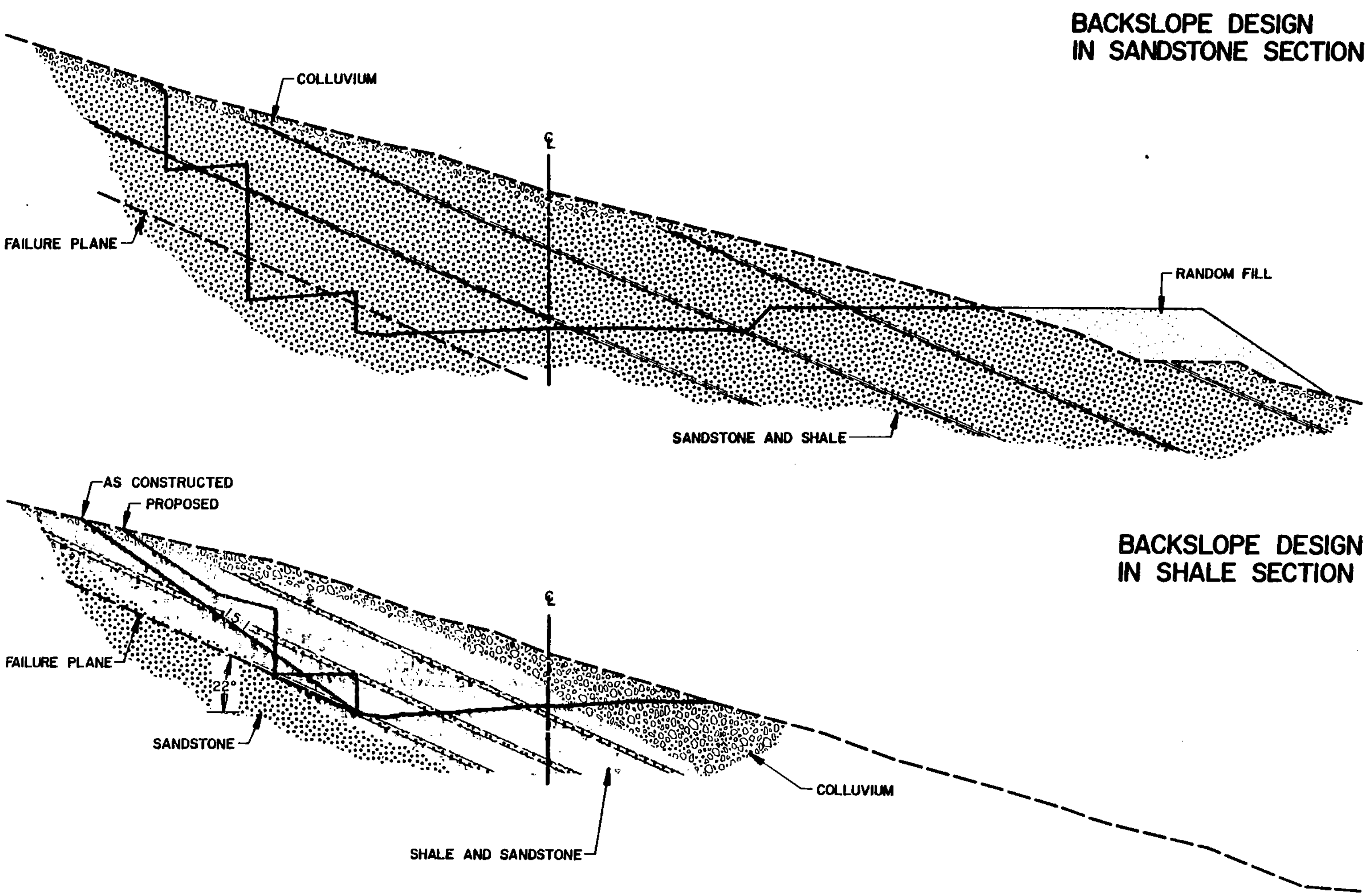


FIGURE 9

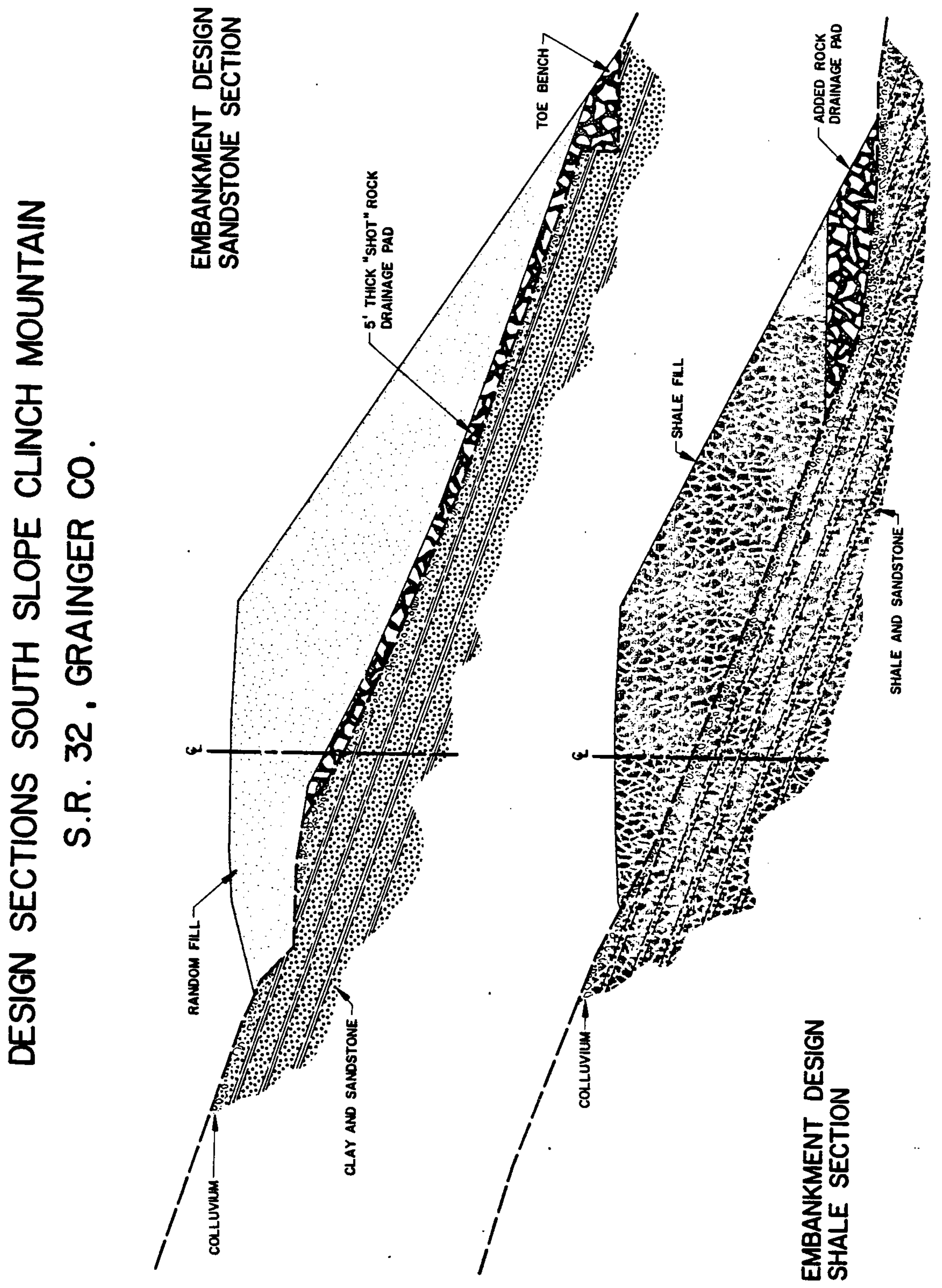

DESIGN SECTIONS SOUTH SLOPE CLINCH MOUNTAIN
S.R. 32, GRAINGER CO.
EMBANKMENT DESIGN
SANDSTONE SECTION
TOE BENCH
5' THICK "SHOT" ROCK DRAINAGE PAD
RANDOM FILL
COLLUVIUM
CLAY AND SANDSTONE
ADDED ROCK DRAINAGE PAD
SHALE FILL
SHALE AND SANDSTONE
COLLUVIUM
EMBANKMENT DESIGN
SHALE SECTION

FIGURE 10

4. All embankments constructed with 1.5:1 slopes to be constructed from rock.

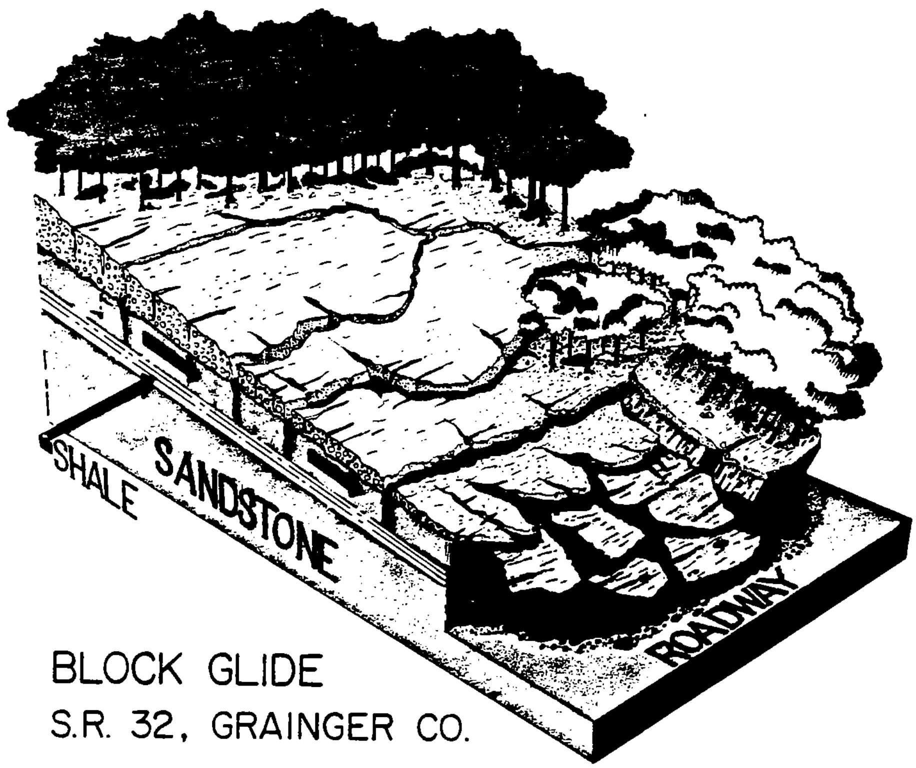
EXPERIENCES DURING CONSTRUCTION - SOUTHEAST SLOPE

Construction of the project on the southeast slope was begun September 13, 1976. Early excavation revealed that the massive Clinch Sandstone was not so massive and stable as concluded from the coring and preliminary field work. The upper lifts were quite friable and severely jointed. Occasional cross faults tilting the strata at more acute downslope angles were discovered. The interbedded shale was more decomposed and saturated with visible water seeping at the contact with the overlying sandstone strata.

Embankments on the southeast slope range up to 27.4m (90') in depth. Every possible precaution was taken to stabilize the embankment foundations. The colluvium in ravines was stripped and replaced with free draining select sandstone from cut excavations. Toe benches were secured in stratified material, although this required more undercutting than originally thought necessary. Rock drainage pads were added in the shale section where none were scheduled. Shale embankment slopes were flattened as much as right of way would permit (from 1.5:1 to 1.8:1). It is believed that if embankment failure does happen it will be a result of failure in the underlying stratified material, perhaps downslope where erosion has truncated a key stratum and left it unsupported. As in the cuts, this potential failure would also be planar, but would be much more difficult and expensive to repair.

During the winter of 1977-1978, planar block glide failures occurred in two of the largest sandstone cuts on the project. These failures were massive and involved most of the backslope down to the lowermost bench. The first failure involved the cut at Station 320+00. The second slide occurred in the interval Station 363+00 to 369+00. Slope stability had been made precarious by a well-developed cross fault which had tilted the north half of the cut at a more precipitous angle toward the roadway. During the winter of 1977-1978 and the following spring and summer, planar failure of some magnitude was precipitated in virtually all shale backslopes to the left of centerline beyond Station 373+00. The decision to flatten the planned vertical slopes to 1.5:1 early in construction proved ineffective, as slides occurred in the repaired intervals (Figure 11).

It became apparent that all the slopes to the left of centerline were potential slides which at some future time may fail and encroach onto the traveled roadway. Several remedial concepts including deep rock anchors and/or "shot-in-place" parallel drainage trenches were considered. The great number of closely spaced, high angle joint sets precluded the use of rock bolts or anchors and the effect of the "shot-in-place" drainage trenches could not be quantified. At the suggestion of David Royster, Director, Division of Soils and Geological Engineering, it was decided, on an experimental basis, to construct a "shot-in-place" rock buttress in one



(David L. Royster, 1978)

FIGURE 11

of the failed sandstone cuts. Based on the success or failure of this experiment, a decision would be made as to similar treatment of other critical backslopes. The "shot-in-place" area was calculated to conform to the configuration of a hypothetical buttress that would be designed for restraint of a cut of equal magnitude. The slope could be "shot" and dressed to a 1.5:1 configuration and the excess swell could then be hauled to the shale interval beyond Station 363+00 and used there in the construction of rock buttresses. Shot holes were drilled on 1.83m (6') centers to a predetermined plane just above grade. This plane slightly undercuts the natural angle of dip, but is sloped to drain toward the roadway (Figure 12).

Since the first experiment seemed successful, it was decided to subject other failed slopes in the sandstone cuts to the same treatment. The decision to form the buttress in place in slopes which had not failed was based on the prevalence of decomposed shale interbeds, the size (height) of the cut, and other features such as the degree of joint development and displaced shot holes from earlier blasting. To date a total of three "shot-in-place" buttresses have been constructed with at least early success. The excess rock was then used to construct three buttresses in failed slopes in the shale interval. If time proves this treatment to be successful, a tremendous savings will have been effected, as the cost of off-site rock borrow hauled to the project for buttress construction would have been prohibitive.

CONCLUSION

The goal, in this complex project, is to catch all failures, actual and potential, during the construction phase. In light of the continuing repair work, both major and minor, required through the years on Interstate 75 in Campbell County and Interstate 40 in Cocke and Roane Counties, this goal is paramount. Remedial work will never be less costly than at the present time. A concerted and dedicated effort is being put forth by the Construction Division and the Division of Soils and Geological Engineering of the Tennessee Department of Transportation to see that costly, long range stability problems are not experienced during the future life of State Route 32-Appalachian Corridor "S".

EPILOGUE

Since the preparation of the original text in October, 1978, the project on the southeast side of Clinch Mountain has been completed. The finishing date was 10 July, 1980, and the roadway is now open to traffic. The "shot-in-place" buttresses have remained intact and are functioning well as are the other remedial design elements. To date, only routine maintainence has been required.

The state of affairs is not as reassuring on the northwest slope project. The process of construction is limping

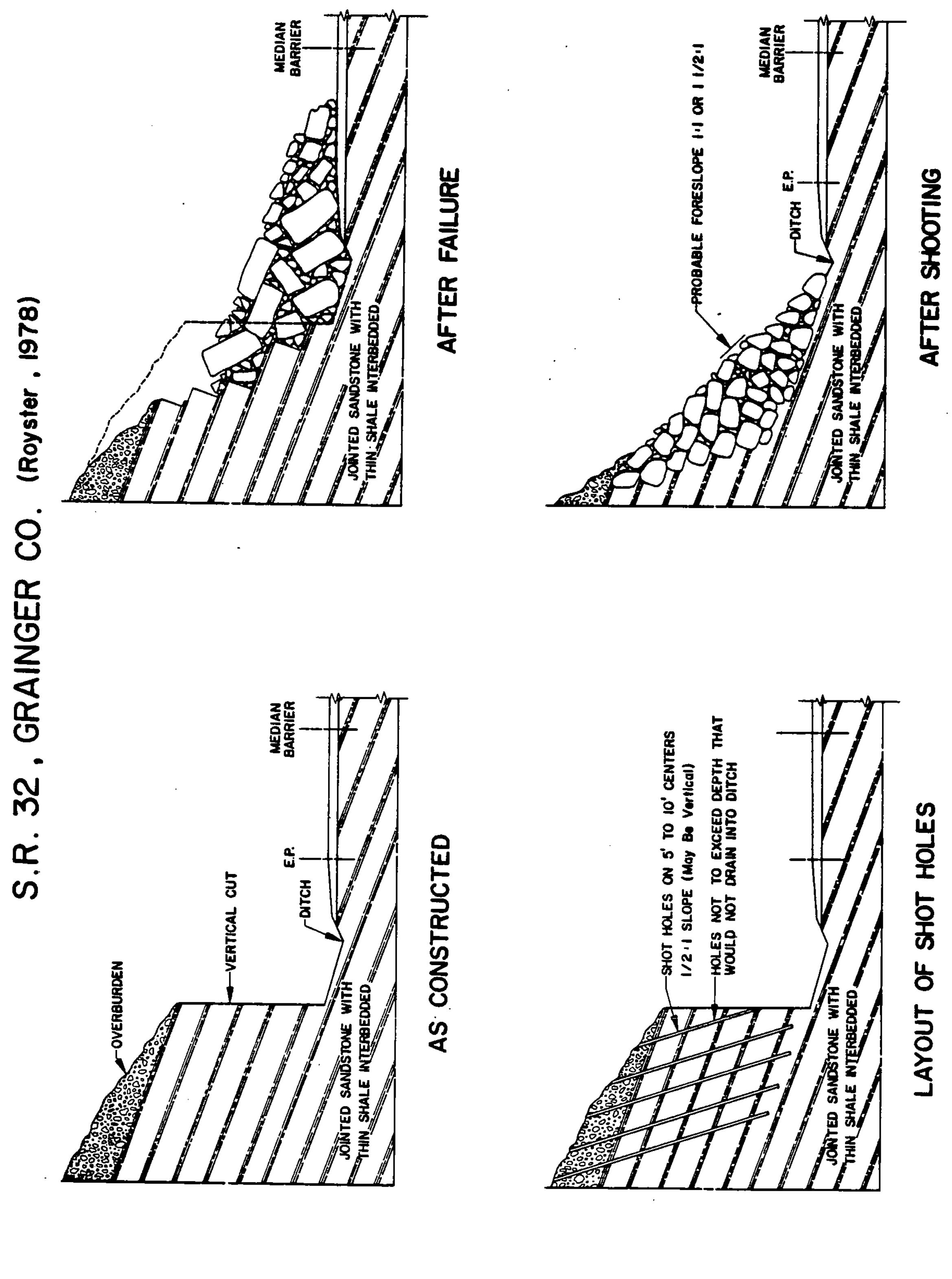

S.R. 32, GRAINGER CO. (Royster, 1978)
OVERBURDEN
VERTICAL CUT
DITCH
E.P.
MEDIAN BARRIER
JOINTED SANDSTONE WITH THIN SHALE INTERBEDDED
AS CONSTRUCTED
MEDIAN BARRIER
JOINTED SANDSTONE WITH THIN SHALE INTERBEDDED
AFTER FAILURE
SHOT HOLES ON 5' TO 10' CENTERS 1/2:1 SLOPE (May Be Vertical)
HOLES NOT TO EXCEED DEPTH THAT WOULD NOT DRAIN INTO DITCH
JOINTED SANDSTONE WITH THIN SHALE INTERBEDDED
LAYOUT OF SHOT HOLES
PROBABLE FORESLOPE 1:1 OR 1 1/2:1
DITCH
E.P.
MEDIAN BARRIER
JOINTED SANDSTONE WITH THIN SHALE INTERBEDDED
AFTER SHOOTING

FIGURE 12

along slowly with no certain tiemtable for ending this project. Several design changes have been required with the most significant being the flattening of the two largest cut slopes in the steep topography nearest Beans Gap. The original design for these cuts specified 9.15m (30') vertical lifts separated by 6.1m (20') wide in-slope benches. The slopes began to deteriorate during construction with movement accelerating after the cut was brought to grade. Stress relief resulted in marked separation along the closely spaced joint systems creating a serious rockfall problem. Wedge failures removed large segments of the in-slope benches creating an undulating slope face. The prominences of this face functioned as inclined planes which directed falling rock into the uncompleted roadway section.

With such large cuts-exceeding 48.8m (160') in height-the only viable solution to the rockfall problem was to flatten the slopes, but in this area of precipitous natural slopes a 1:1 ratio was the flattest that could be achieved. This ratio, however, was too steep still to eleminate the rockfall problem. The final decision was to utilize 1:1 backslopes with a 12.2m (40') wide catchment area at grade. At this writing, the "en masse" stability of the slopes seems assured but the rockfall problem continues. It appears that catchment fences established in the fallout area will be required in order to prevent rock fragments from being launched into the traffic lanes.

TEMPORARY LANDSLIDE CORRECTIVE TECHNIQUES AVERT CATASTROPHE

BY

HENRY MATHIS (1)

INTRODUCTION

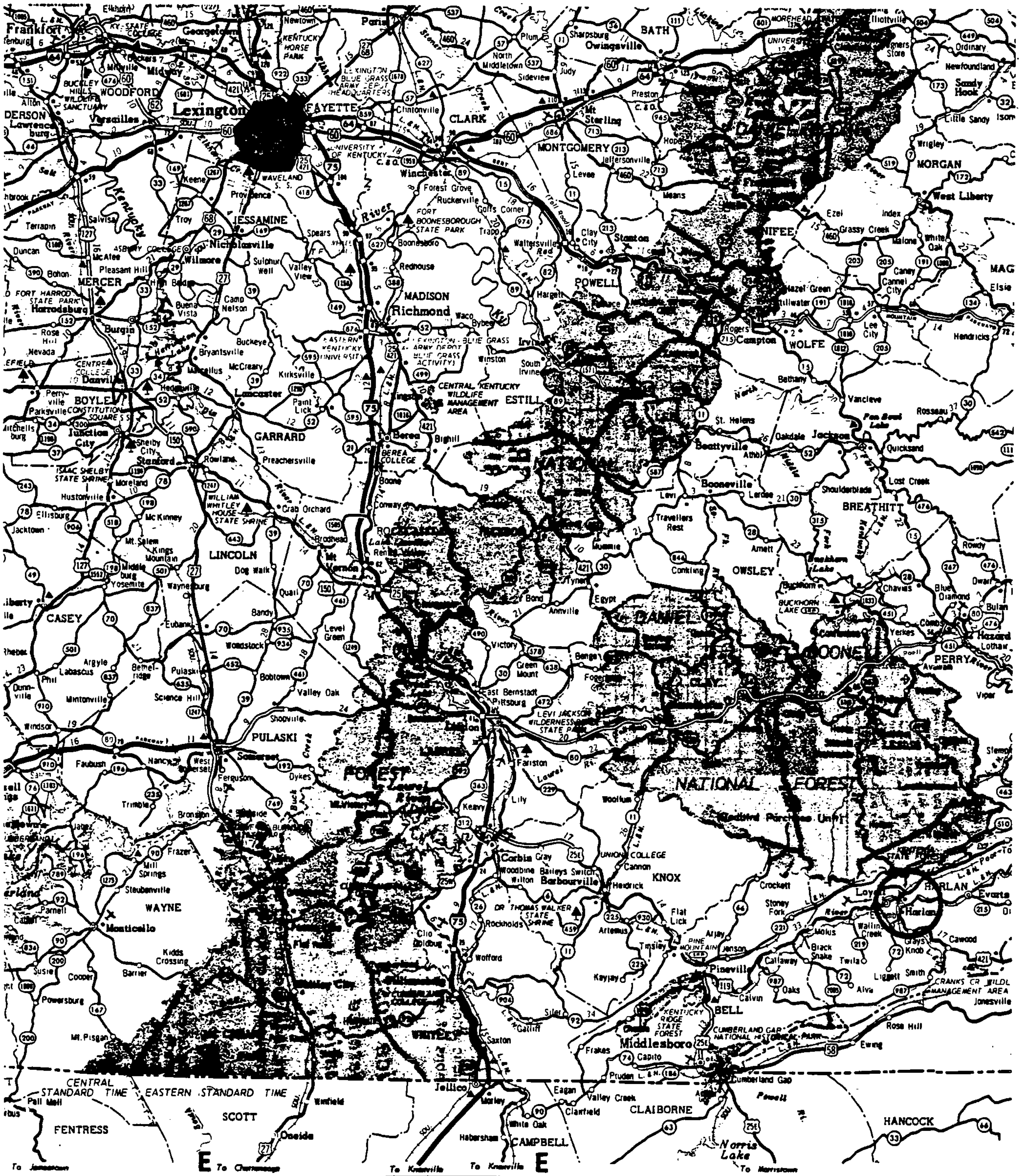
The subject landslide occurred during the construction of U.S. 421 in Harlan, Kentucky. The failure occurred in a cut section of the roadway between stations 154+00 and 161+00 with the main scarp located 275 feet to the right of the new construction centerline. The major concern about this landslide was its effect on a 10 inch water line located in the ditch line of existing U.S. 421. This water line serves the southern portion of Harlan and a hospital located a few miles south of this project. Interruption of service in the water line for an extended length of time would be a catastrophe because this is the only source of water for the area.

The catastrophe was adverted by constructing a small temporary stabilization berm at the toe of the landslide and installing railroad rails in the shoulder of existing U.S. 421 to slow down the landslide movement until a permanent correction could be constructed. This case history is unique because the Geotechnical Section was directly involved in one of the landslide corrective methods by drilling holes for the railroad rails. Subsurface data obtained from this operation was utilized in designing a permanent correction for the landslide.

PROJECT FACTS

The project is located in Harlan County between Harlan and Dressen. Refer to Figure 1 for the location map. Highway U.S. 421 is being upgraded between Harlan and the Virginia State Line. This project is a portion of the new construction which begins approximately 0.6 mile south of the Harlan City limits at station 127+50 and extends northerly for approximately 0.9 mile to station 188+10. This is a four lane facility with a 20 foot mountable median. The type of terrain encountered in this 0.9 mile is classified as light mountaineous.

(1) Assistant Director of Materials
Geotechnical Section
Kentucky Dept. of Transportation



Location Map

FIG.1

Utilizing fifty percent federal and fifty percent state funds, this project, grade, drain and flexible pavement, was awarded on April 17, 1978 for a total contract price of $5,078,098.

ORIGINAL DESIGN

Original design for this project took place in the early part of the 70's with the geotechnical investigation being conducted during 1970 and 1971.

The limits of the cut section in which the landslide occurred was between stations 155+00 and 164+50 with a maximum cut depth of 41 feet encountered at station 162+50. From the beginning of the cut to station 158+50 the roadway cross sections depicted a 2 horizontal to 1 vertical slope, however, in the deeper part of the cut, stations 159+00 to 164+50, the plans specified the cut slope to be 1 1/2 horizontal to 1 vertical. Refer to Figure 2 for the original design sections at stations 158+00 and 159+00.

Original design earthwork quantities and contract bid prices are as follows:

Roadway Excavation (Unclassified)	*$	547,978 yd.^{3} at $3.40 yd.^{3}
Solid Rock Borrow	$	85,443 yd.^{3} at $4.00 yd.^{3}

*(273,423 yd.^{3} of the roadway excavation was waste.)

HISTORY

In December 1978, the Geotechnical Section received a request from the District Construction Personnel to inspect a small slope failure in a newly constructed cut slope on U.S. 421. At the inspection a shallow cut failure was observed between stations 157+00 and 158+50, and a considerable amount of water was observed flowing from an area midway up the slope. The cut depth at these stations was 14 and 19 feet respectively.

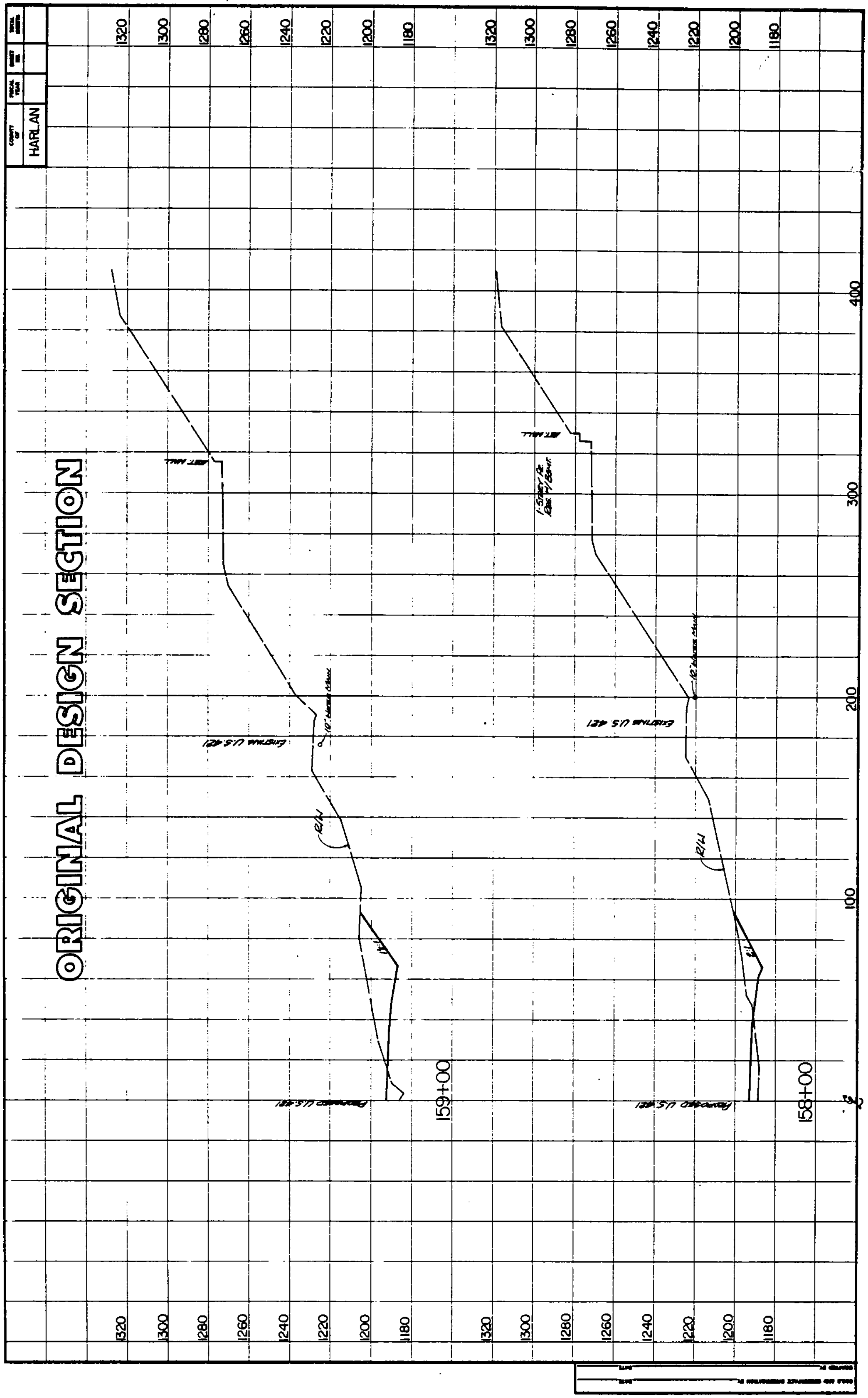

ORIGINAL DESIGN SECTION
HARLAN
159+00
158+00
R/W
1320
1300
1280
1260
1240
1220
1200
1180
100
200
300
400

FIG. 2

The following recommendations were suggested at the inspection:

1/. Construction in the immediate area should be stopped.

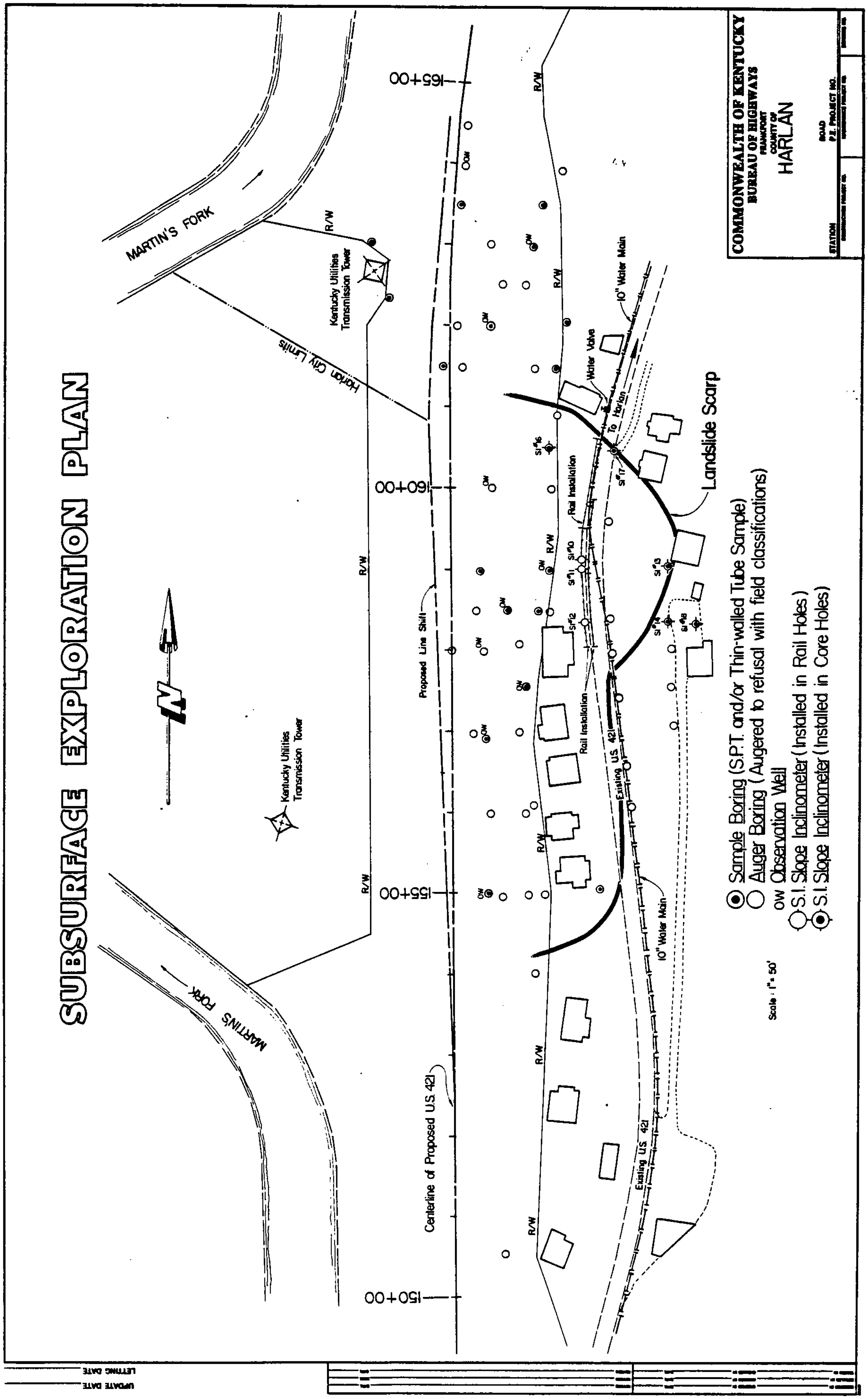
2/. A small temporary stabilization berm should be constructed in the ditch line of the cut as soon as possible between stations 157+00 and 158+50 to offer resistance to the movement in the cut slope.

3/. The Resident Engineer should employ survey methods to monitor the surface movement in the area.

4/. A geotechnical investigation should be made immediately to determine the type of corrections necessary for the cut slope failures.

On February 23, 1979, the Geotechnical Section received an emergency request for immediate assistance as the small cut slope failure had propogated back beyond the toe of the cut slope off of the highway right-of-way and had endangered some dwellings. The major concern, however, was the displacement and tension cracks that had developed in the pavement of existing U.S. 421 affecting a ten inch water line located in the ditch line of the existing roadway. Refer to Figure 3 for the location of the water line. This water line serves a hospital and the southern portion of Harlan, therefore, the line could not be out of service for any extended length of time without causing a catastrophe. Several ruptures did occur in the water line, however, they were repaired and did not disrupt service for any significant length of time.

In order to minimize damage to the existing roadway, water line, and dwellings, it was decided to install railroad rails in the critical area of the landslide. This temporary correction was an economical and expedient attempt to slow down the movement until a permanent correction could be implemented. We suggested that our two crews perform the drilling operations and Maintenance personnel install the rails. The subsurface data obtained from the drilling operations was utilized in analyzing the landslide.



SUBSURFACE EXPLORATION PLAN
MARTIN'S FORK
MARTIN'S FORK
Kentucky Utilities Transmission Tower
Kentucky Utilities Transmission Tower
Harlan City Limits
R/W
N
Proposed Line Shift
Centerline of Proposed U.S. 421
165+00
160+00
155+00
150+00
Rail Installation
Rail Installation
Existing U.S. 421
Existing U.S. 421
10" Water Main
10" Water Main
Water Valve
To Harlan
Landslide Scarp
Sample Boring (S.P.T. and/or Thin-walled Tube Sample)
Auger Boring (Augered to refusal with field classifications)
OW Observation Well
S.I. Slope Inclinometer (Installed in Rail Holes)
S.I. Slope Inclinometer (Installed in Core Holes)
Scale : 1" = 50'
COMMONWEALTH OF KENTUCKY
BUREAU OF HIGHWAYS
FRANKFORT
COUNTY OF
HARLAN

FIG. 3

On February 27, 1979, the two drill crews began drilling operations working alternately on twelve hour shifts. The rails were installed in 10 inch diameter holes drilled into rock with the first row located in the shoulder of the existing roadway between stations 158+00 and 159+84 spaced on 4 foot centers. A second row of rails installed between stations 158+00 and 159+84 were also located in the shoulder of the existing roadway spaced on 4 foot centers. However, these were staggered with the ones in the first row for an effective spacing of 2 foot centers with the first row. Refer to Figure 3 for the location of the rail installations and Figure 4 for a typical section.

The landslide continued to progress and by the middle of March, 1979, the small cut slope failure had developed into a landslide of major proportions. The scarp continued to propogate up the slope from the existing roadway and damaged additional dwellings. At this time, five dwellings below the existing roadway and one located on the hillside above the roadway was damaged. Refer to Figure 3 for the main scarp location.

No appreciable movement was observed between the period of March 14, to March 31, 1979. On April 2, 1979 the Resident Engineer was informed that the basement in one of the dwellings had developed cracks and water had begun to leak into the basement. Therefore, an additional row of rails were installed in the shoulder on 4 foot centers between stations 159+84 and 160+60. Also, the small stabilization berm constructed in the ditch line of the new roadway (toe of the landslide) was extended from stations 158+50 to 161+00. The berm and rail installations were completed on April 18, 1981. Slope inclinometer data indicated the movement had slowed down a considerable amount upon completion of the temporary corrective methods.

PHYSIOGRAPHY

The subject project is located in the Eastern Coal Field physiographic region of Kentucky. This region is a maturely dissected plateau of varying altitude and relief. In this region a dendritic drainage pattern has developed. Deep, narrow, v-shaped valleys and irregular winding, narrow crested ridges dominate the topography. Local relief is approximately 2,000 feet with elevations ranging from 1150 feet to 3200 feet.

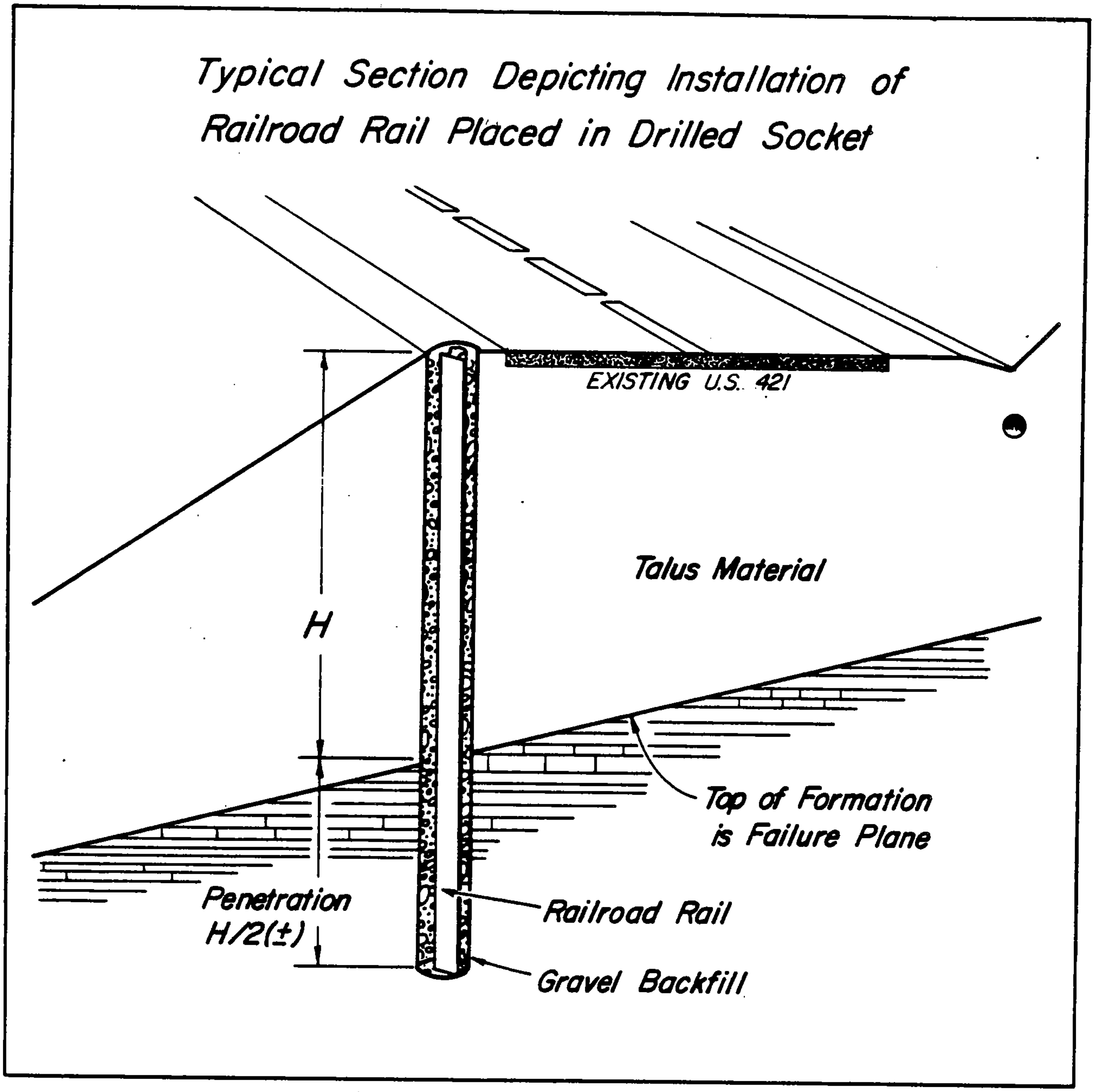

Typical Section Depicting Installation of
Railroad Rail Placed in Drilled Socket
EXISTING U.S. 421
Talus Material
H
Top of Formation
is Failure Plane
Penetration
H/2(±)
Railroad Rail
Gravel Backfill

FIG. 4

GEOLOGY

Rocks outcropping in the landslide area are mapped as the Lower Series of the Pennsylvania System, namely the Hance Formation of the Breathitt Group. Sandstones, siltstones, and shales interbedded with coal generally form the hillsides at the base of the mountains throughout the Harlan area.

The colluvium deposit at the base of the mountain was one of the main factors causing the landslide. The colluvium consists mainly of angular sandstone blocks, and boulders in sand and clay matrix. This deposit is material that has weathered on the slopes of the mountain and moved down the slope to the base by gravitational force, therefore, the colluvium deposits are very loose and unconsolidated. Also, the dip of the rock strata in the formations has a very general dip toward the centerline of the new roadway, which permits ground water to percolate into the colluvium deposit resulting in an unstable mass.

Field observations and subsurface data indicated the cut slopes that failed between stations 155+00 and 161+00 was constructed in colluvium material. The remainder of the cut slopes that did not fail were constructed in highly weathered sandstones interbedded with shales.

DRILLING AND SAMPLING

Drilling and sampling for the landslide began in December, 1978 using one of the Geotechnical Sections' drill crews. Equipment used for the subsurface investigation and drilling operations for the railroad rails included the following:

Mobile B-53 Mounted on an All-Terrain Vehicle
Mobile B-40 Mounted on a John Deere-450 Dozer
Mobile B-61 Mounted on a Truck

Holes for the railroad rail installations were drilled with the truck mounted drill rig using 10 inch solid stem augers and the data concerning the depth to rock and water elevations were recorded.

Sampling the colluvium material was very difficult due to the presences of boulders, however, a total of 23 thin-walled tubes and 16 standard penetration test samples were obtained from 14 borings. In addition, 5 rock cores were obtained for a total of 64 feet of cores. Also obtained were 32 auger borings to define rock line. This is a total of 51 borings not counting the holes drilled for the railroad rails. Locations of the borings are shown on Figure 3.

One inch p.v.c. perforated pipe was installed in 11 of the borings in order to monitor the water table in the landslide area. These observation well locations are depicted on Figure 3.

LABORATORY TESTING

A. Disturbed Testing

Moisture content, unit weights, and tests for classifications were performed on selected samples obtained from the thin-walled tubes and standard penetration tests. Predominantly the soils classified ML-CL with an average moisture content of 18 percent and the average wet unit weight of 130 pounds per cubic foot.

Summary of Disturbed Soil Test Results

Station	Location	Depth	Moisture Content (%)	Wet Density (pcf)	Classification Unified	Classification AASHTO
155+00	44'Rt.	15-17'	23.5		ML-CL	A-6 (8)
		20-22'	29.9		ML-CL	A-4 (4)
155+00	180'Rt.	5-7'	14.8	132.8		
156+90	41'Rt.	10-12'	22.7			
158+00	40'Rt.	5-7'	15.8	128.6	ML-CL	A-4 (2)
		10-12'	27.7	122.3	ML-CL	A-7-6 (17)

Station	Location	Depth	Moisture Content (%)	Wet Density (pcf)	Classification Unified	AASHTO
158+35	300'Rt.	3 1/2-5'	12.8	130.2	CL	A-4 (3)
		8 1/2-10 1/2'	11.0	128.1		
		13 1/2-14 1/2'	13.3	137.6		
		15 1/2-17 1/2'	12.5	128.4		
		18 1/2-20 1/2'	17.6	136.8		
158+50	105'Rt.	4-6'	8.8	125.6		
		9-11'	16.6	138.8		
159+00	35'Rt.	0-2'	21.6		SM-SC	A-4 (1)
		5-7'	23.8		ML-CL	A-6 (6)
159+00	120'Rt.	2-4'	13.5		GM-GC	A-4 (1)
		4-5'	14.1		ML-CL	A-4 (3)
		9-11'	15.4		ML-CL	A-4 (8)
		14-16'	17.8		ML-CL	A-4 (2)
161+50	15'Lt.	5-7'	34.6	121.9	ML-CL	A-4 (4)
			16.1		ML-CL	A-4 (3)

Slake Durability Tests were also performed on rock core samples obtained from the shale underlying the colluvial material and from representative samples of shale from roadway excavation designated for the rock drainage blanket. The results ranged from a low of 95 percent to a high of 98 percent. Therefore, the shale classified as rock - like.

B. Undisturbed Testing

Four consolidated undrained triaxial tests with pore pressure measurements were performed on selected thin-walled tube samples from the landslide area. The results of the tests are shown below:

Summary of Undisturbed Soil Test Results

Station	Location	Depth	$\overline{\Phi}$ (Degrees)	$\overline{C}$ (psf)
155+00	44'Rt.	15-17'	35°	90
		20-22'		
158+00	40'Rt.	10-12'	34°	0

Station	Location	Depth	$\overline{\Phi}$ (Degrees)	$\overline{C}$ (psf)
158+35	300'Rt.	3 1/2-5' 8 1/2-10 1/2' 13 1/2-14 1/2'	38	0
159+00	120'Rt.	14-16'	26	480

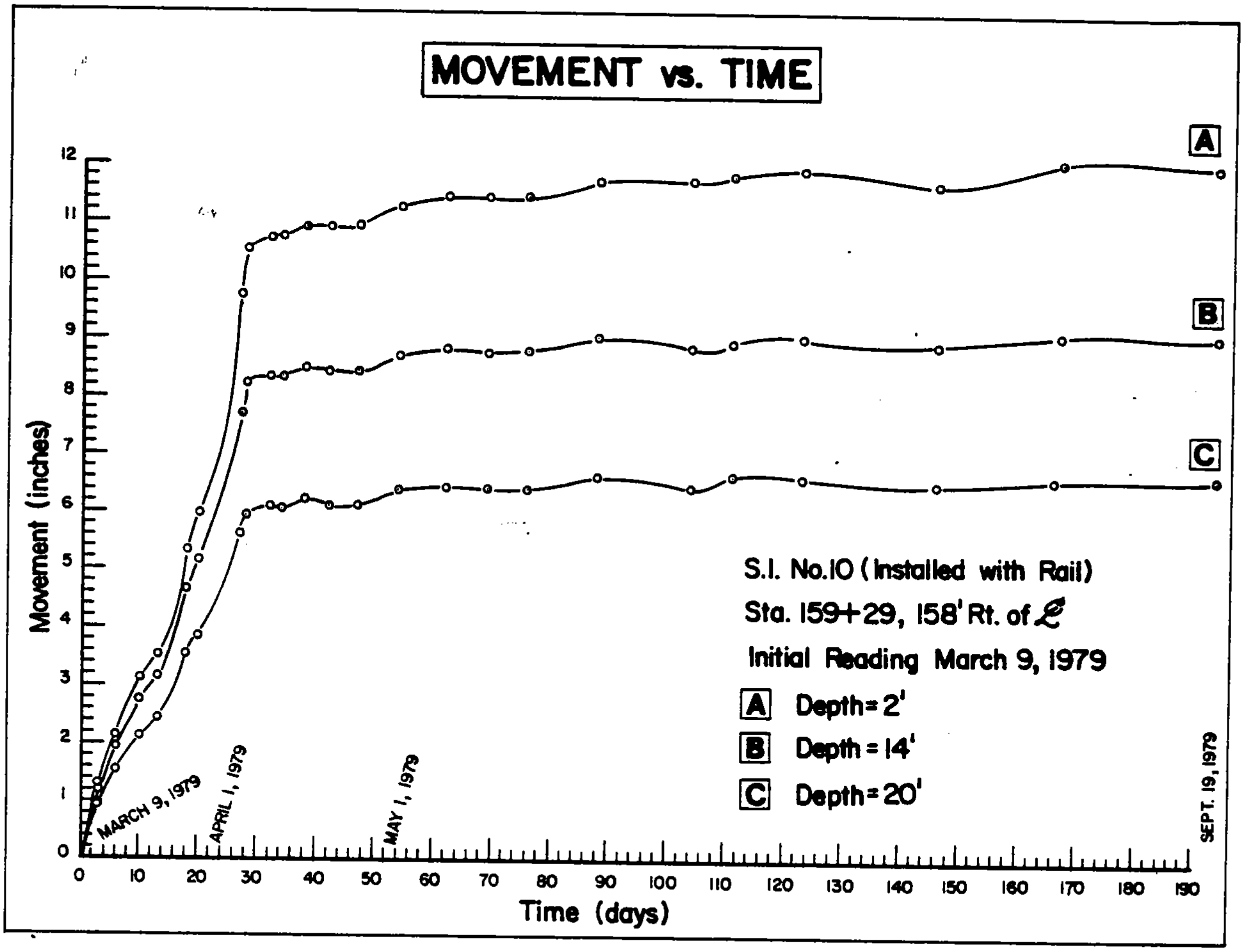
SLOPE INCLINOMETERS

Eight slope inclinometers (numbers 10, 11, 12, 13, 14, 16, 17, and 18) were installed at various locations in the landslide to monitor the depth, magnitude, and direction of movement. Refer to Figure 3 for these locations.

Slope inclinometer numbers 10, 11, and 12 were installed in the holes drilled for the railroad rails in order to monitor the effectiveness of the rails and the small stabilization berm until a permanent correction could be constructed. The depth of movement on these inclinometers was located at the colluvium - bedrock contact. A plot of the Movement vs. Time for slope inclinometer numbers 10 and 12 indicated an accelerated movement from the first of March until the first of April. After the first of April the movement decreased substantially in the area when the second row of rails and small stabilization berm was completed. The plots indicated the temporary corrections were very successful in slowing down the landslide movement, thus avoiding any long term interruption of water service. Slope inclinometer number 11 closed off six days after installation, therefore, sufficient data was not available for a Movement vs. Time plot. Refer to Figure 5 for the Movement vs. Time Plots.

Slope inclinometer number 16 was located below the existing roadway 120 feet right of station 160+50. This inclinometer also served the purpose of monitoring the effectiveness of the rails and small stabilization berm. Installed on April 1, 1979 this one showed rapid acceleration during the first eighteen days, however, the movement decreased substantially over the next 73 days. This inclinometer also showed the success of the temporary corrections. A total amount of approximately 0.7 of an inch has been recorded to date.

Slope inclinometer number 17 was located in a driveway adjacent to the Smith House. Refer to Figure 3 for the location. As you can see from the drawing the inclinometer was located on the right flank of the landslide, therefore, no significant movement was noted in this inclinometer.


MOVEMENT vs. TIME
S.I. No.10 (Installed with Rail)
Sta. 159+29, 158' Rt. of ℄
Initial Reading March 9, 1979
A Depth = 2'
B Depth = 14'
C Depth = 20'
A
B
C
MARCH 9, 1979
APRIL 1, 1979
MAY 1, 1979
SEPT. 19, 1979
Movement (inches)
Time (days)
0
1
2
3
4
5
6
7
8
9
10
11
12
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190

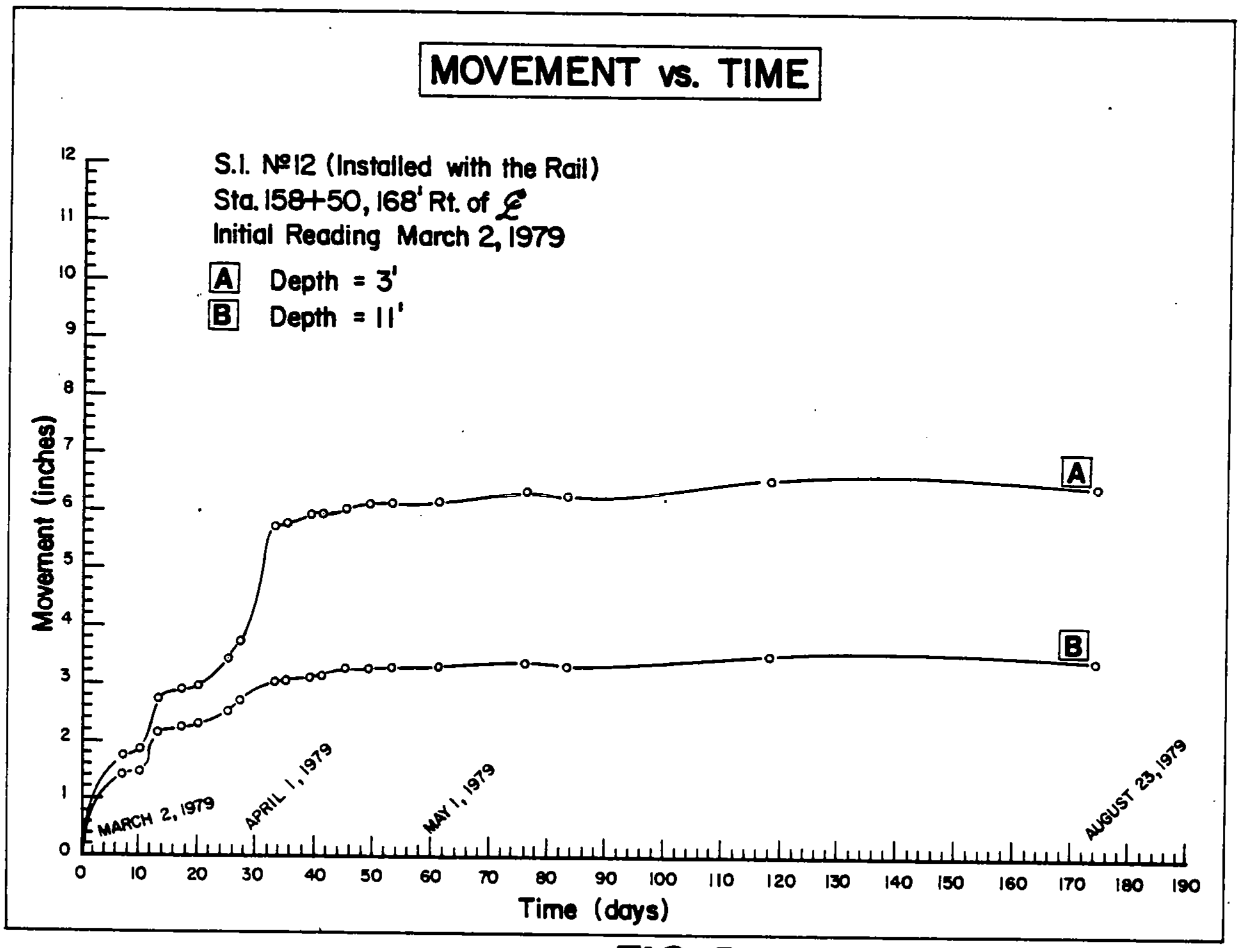

MOVEMENT vs. TIME
S.I. №12 (Installed with the Rail)
Sta. 158+50, 168' Rt. of ℄
Initial Reading March 2, 1979
A Depth = 3'
B Depth = 11'
A
B
MARCH 2, 1979
APRIL 1, 1979
MAY 1, 1979
AUGUST 23, 1979
Movement (inches)
Time (days)
0
1
2
3
4
5
6
7
8
9
10
11
12
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190

 FIG. 5

Slope inclinometer numbers 13, 14, and 18 are located above the existing roadway in front of two houses. Refer to Figure 3 for the locations. Numbers 13 and 14 are located at the head of the scarp and 18 was located behind the scarp. Total movements of approximately 1 1/2 inch and 1 inch respectively were recorded for slope inclinometer numbers 13 and 14. Significant movement occurred at a depth of approximately 22 feet in both casings. This is the approximate contact between the colluvium and the bedrock. Refer to Figure 6 for the boring data and slope inclinometer numbers 12, 14 and 18. No significant movement has been noted in slope inclinometer number 18 as of October 29, 1980.

The resultant movement recorded in all of the slope inclinometer was in the direction of the excavation for the cut slope.

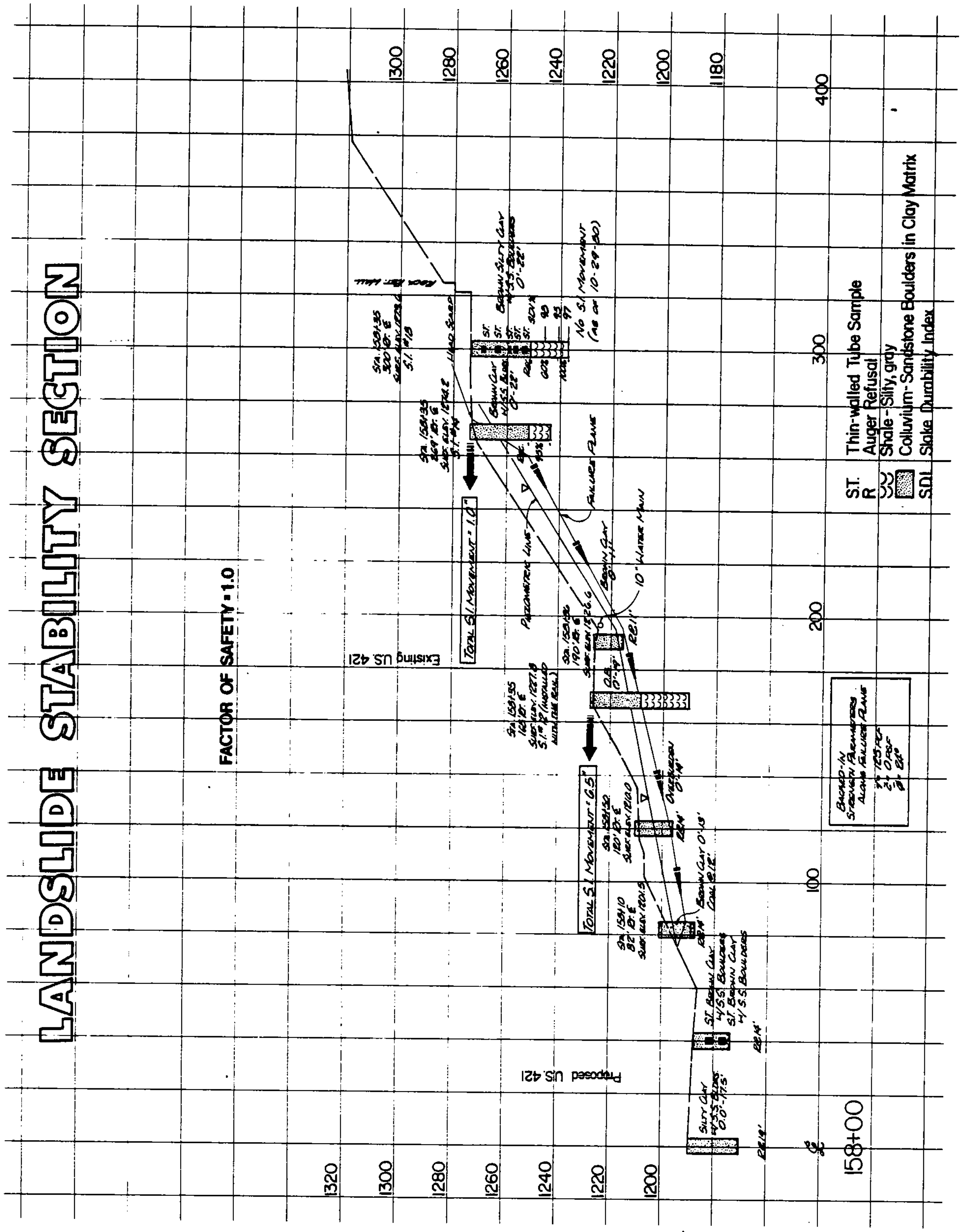
STABILITY ANALYSES

A. Existing Conditions at Failure

Three critical cross sections were chosen for stability analysis, however, only the one for station 158+00 will be discussed in this report because this one is the most critical. The head scarp on this section is located approximately 275 feet right of the centerline above the existing roadway. The slope inclinometer on this section indicated the movement to be located along the contact between the colluvium and bedrock.

Utilizing the slope inclinometer data, field observations of the scarps, and water table information, the shear strength parameters along the failure plane were calculated by setting the effective cohesion equal to zero and varying the internal friction angle until a factor of safety of 1.0 was obtained. Since the type of failure was a wedge, a computerized version of a translation failure analysis shown in NAVFAC, DM-7, Design Manual, "Soil Mechanics, Foundations, and Earth Structures", was utilized. Refer to Figure 6 for the landslide stability section.

The shear strength parameters along the failure plane for this analysis, and for the two other sections not shown in this report are as follows:



LANDSLIDE STABILITY SECTION
FACTOR OF SAFETY = 1.0
Existing U.S. 421
Proposed U.S. 421
158+00
100
200
300
400
1320
1300
1280
1260
1240
1220
1200
1180
ST. Thin-walled Tube Sample
R Auger Refusal
Shale - Silty, gray
Colluvium - Sandstone Boulders in Clay Matrix
SDI Slake Durability Index

 FIG. 6

Station	Unit Weight (psf)	$\overline{C}$ (psf)	$\overline{\Phi}$ (Degrees)
155+00	125	0	22
158+00	125	0	24
160+50	125	0	24

B. Recommended Corrections

Several factors were considered in choosing the most economical and feasible type of correction. Probably the most important consideration was the fact that traffic had to be maintained on existing U.S. 421 and this facility was planned to serve local traffic after the new roadway was in operation. Another consideration was a quantity of waste material was available on the project from adjacent roadway excavation. Considerating these factors it was decided to restrain the driving forces utilizing a stabilization berm at the toe of the landslide. In order for this to be accomplished the grade of the new roadway had to be raised (maximum 6 feet) and the centerline had to be shifted (maximum 21 feet) away from the landslide in order to provide space for the berm construction.

Other types of corrective measures considered were the construction of a retaining wall, reducing the driving forces by unloading the landslide and dewatering. These methods were not considered as feasible and/or economical as the method chosen.

In order to size the berm, analyses were performed on the same three critical cross sections referred to previously, however, only the one for station 158+00 will be shown. The new configurations, revised alignment and grade, were used in the analysis and the following conditions were assumed:

1. The most critical failure plane was established by slope instrumentation measurements, field observation of the toe of the landslide and the scarps. The shear strength along the failure plane is the same as determined from the analysis of existing conditions at failure.

2. Water will drain out of the rock drainage blanket material and thus prevent the rise of the water above the elevations assumed in this analysis.

Slope stability analysis, translational mode of failure, using a two horizontal to one vertical slope for the stabilization berm yielded an overall factor of safety of 1.3. The factor of safety for the berm slope assuming a rotational mode of failure was 1.5 using the ICES-LEASE computerized slope stability program. The factor of safety for the overall stability was lower than we normally accept, however, we were conservative in our assumption of the strength parameters and neglected any strength gain as a result of the railroad rails used for the temporary correction. Refer to Figure 7 for details concerning the proposed correction and slope stability analysis.

RECOMMENDATIONS

The following recommendations were suggested for the correction of the landslide:

1. Raise the grade and shift the roadway alignment in order to provide a sufficient area for the stabilization berm.

2. Construct a metal bin type retaining wall around the Transmission tower located left of station 162+50. The wall shall begin at station 162+10 and extend to station 163+00. The wall is necessary to accommodate the revisions in the alignment and grade.

3. Construct a stabilization berm as shown in Figure 6. The berm material may be soil and/or rock and shall be compacted to the same specification as the roadway embankment.

4. Place a 3 foot rock drainage blanket between the existing ground and the stabilization berm. The material for this drainage blanket shall be sandstone and/or rock - like shale. A perforated pipe shall be placed in the ditchline of the new roadway to collect the water from the drainage blanket. Refer to Figure 7 for details.

LANDSLIDE STABILITY SECTION

RECOMMENDED CORRECTION

ANALYSIS	SAFETY FACTOR	SLOPE	CONDITION
WEDGE	1.3	2:1	OVERALL STABILITY
CIRCLE	1.5	2:1	BERM STABILITY

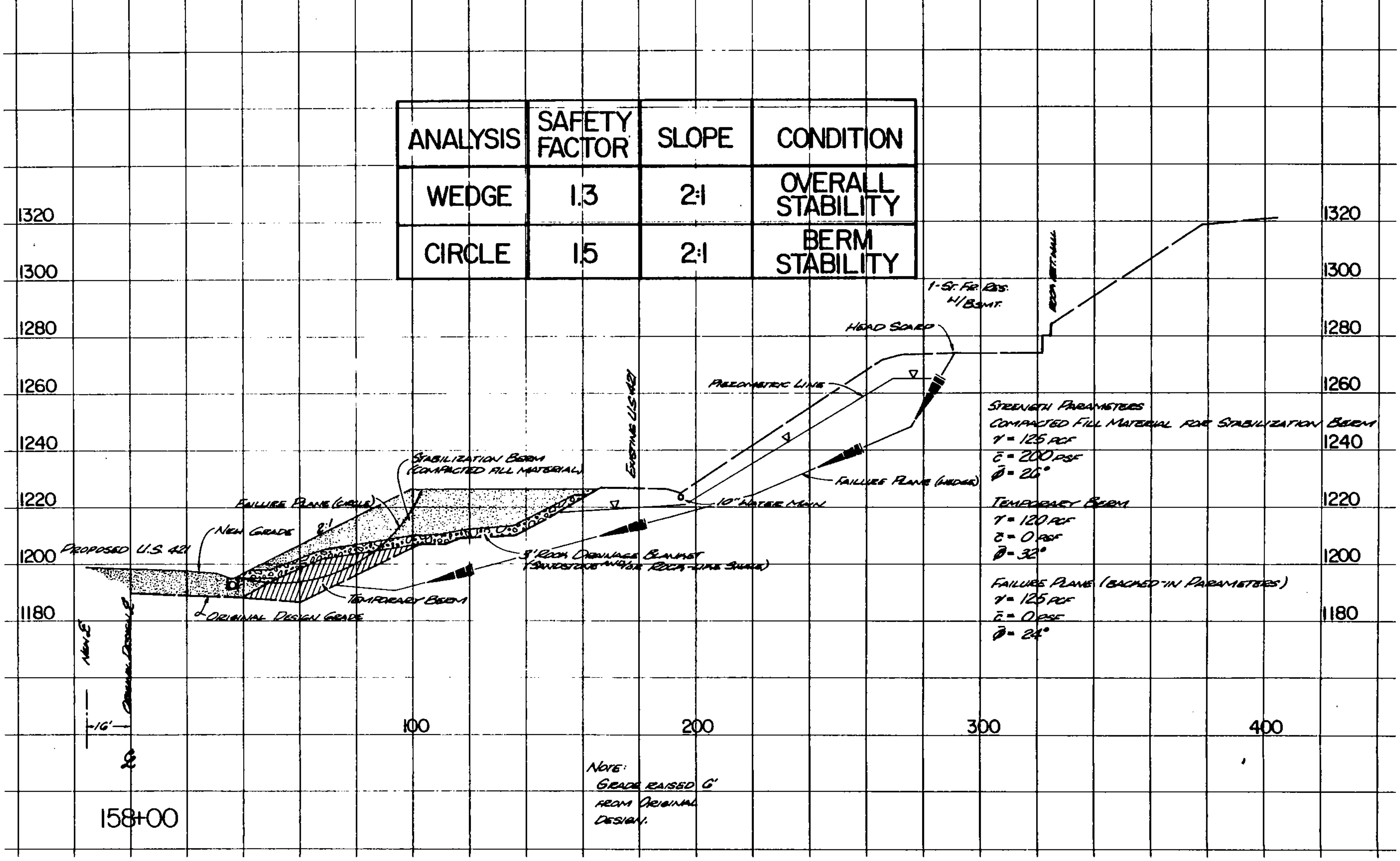


 FIG. 7

LANDSLIDE COST

The total cost of the landslide correction was $315,768 for the construction and $178,400 for the right-of-way. Right-of-way cost included ten dwellings.

CONCLUSION

The cause of the landslide was oversteepened cut slopes constructed in a colluvium deposit during a wet season of the year. Although there were a few borings obtained in this cut during the design phase, adequate consideration was not given to the stability of the cut slope. Even though the cut was small in magnitude a large landslide developed. This case history illustrates that in dealing with colluvium material, slope stability is a very important factor <u>regardless of the size of the excavation</u>, if important structures or facilities could be adversely effected by a failure.

ACKNOWLEDGEMENTS

I would like to express my appreciation to Mr. C. S. Bishop, Geotechnical Engineer, responsible for this landslide investigation. Thanks also to Geotechnical Engineers' Mr. W. E. Munson, and Mr. R. Craycraft and Mr. G. Scott for their assistance. Appreciation is expressed to Mr. L. C. McCord for his work involving the field instrumentation, Mr. W. C. Preston for his graphics, and Mrs. K. R. Gunn for her secretarial work. Also thanks to all of the other Engineers, Geologists, Technicians, and Drillers from the Geotechnical Section, Division of Materials who made this investigation a successful one.

Appreciation is also expressed to Mr. E. L. Middleton, Resident Engineer and Mr. J. W. Stallard, Assistant Resident Engineer for all of their assistance.

REFERENCES

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"Remedial Corrective Measures and State of the Art for Rock Cut Slopes in Eastern Kentucky"

By

E. M. Wright (1)

ABSTRACT

The Kentucky Bureau of Highways utilizes benches in their cut slope configurations. Lithology of the strata, with special consideration for discontinuities and weathering, is considered in the design. Cuts are so designed that they may be modified if unforeseen geological conditions are encountered during construction or maintenance operations. This paper will briefly discuss the cut slope design criteria.

Remedial corrective measures for existing hazardous roadway cuts consists mainly of removal of material by utilizing benches. Four case histories illustrating different types of failures and associated corrective measures are included in the paper.

INTRODUCTION

The geological formations in Kentucky consist of sedimentary rocks, such as sandstone, limestone, shale, siltstone and coal. With the exception of the Pine Mountain Thrust Fault area (Eastern Kentucky) the majority of the formations are essentially flat-lying with local variations of dip.

The state of the art for slope design is based on information available from open face logs, rock cores, soil borings, rock soundings, undisturbed samples, etc. to determine cut slopes, lift heights, bench widths, rock disintegration zones, soil overburden thickness and overburden bench requirements. A geological review of existing cut sections in the general area is conducted during the planning or pre-design phase of highway development. Eact cut is considered independently and is designed according to the lithology with special emphasis on discontinuities and the strike and dip of the beds. There are no inflexible rules for rock cut slopes in formations that contain vertical and/or horizontal changes in lithology; however, the slopes are usually designed to accommodate the worst conditions.

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I. CUT SLOPE DESIGN . . .

Cut slopes in shale are designed according to weathering characteristics indicated by the Kentucky Method 64-513-78 for determination of Slake Durability Index and the Jar Slake Test (Kentucky Method 64-514-77).

Soil-like shale, with or without laminations, will usually be in category 1 or 2 of the Jar Slake Test and have a Slake Durability Index range of 0 to 50 percent. Typical cut slope recommendations will vary from 2 horizontal to 1 vertical (2:1) or flatter to 1 horizontal to 1 vertical (1:1). The 2:1 (or flatter) slope from ground line to grade is normally without a roadside ditch bench, intermediate bench, or overburden bench. Refer to Figure 1.

Intermediate shales will have a Slake Durability Index range from 51 percent to 94 percent and may be in category 3, 4, or 5 of the Jar Slake Tests. Typical cut slope recommendations will vary from 1:1 to 1/2:1 with roadside ditch benches, intermediate benches 15 to 18 feet in width, and approximate lifts of 30 feet depending on rock competency. Refer to Figure 2.

Rock-like shales are usually fissile and very durable with a Slake Durability Index of 95 percent or greater, and the Jar Slake Test results will usually be in category 6. Typical cut slope recommendations will vary from 1/2:1 to 1:20 with roadside ditch benches, intermediate benches 15 to 18 feet wide, and approximate lifts of 30 feet.

Massive bedded limestones or sandstones will have typical cut slopes which vary from 1:20 (vertical) to 1/2:1. The 1:20 slopes may have lift heights up to 60 feet (maximum) with 18 foot intermediate benches; however, special consideration must be given to discontinuities. Refer to Figure 3.

A typical cut slope recommendation for shaly limestone and shaly sandstone is a 1/2:1 slope with lift heights from 30 to 45 feet, and 18 foot intermediate benches. Flatter slopes may be required, depending on the percent of shale present. Refer to Figure 4.

SERRATED SLOPES

Serrated slopes are utilized as a means of controlling erosion and establishing vegetation on soft rock formations, shale, or other material that can be excavated by bulldozing or ripping. Serrations may be recommended for 1:1 or flatter cut slopes. Typical step risers will vary from two to four feet. Refer to Figure 5.

INTERMEDIATE BENCHES

The elevation of most intermediate benches is determined by breaks in lithology, with the bench being on top of the least resistant material. If the slope lift exceeds 60 feet and there are no distinct lithology breaks, an intermediate bench is utilized to divide the cut into equal lifts that fit the terrain and cut slope configuration. Typical bench widths are 18 feet; however, benches may vary from 15 to 25 feet depending upon the purpose for which they are intended. Intermediate bench widths of 20 or 25 feet may be required in situations where (a) shale is expected to weather rapidly and undercut a weak massive-bedded sandstone, (b) coal mine openings are encountered, or (c) other unstable slopes which may have a heavy rock fall.

OVERBURDEN BENCHES

Overburden benches are placed beneath the weathered rock disintegration zone with typical bench being 15 feet wide. In hazardous areas they may be 25 to 50 feet wide depending on subsurface conditions and probability of a failure in the overburden. These benches are drawn flat on cross sections and will have some grade through the cut depending on variations in depth of material. In mountainous terrain where the ground slopes away from the roadway, the overburden bench may be omitted from that side of the cut. Refer to the top of the cut sections shown in Figure 6.

The depth of overburden and disintegrated rock is measured vertically from ground line and is highly variable. Each cut section requires a detailed examination of the rock core samples.

ROADSIDE DITCH BENCHES

When cut slopes are steeper than 1 1/2:1 and the 30 foot safety clear zone from the edge of pavement to the cut slope is not required, a roadside ditch bench is recommended. Typically, the width of the roadside ditch bench from the shoulder to the cut slope will be 12 or 14 feet. Roadside ditch benches are shown on Figures 3 and 4.

OVERBURDEN SLOPES

Cut slopes in overburden and weathered rock vary from 2:1 or flatter to 1:1. Typical recommendations are usually 2:1, but in mountainous terrain, where the depths are shallow (5 to 15 feet), it is quite often necessary to steepen slopes to 1 1/2:1 or 1:1. In some areas where the depth of overburden and rock disintegration zone exceeds 20 feet, a slope stability analysis is performed.

CUT SLOPES IN HIGHLY TILTED STRATA

Cut slope recommendations in highly tilted strata are more complicated. Recommendations are influenced by the apparent dip of strata along the centerline and cross sections in addition to the normal considerations given for lithology changes and discontinuities. A complete field reconnaissance of each cut section is required prior to slope design.

Normal design criteria for slopes may be utilized when the apparent dip along centerline is less than two degrees and apparent dip on the cross section is away from the roadway. Intermediate bench elevations should follow apparent dip and will have a slight grade. These benches are flat and will cross cut strata in one direction.

In cuts where the apparent dip along centerline is more than two degrees and the apparent dip on the cross section is away from the roadway, intermediate benches with widths from 18 to 25 feet may be utilized and should be designed as horizontal, cross cutting strata in two directions. Cut slopes are recommended for each lift according to the weakest characteristics encountered in the structural domain.

In cuts where the apparent dip on the cross sections is towards the roadway, intermediate benches are omitted and one presplit slope should be recommended from the top of rock to grade and it may be necessary to utilize a 25 foot roadside ditch bench. In special areas where a large mass of material could create a major landslide, the design slope should follow the full dip of the strata.

Figures 6 and 7 illustrate typical cut slope configurations utilized in Kentucky.

II. CASE HISTORIES OF REMEDIAL CORRECTIVE MEASURES ...

In some situations where massive rock sections are endangering the roadway, remedial corrective measures consist mainly of removal of material by utilizing benches. The different types of failures and associated corrective measures are illustrated by case histories of Kentucky 160 in Harlan County, U.S. 119 in Bell County, I-75 in Rockcastle County, and U.S. 23 in Lawrence County.

KENTUCKY 160 IN HARLAN COUNTY

On February 17, 1979 a massive rock cut failure occurred on the Cumberland - Benham Road (Kentucky 160) in Harlan County. The existing rock cut was approximately 80 feet high and 1,100 feet long in Pennsylvanian strata which consisted of a massive-bedded sandstone underlain by siltstone and shale. The sandstone in the top of the cut was fractured and contained intersecting vertical joint planes. The joints were continuous through the underlying shale and siltstone beds. After weathering the siltstone and shale were not capable of providing support, and wedge failures began to occur. Corrective methods were complicated by the presence of a collapsed coal mine above the cut. The coal mine, at an approximate elevation of 1628, collapsed and caused a break to develop in a zig-zag pattern ± 300 feet right and parallel to Kentucky 160 and Looney Mountain. This break varies from a top ground elevation of 1775 down to elevation 1658 between station 4+50 and station 11+50. The maximum measured width of the break is 15 feet, and the total amount of collapsed strata is assumed to be 125 feet. Surface water running off the upper slope is being intercepted and may be accumulating in the mine. The regional dip of the strata is less than 3 degrees towards Kentucky 160, and as a result water seeps occur along the rock slopes above the roadway. Local residents have reported that smoke has been observed in several different places along the break. It is imperative that corrected slopes not encounter the mine or disturb overburden that could slide and expose the openings.

Alternates considered were:

1. Relocate Kentucky 160 through the community of Clutts and avoid cutting into the mountain. This alternate was not considered practical due to right-of-way problems, new bridge requirements and an at-grade railroad crossing.

2. Move Kentucky 160 away from the cut slope by making a channel change and shifting the alignment. This alternate was not considered practical due to problems associated with a channel change and relocation for several residences.

3. Reconstruction of the complete cut section would provide a wider roadway through the cut area and significantly reduce the probability of rockfall. However, the main disadvantage was that the slopes might encounter the collapsed mine zone and significant landslides could result in additional costs.

4. Reconstruct the top part of the cut by removing the sandstone overhang and not redesign the siltstone and shale slopes. The potential for additional landslides was less than alternate #3; however, this alternate only removed the existing hazard in the sandstone and did not permit upgrading of Kentucky 160. Refer to Figure 8.

5. The do nothing alternate provided no relief to the affected citizenry who had expressed much concern over the existing hazard.

The Bureau of Highways agreed to alternate #4 and the contract was let to Mountain Construction for a bid price of $10.00/cubic yard of rock excavation. During construction it became necessary to close traffic on Kentucky 160 and the rock excavation price was lowered to $9.50/cubic yard. Plan quantities of rock excavation was 34,831 cubic yards, and the completed project cost is approximately $413,000.00.

U.S. 119 IN BELL COUNTY

Rock cut slopes on the Pineville - Harlan Road (U.S. 119) are situated on the southeast flank of the Pine Mountain Thrust Fault. Near the locality of Calloway, the Pennsylvanian Age strata dips approximately 55 degrees towards the roadway. The Hance Formation in this cut consists of interbedded sandstone and shale with the sandstone beds varying in thickness from two to four feet, and the interbedded intermediate and soil-like shale beds varying in thickness from one to three feet. The original cut was approximately 600 feet long and 140 feet high, and material was removed along the bedding planes of the sandstone with some minor cross cutting. The surface water draining from the southeast Pine Mountain slope gradually washed the shale out from between the sandstone beds. After a period of five to eight years the sandstone began to collapse at the toe completely blocking the roadside ditch. As a result, surface water began to undercut the roadway, and large sandstone boulders were falling into the highway as the beds collapsed.

Several alternate methods of correction were reviewed in the field, but basically consisted of removal of material along the exposed sliding plane utilizing a 25 foot (variable) width intermediate bench at elevation 1325. The broken material below elevation 1325 was left as support with a 1:1 (variable) slope to ditch grade. The roadside ditch bench width was also variable (10'-20') and was transitioned in at station 596+00 and out at station 602+00.

The exact condition of the bedding plane slope was unknown; however, it was assumed to be relatively free of fractures, and removal of material was accomplished without heavy blasting. The project was let in 1980 to London Bridge Company for a bid price of $7.85/cubic yard. Planned quantities of 29,939 cubic yards were to be removed for a total cost of $258,240.00. Due to slope revisions made during construction the quantities underran and the contractor is in the process of renegotiating the rock excavation price.

I-75 ROCKCASTLE COUNTY

In Rockcastle County, .6 mile northeast of Mt. Vernon, a north-south cut section on I-75 caused the Bureau's Maintenance Division considerable expense. Original construction was 1969 and daily inspection patrols were required through the cut to remove fallen rocks and boulders from the roadway.

The cut section was approximately 1,600 feet long, 210 feet deep, and encountered strata of Mississippian and Pennsylvanian age. The main problem was caused by an abundance of solution channels in the upper Newman Limestone. As the clay shale and/or soil eroded 15'-20' pinnacles were left in a hazardous position and topping failures occurred. The Pennsylvanian rock (above elevation 1230) is the Lee Formation and consists of interbedded shales and sandstones with some thin coal beds and underclay. The beds are highly distorted due to an east-west trending slump block. Differential weathering contributed to some failures in the sandstone sections.

The optimum solution for this cut was a complete redesign utilizing new 18 foot intermediate benches in the limestone formation and a 25 foot intermediate bench below the Pennsylvanian strata with the shale and sandstone on 1/2:1 slopes. Figure 9 is a pictorial representation of the east side (NBL) for this type correction. In general, this correction was acceptable; however, the quantity of rock excavation was excessive and construction time would have been longer.

In an effort to redesign a safe, economical cut, special attention was given to the ends of the sections where most of the problems occurred. The basic procedure was to eliminate the channeled zones with a series of step ups with wider benches. Bench widths and step up stations were plotted by field surveys and aerial photographs. Bench width was governed by the existing fallout zones of the solution channels along the slopes of the hill. Step up points were determined by selecting stations that eliminated the channeled portion and left solid limestone in place. Figures 10 (west side of SBL) and 11 (east side of NBL) are pictorial representations for these recommendations which could be constructed in conjunction with the existing cut slope configurations.

The project was let out for construction on January 23, 1975 and awarded to R.C. Durr for a bid price of \$2.96/cubic yard of excavation. The actual construction started April 2, 1975 and was completed November 26, 1975. Traffic was maintained by closing two lanes and working on one side at a time. The contractor utilized a dirt pad to avoid excessive damage to the roadway. Several changes were required during construction, but the most significant change was with the problems encountered in attempting to construct 90 degree corners in the limestone sections. Excessive breakage necessitated using a 45 degree angle and extending some of the benches. The top slope on the west side (SBL) was changed to a 70 foot lift on a 1:20 presplit slope with a 60 foot intermediate bench below the Pennsylvanian strata.

Total excavation quantities for this project was 229,272 cubic yards, and the total project cost was $787,893.00.

U.S. 23 LAWRENCE COUNTY

An unusual rock cut slope failure occurred during the construction phase of the Louisa - Catlettsburg Road (U.S. 23). The cut slope limits were from station 211+50 to station 255+00, and the contractor had completed presplitting operations. The 1/2:1 cut slopes appeared stable, but when the final few feet of material were removed from the ditch line a crack appeared in the presplit slope left of station 251+00, extended behind the slope in a northeasterly direction, and disappeared 200 feet left of station 253+50. Three days later the crack had opened to 2 feet, and the benches above settled. The extreme east end of the mountain moved on a thin coal and underclay bed 2 feet above the roadway ditch elevation. The dip of the strata was less than one degree towards the roadway and the discontinuity was near vertical. This was not detected during the design phase even though a total of 22 rock cores were taken in this cut.

Remedial corrections for this type failure are limited; however, total removal of material was required. One presplit slope (1/2:1) $\pm$ 90 feet high behind the discontinuity trimmed off the end of the mountain and the failure. A variable width fall bench was automatically transitioned in from 18 feet at station 251+00 to approximately 200 feet at station 255+00. Bizzack Brothers Construction Company removed 83,391 cubic yards of rock at $2.77/ cubic yard for a total cost of $230,993.07.

ACKNOWLEDGMENTS

The writer would like to express appreciation to Henry Mathis and Gordon Scott for their assistance in preparing this paper, Kim Gunn for her secreterial work, and William Preston for his graphics. Many highway engineers, geologists, and consulting personnel contributed to development of the existing cut slope design criteria in Kentucky.

TYPICAL SLOPE CONFIGURATIONS
SOIL LIKE SHALE

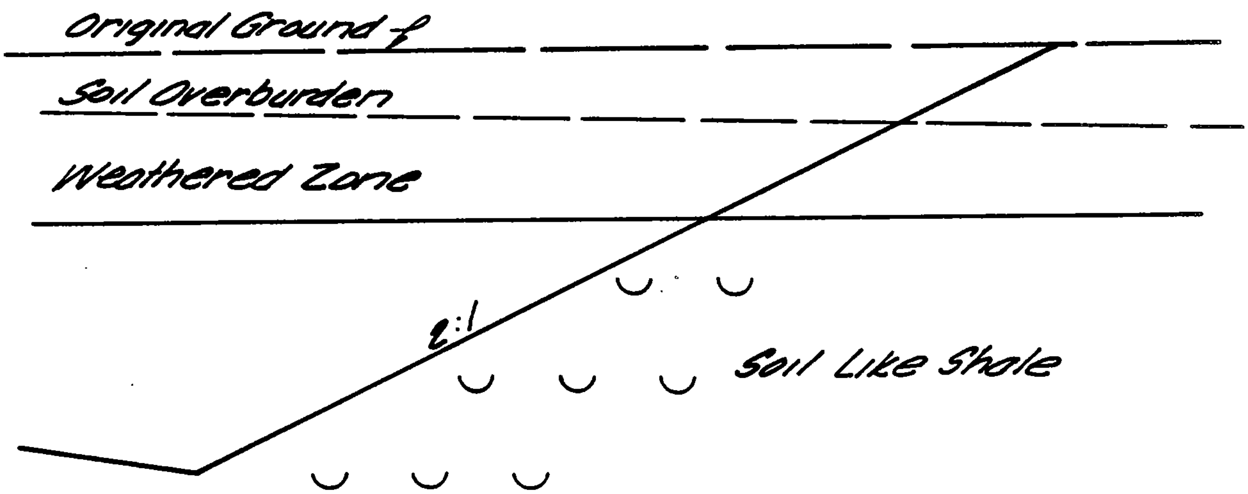


 Fig. 1

TYPICAL SLOPE CONFIGURATIONS
INTERMEDIATE SHALE

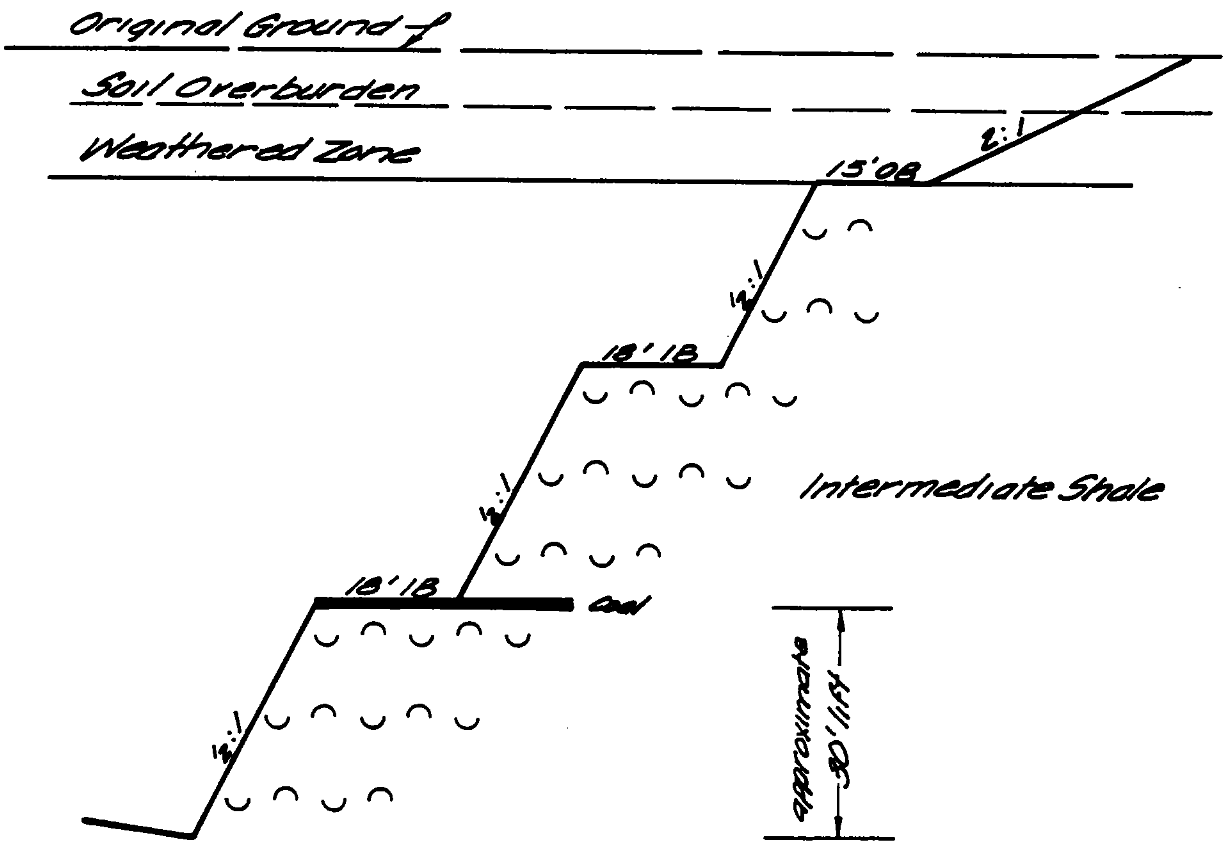


 Fig. 2

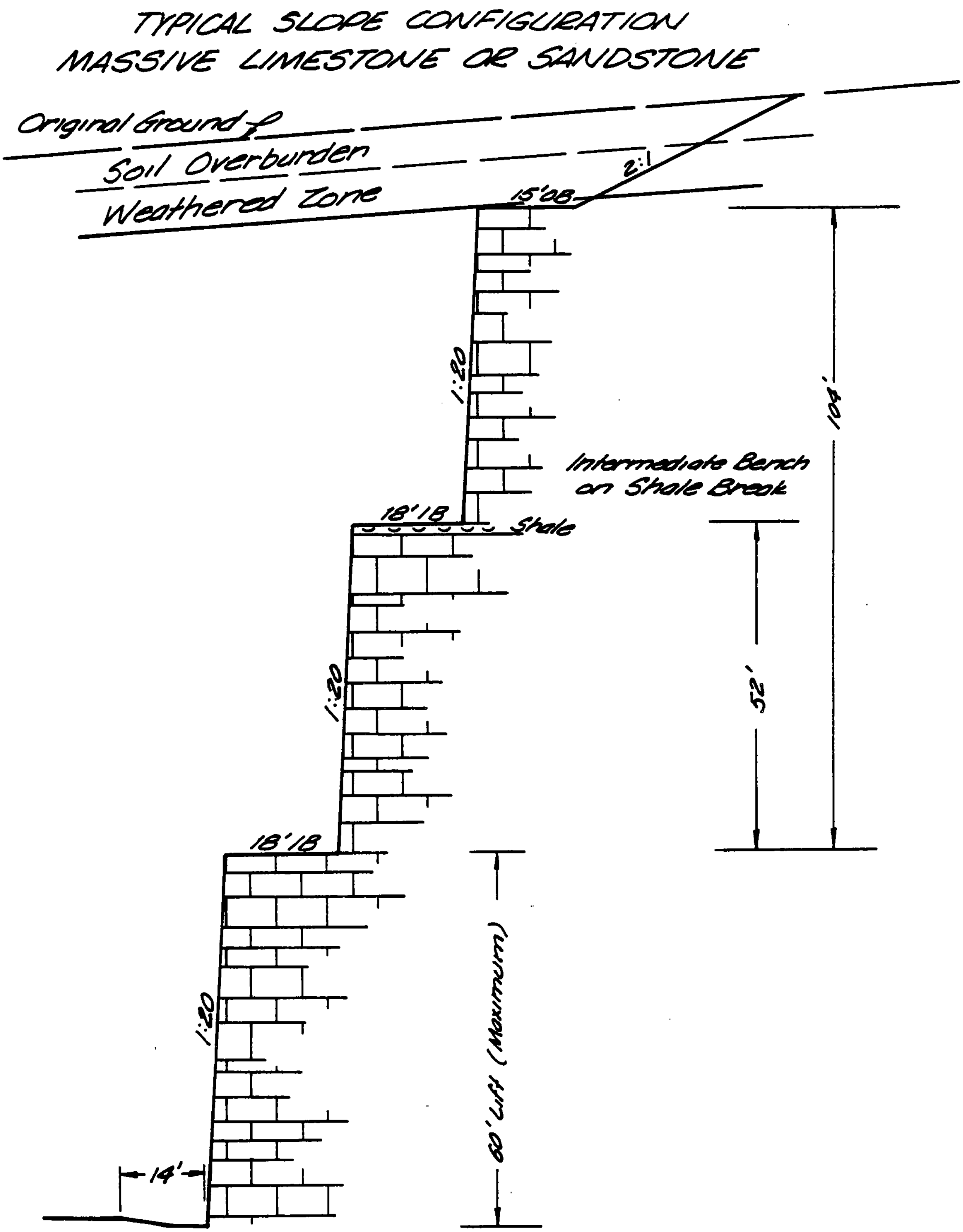

TYPICAL SLOPE CONFIGURATION
MASSIVE LIMESTONE OR SANDSTONE
Original Ground
Soil Overburden
Weathered Zone
15'08
2:1
1:20
104'
Intermediate Bench
on Shale Break
18'18
Shale
52'
18'18
60' Lift (Maximum)
14'

 Fig. 3

TYPICAL SLOPE CONFIGURATION

SHALY LIMESTONE OR SANDSTONE

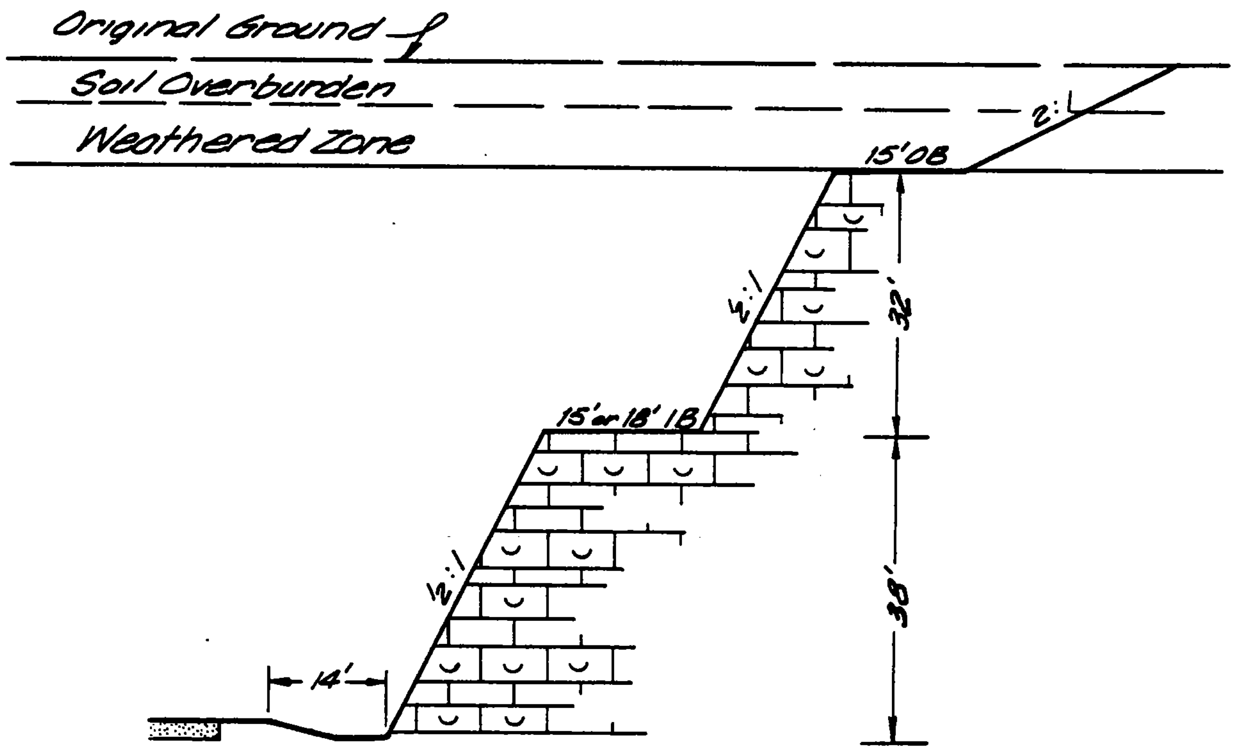


 Fig. 4

TYPICAL SLOPE CONFIGURATION
SERRATED SLOPES

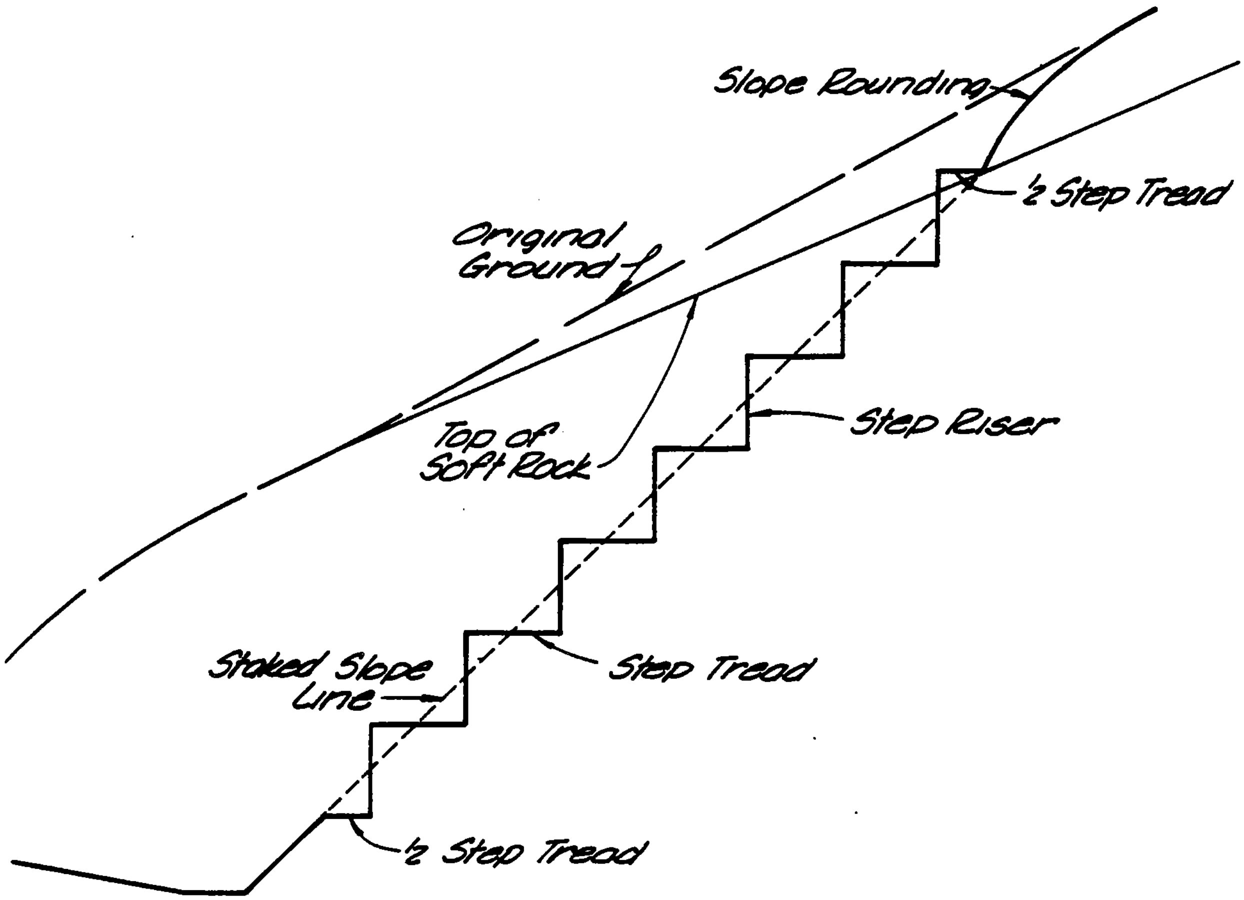


 Fig. 5

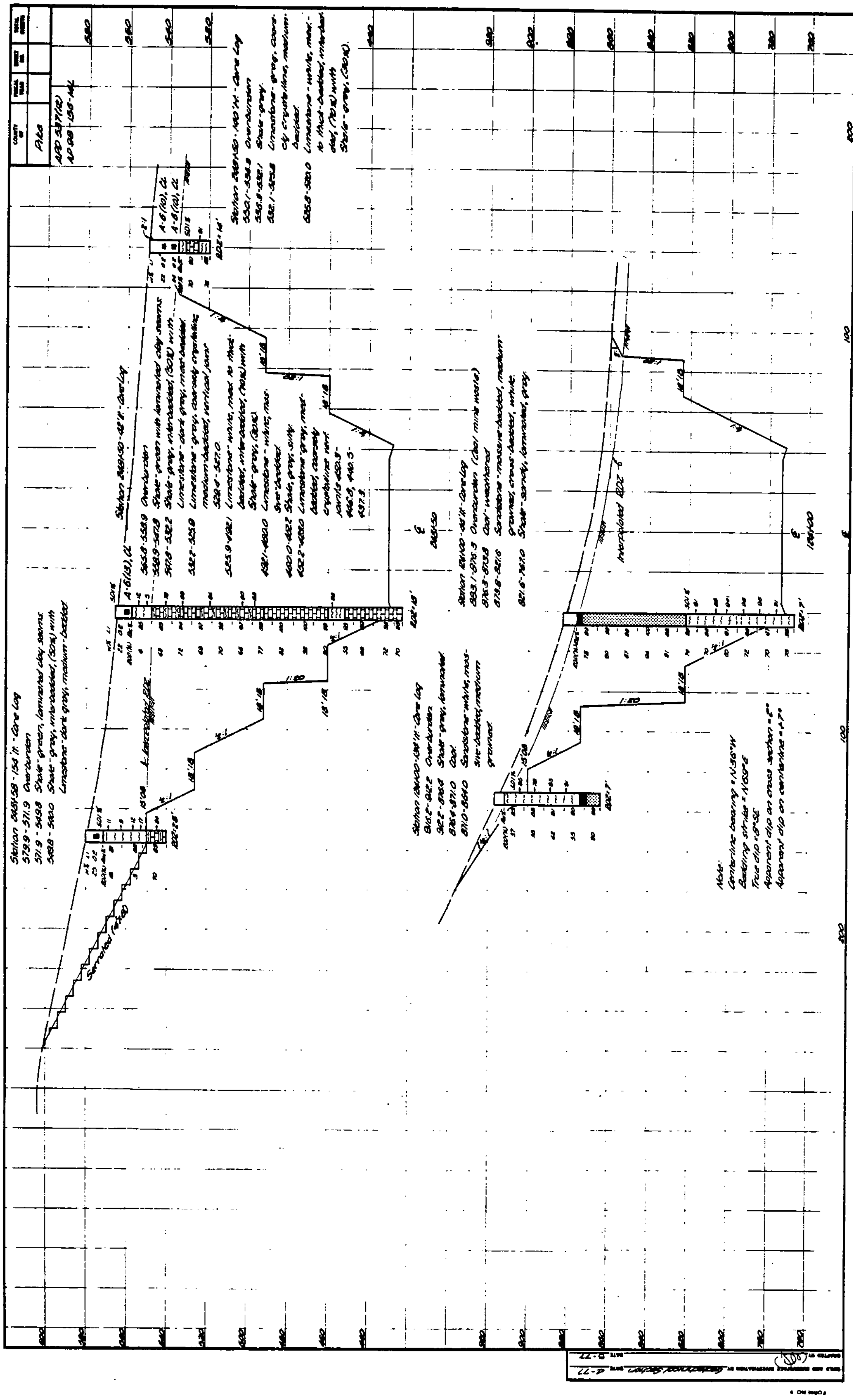


 Fig. 6

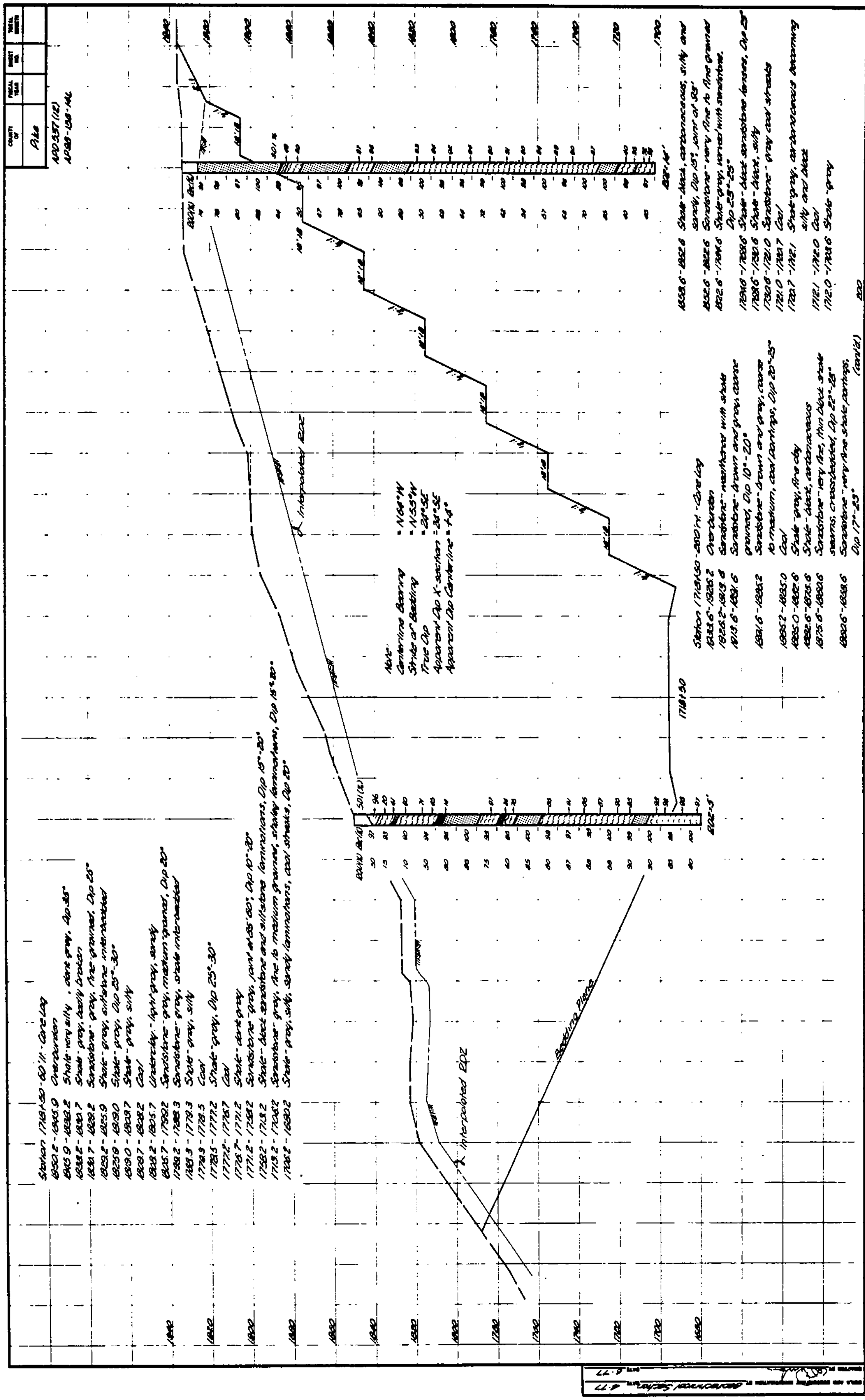

Pike
APD 537 (12)
AP 98-138-14L
Station 1718+50 - 60' lt. - Core Log
1850.2 - 1845.9 Overburden
1845.9 - 1838.2 Shale - very silty, dark gray, Dip 35°
1838.2 - 1830.7 Shale - gray, badly broken
1830.7 - 1829.2 Sandstone - gray, fine-grained, Dip 25°
1829.2 - 1825.9 Shale - gray, siltstone interbedded
1825.9 - 1819.0 Shale - gray, Dip 25°-30°
1819.0 - 1809.7 Shale - gray, silty
1809.7 - 1808.2 Coal
1808.2 - 1805.7 Underclay - light gray, sandy
1805.7 - 1799.2 Sandstone - gray, medium grained, Dip 20°
1799.2 - 1788.3 Sandstone - gray, shale interbedded
1788.3 - 1779.3 Shale - gray, silty
1779.3 - 1778.5 Coal
1778.5 - 1777.2 Shale - gray, Dip 25°-30°
1777.2 - 1776.7 Coal
1776.7 - 1771.2 Shale - dark gray
1771.2 - 1758.2 Sandstone - gray, joint at 85° 60°, Dip 10°-20°
1758.2 - 1713.2 Shale - black sandstone and siltstone laminations, Dip 15°-20°
1713.2 - 1706.2 Sandstone - gray, fine to medium grained, shaley laminations, Dip 15°-20°
1706.2 - 1690.2 Shale - gray, silty, sandy laminations, coal streaks, Dip 20°
Interpolated RDZ
Bedding Plane
Note:
Centerline Bearing = N 64° W
Strike of Bedding = N 53° W
True Dip = 24° SE
Apparent Dip X-section = 34° SE
Apparent Dip Centerline = 4.4°
Station 1718+50 - 260' rt. - Core Log
1933.6 - 1926.2 Overburden
1926.2 - 1919.8 Sandstone - weathered with shale
1919.8 - 1891.6 Sandstone - brown and gray, coarse grained, Dip 10° - 20°
1891.6 - 1885.2 Sandstone - brown and gray, coarse to medium, coal partings, Dip 20°-25°
1885.2 - 1885.0 Coal
1885.0 - 1882.6 Shale - gray, fire clay
1882.6 - 1875.6 Shale - black, carbonaceous
1875.6 - 1860.6 Sandstone - very fine, thin black shale seams, crossbedded, Dip 22°-28°
1860.6 - 1853.6 Sandstone - very fine shale partings, Dip 17°-23° (cont'd.)
1853.6 - 1852.6 Shale - black, carbonaceous, silty and sandy, Dip 15°, joint at 85°
1852.6 - 1822.6 Sandstone - very fine to fine grained
1822.6 - 1784.6 Shale - gray, varved with sandstone, Dip 23°-25°
1784.6 - 1768.6 Shale - black sandstone lenses, Dip 25°
1768.6 - 1730.6 Shale - black, silty
1730.6 - 1721.0 Sandstone - gray coal streaks
1721.0 - 1720.7 Coal
1720.7 - 1712.1 Shale - gray, carbonaceous becoming silty and black
1712.1 - 1712.0 Coal
1712.0 - 1703.6 Shale - gray
1718+50
300
200
100
℄
100
200
Cut Stability Section Station 1718+50

 Fig. 7

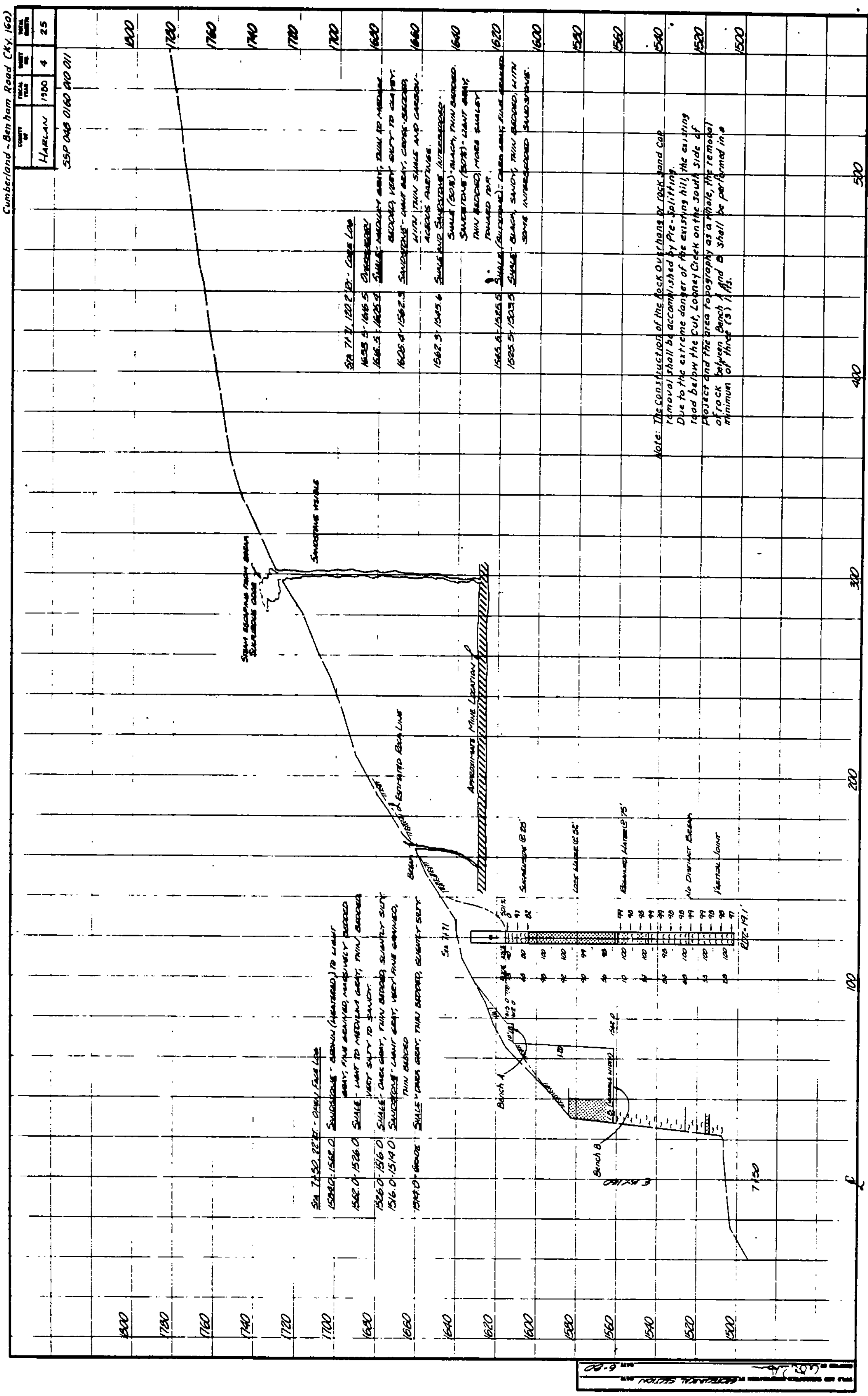

Cumberland - Benham Road (Ky. 160)
Harlan
1980
SSP 048 0160 010 011
Note: The construction of the Rock Overhang or rock band cap removal shall be accomplished by Pre-Splitting.
Due to the extreme danger of the existing hill, the existing road below the cut, Looney Creek on the south side of project and the area topography as a whole, the removal of rock between Bench A and B shall be performed in a minimum of three (3) lifts.
Sandstone Visible
Approximate Mine Location
Estimated Rock Line
Bench A
Bench B
Cut Stability Section
Station 7+50

Fig. 8

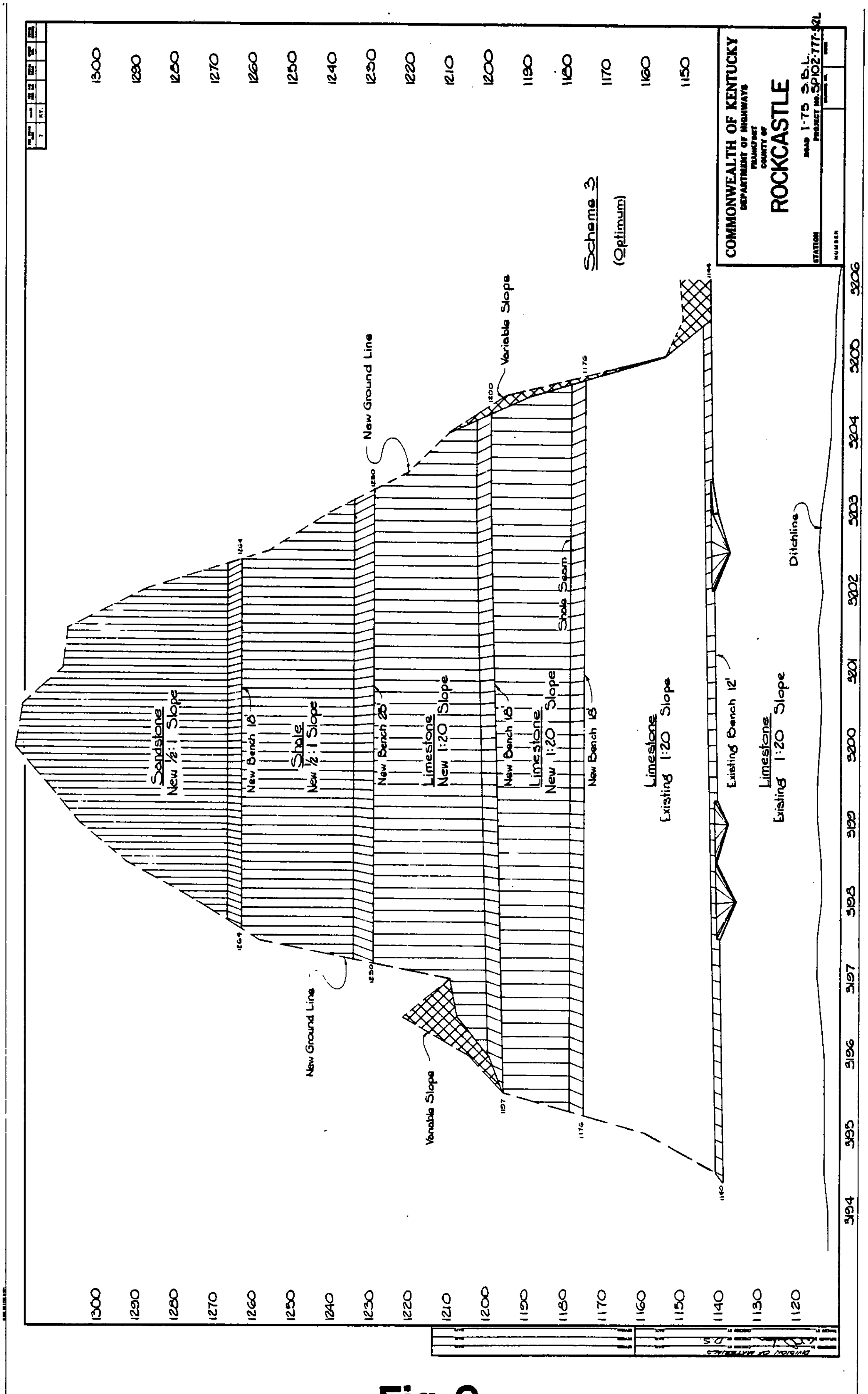
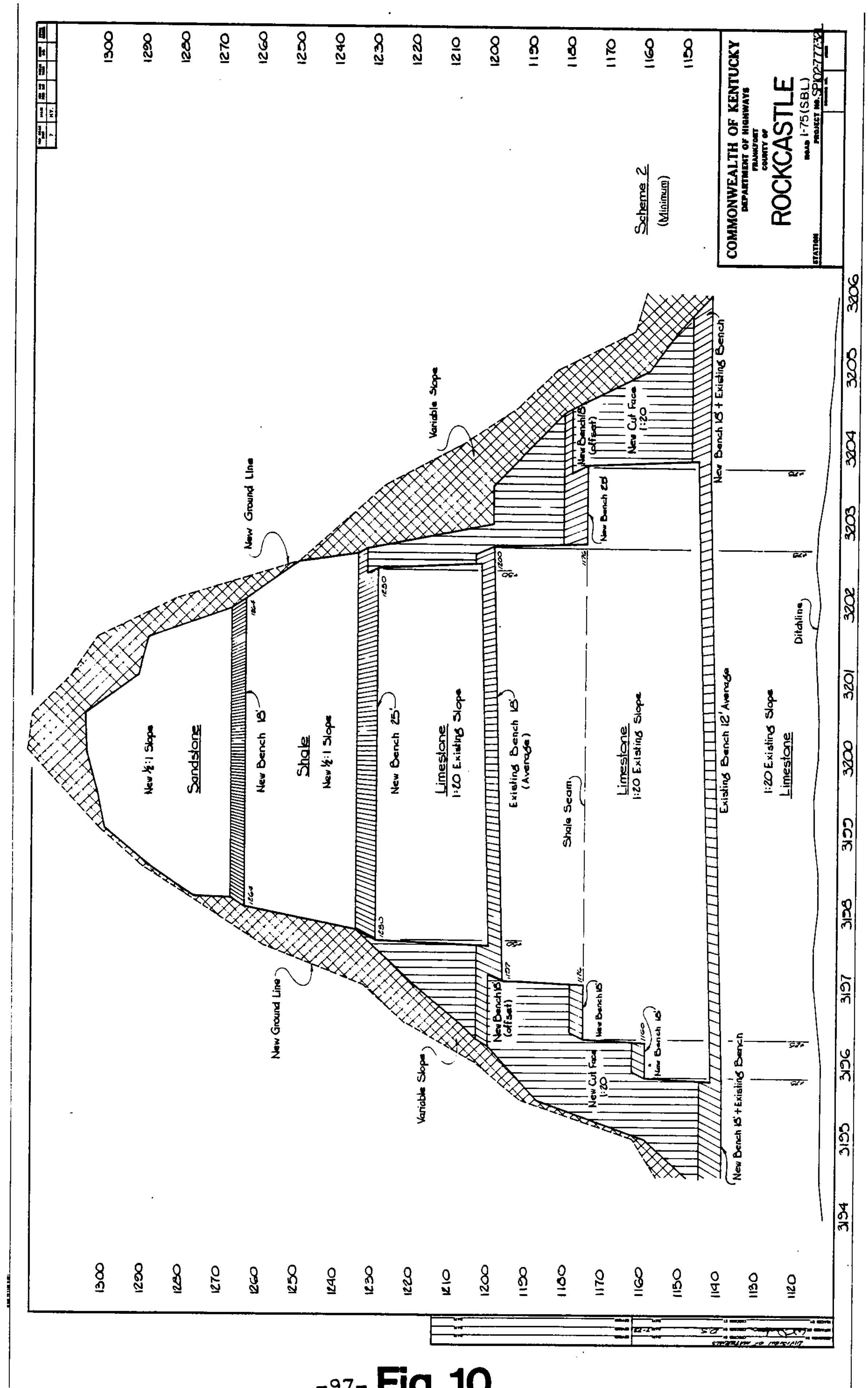
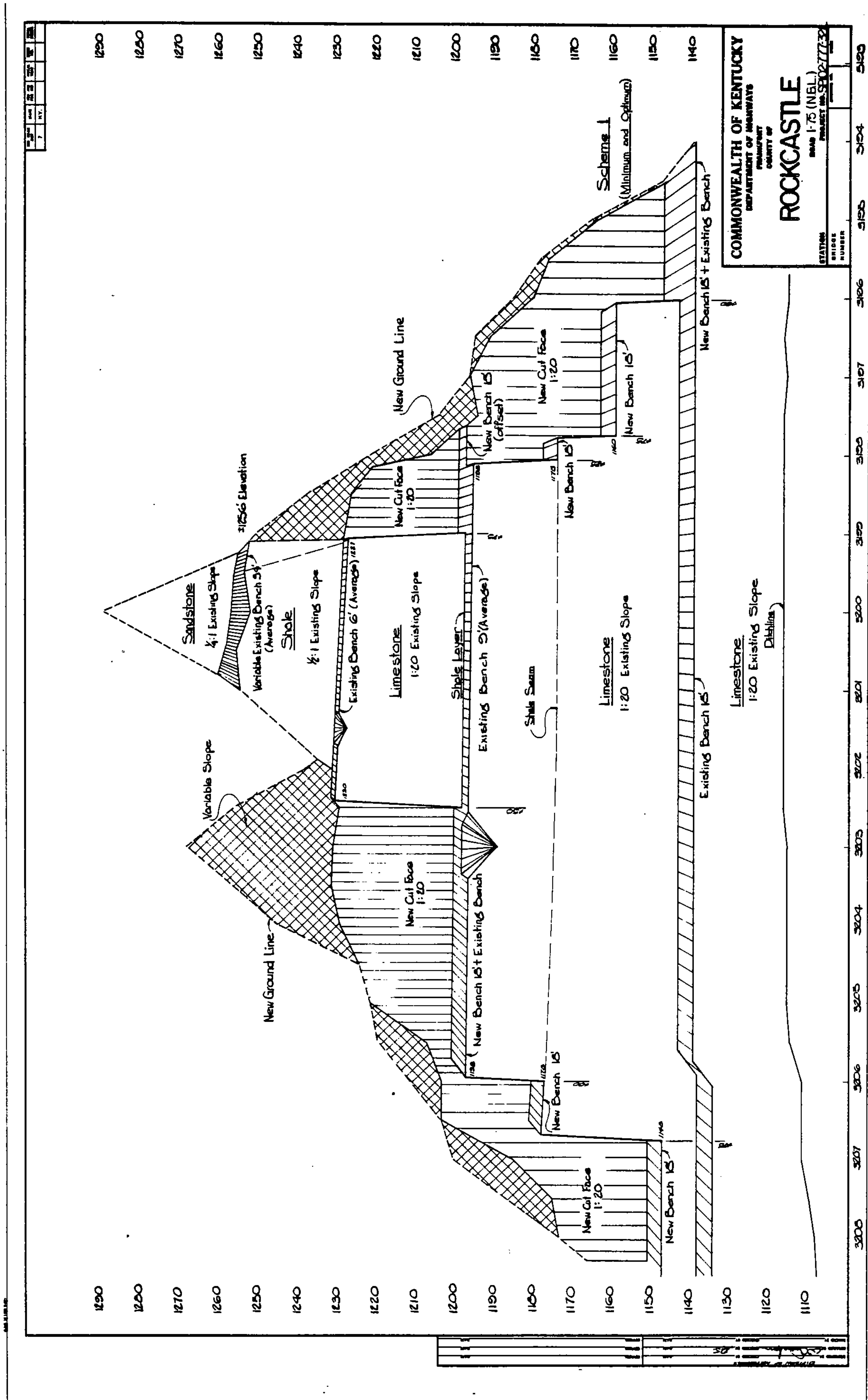

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT
COUNTY OF
ROCKCASTLE
ROAD I-75 S.B.L.
PROJECT NO. SP102-777-32L
Scheme 3
(Optimum)
Sandstone
New ½:1 Slope
New Bench 18'
Shale
New ½:1 Slope
New Bench 25'
Limestone
New 1:20 Slope
New Bench 18'
Limestone
New 1:20 Slope
New Bench 18'
Limestone
Existing 1:20 Slope
Existing Bench 12'
Limestone
Existing 1:20 Slope
Shale Seam
New Ground Line
New Ground Line
Variable Slope
Variable Slope
Ditchline
1300
1290
1280
1270
1260
1250
1240
1230
1220
1210
1200
1190
1180
1170
1160
1150
1140
1130
1120
3194
3195
3196
3197
3198
3199
3200
3201
3202
3203
3204
3205
3206
DIVISION OF MATERIALS

 Fig. 9


COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT
COUNTY OF
ROCKCASTLE
ROAD I-75 (SBL)
Scheme 2
(Minimum)
New ½:1 Slope
Sandstone
New Bench 15'
Shale
New ½:1 Slope
New Bench 25'
Limestone
1:20 Existing Slope
Existing Bench 15'
(Average)
Shale Seam
Limestone
1:20 Existing Slope
Existing Bench 12' Average
1:20 Existing Slope
Limestone
Ditchline
New Ground Line
Variable Slope
New Bench 15' (offset)
New Cut Face
1:20
New Bench 25'
New Bench 15'
New Bench 15' + Existing Bench
1300
1290
1280
1270
1260
1250
1240
1230
1220
1210
1200
1190
1180
1170
1160
1150
3194
3195
3196
3197
3198
3199
3200
3201
3202
3203
3204
3205
3206

-97- **Fig. 10**


COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT
COUNTY OF
ROCKCASTLE
ROAD I-75 (NBL)
Scheme 1
(Minimum and Optimum)
New Ground Line
New Cut Face 1:20
New Bench 15' (offset)
New Bench 15'
New Bench 15' + Existing Bench
1256' Elevation
Sandstone
¼:1 Existing Slope
Variable Existing Bench 3' (Average)
Shale
½:1 Existing Slope
Existing Bench 6' (Average)
Limestone
1:20 Existing Slope
Shale Layer
Existing Bench 5' (Average)
Shale Seam
Existing Bench 15'
Ditchline
Variable Slope
1290
1280
1270
1260
1250
1240
1230
1220
1210
1200
1190
1180
1170
1160
1150
1140
1130
1120
1110
3193
3194
3195
3196
3197
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3199
3200
3201
3202
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3208

 Fig. 11

A System for Rapid Collection and Evaluation of Geologic-Structure Data for Rock Slope Stability Analysis

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ABSTRACT

As construction of the interstate highway system nears completion, emphasis is now shifting to maintenance. A concern in the Valley and Ridge Province is stability analysis of highway rock-cuts to ascertain maintenance needs and safety.

Several methods of rock-slope stability analysis are employed, with limit analysis procedures involving stereographic representation of data the most common. Regardless of the specifics, all depend upon detailed surveys of orientations and characteristics of weakness planes within the rock mass. Field surveys are extremely time-consuming and typically involve a two-member team. Subsequent hand-plotting of data points is also slow and tedious.

A system for rapid data-collection and evaluation is being developed at Purdue University. It allows for rapid, direct collection of field data and provides a preliminary computer analysis of the slope while on-site. It consists of:

1. A lightweight data-collection package carried in the field which stores on magnetic tape, orientations of rock discontinuities;
2. A fairly inexpensive, portable microcomputer maintained in a field vehicle or motel room and,
3. A specially designed, computer and software interface which provides data exchange from collection package to microcomputer.

Advantages of the system include: 1) analysis of more data and hence more slopes in the same time as compared to conventional means 2) on-site preliminary computer analysis to pinpoint data discrepancies thus showing a need for further work before leaving site, 3) a microcomputer providing the link to a larger computing system if greater detail required, 4) data transmission by telephone from the field if desired, yielding major time saving, 5) input of data without costly keypunching and 6) potentially, an efficient, one-man field party.

INTRODUCTION

The engineering geology group at Purdue University is developing a system which will aid in the collection and analysis of rock-slope stability data. Its purpose is not to replace current techniques but to improve the efficiency of those existing procedures. The system will speed up the data collection process, provide preliminary analyses of data in the field, and facilitate the transfer of collected data to large computer systems for detailed analyses. A brief introduction to the analysis of rock-slope stability is provided below, followed by a description of the data collection system.

ROCK-SLOPE STABILITY ANALYSIS

Many methods of rock-slope stability analysis exist, with limit analysis procedures involving stereographic representation of data, being the most common. For more information concerning stereonets, the reader is referred to Hoek and Bray (1977), chapter 3. Regardless of the specifics, all methods depend upon detailed surveys of orientations and characteristics of discontinuities within the rock mass.

A discontinuity is defined as a structural weakness plane or surface along which movement can take place. Types of rock discontinuities are joints, faults, shear zones, bedding surfaces, and foliations. The significant characteristics of discontinuities are: 1) geometry, 2) continuity, 3) spacing, 4) surface irregularities, 5) physical properties of adjacent rock, 6) infilling material, and 7) ground water (West, 1979). A detailed table of various characteristics and their features was developed for the field manual (or Part G) of the FHWA publication, Rock Slope Engineering (see reference list) and is reproduced here as Figure 1. These data are generally recorded in the field for subsequent detailed analyses using sophisticated computer systems and techniques.

Detailed computer analyses are expensive and time consuming; and commonly, they are not performed unless a simplified preliminary analysis first suggests potential slope failure. Such simple analysis would involve plotting a representative sample of discontinuities on an equal-area stereonet and examining the plot regarding kinematically possible failure surfaces. Most techniques assume that the discontinuities are through-going within the critical portion of the slope and that failure would occur entirely along a single plane. They also assume that the cohesion contribution along the plane is negligible or essentially zero. In that case, the factor of safety reduces to a function of the angle of dip of the discontinuity, θ, and the angle of sliding resistance along the discontinuity, $\emptyset$.

Hence $$FS = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{\text{Tan } \emptyset}{\text{Tan } \theta}$$

SITE: ________

COORDS. OR CHAINAGE

NORTH ☐☐☐☐☐☐☐ ELEV ☐☐☐☐☐ TREND ☐☐☐ OBS. ☐☐ NAME: ________

TRAVERSE STATION ☐☐☐☐

EAST ☐☐☐☐☐☐☐ LOCAT. ☐☐☐☐☐ PLUNGE ☐☐☐ LENGTH ☐☐☐☐ DATE: ________

STATION (FROM)	TRAV. TREND	DISTANCE (FEET)	ROCK		STR. TYPE	NO. of JNS	JN SPCG (FT)	STRIKE	DIP			SIZE		INFILLINGS			W T R	R G S	WAVINESS		REMARKS
			TYPE	HDNS					±	ANG	DIR.	LENGTH (FT)	ENDS	TYPE	TH	HDNS			ILA	WAVE LTH.	
1	5	8	12	15	17	19	21	23	26	27	29	31	34	35	38	39	41	42	43	46	

Figure 1. Field Sheet for Traverse Line Data (after Piteau, 1980).

Other procedures utilize a friction cone concept which may be modeled to include a cohesion contribution.

Figure 2 illustrates the primary types of rock-slope failures, along with the structural conditions which lead to such failures. It is the preliminary analysis of these failure modes which the rapid data collection and analysis system can provide in the field. If the preliminary analysis suggests potential failure, then the data collection system is used again to obtain additional data, and subsequently to transfer all relevant data directly to a larger computer system for detailed analysis.

THE RAPID DATA COLLECTION AND EVALUATION SYSTEM

Equipment Description

The system consists of three basic components (See Fig. 3).

I. A lightweight, data-collection package carried in the field, which stores orientations of rock discontinuities on magnetic tape;

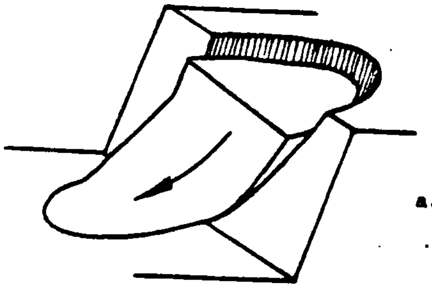
II. A specially-designed, computer hardware and software interface which provides for data exchange from the collection package to the microcomputer; and

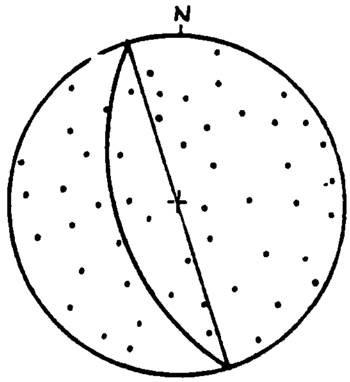
III. A fairly inexpensive, portable microcomputer maintained in a field vehicle or motel room.

A photograph of the system is shown as Figure 4.

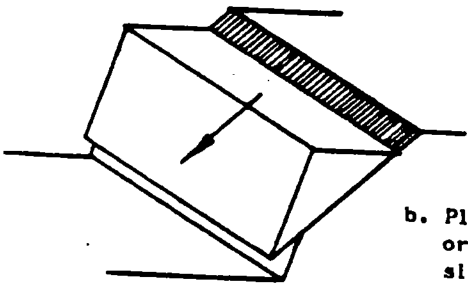
Component I: Data-Collection Field Package

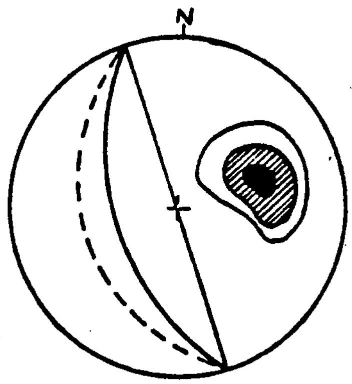
The early designs for the field package called for the construction of a calculator-type keyboard. This was to be used in the field for direct input of data to a lightweight cassette tape recorder. Also within the system would be a modified engineering compass which would digitize discontinuity dip value and direction readings for direct tape input, thus eliminating much of the keyboard entry. At approximately this same time the Sharp Electronics Company marketed a pocket computer in Japan, the Sharp PC-1211, which, as it develops, can act as the system's entry keyboard and temporary memory for structural data. Radio Shack introduced a similar pocket computer in the United States and so it was decided to use it in the prototype system. The Radio Shack Pocket Computer has since been successfully adapted for the system by the authors. Modification of the engineering compass to supply data automatically onto tape is still in a developmental stage.



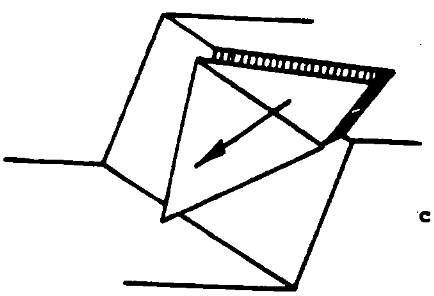


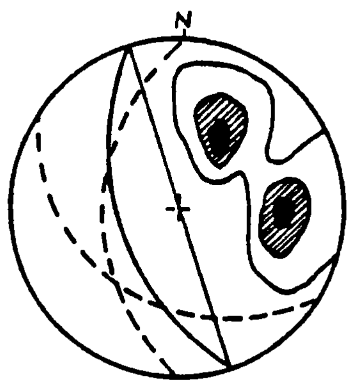
a. Circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern.



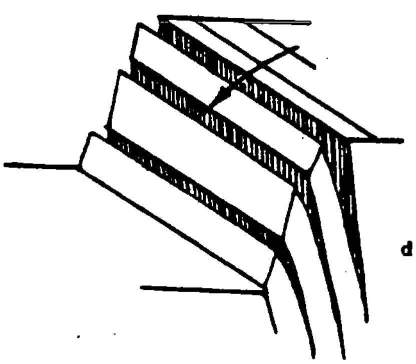


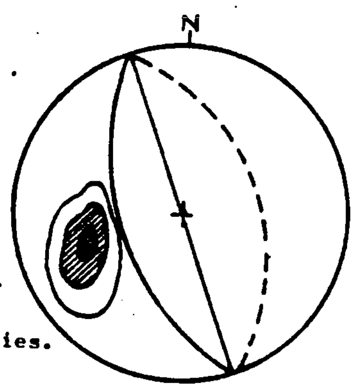
b. Plane failure in highly ordered structure such as slate.





c. Wedge failure on two intersecting discontinuities.





d. Toppling failure in hard rock which can form columnar structures separated by steeply dipping discontinuities.

Figure 2 : Main types of slope failure and appearance of stereoplots of structural conditions likely to give rise to these failures. (after Hoek and Bray, 1977)

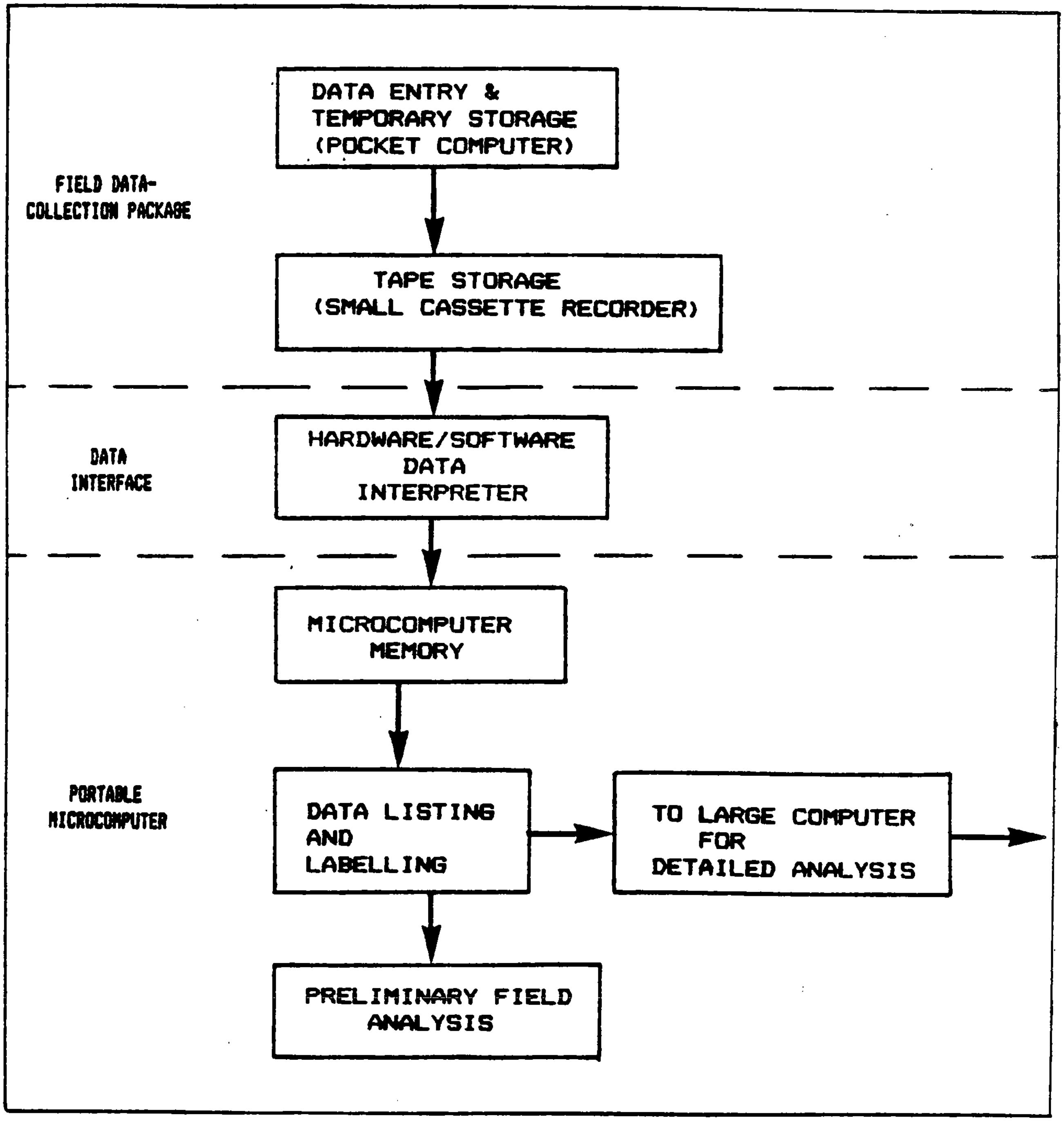


Figure 3. Flow Chart of the Data Collection and Evaluation System.

The pocket computer is a portable, calculator-size computer that is programmable in the BASIC computer language. Radio Shack sells a cassette interface which allows programs or data for the pocket computer to be stored and retrieved from cassette tape. A software package developed at Purdue University for the pocket computer can be used to prompt an investigator while collecting the structural data. This package was developed to conform with the data collection procedure outlined in the FHWA short course on rock slope engineering by D.R. Piteau (see Figure 1). It will temporarily store data in the pocket computer, and later transfer it onto the tape of the lightweight cassette recorder, also carried in the field. The combined pocket computer, cassette interface, and cassette recorder make a portable, easy to operate data collection package. The efficiency of this package will be increased even more when the automatic transfer of engineering compass data to tape is accomplished.

Component II: Pocket Computer to Microcomputer Interface

The Pocket Computer, as sold, is not communication-compatible with larger computer systems. Therefore, some means of translating the stored data and transferring it to a larger computer system had to be found. This resulted in the development of an electronic interface circuit which shapes the tones recorded by the pocket computer into RS-232* compatible signals and properly controls the playback speed of the recorder. This electronic circuit thus makes the pocket computer tapes compatible with almost any larger type of computer system. Unfortunately, the pocket computer binary coding for letters and numerals is not ASCII.+ Therefore, computer software had to be written for the microcomputer which would translate the data from the pocket computer code into a more conventional code, organize it, and finally provide a hard-copy printout of the data. These steps are usually accomplished first in programs that perform data analysis.

The most difficult (and time consuming) part of the design for the total data collection and evaluation system was determining the specific coding used by the pocket computer; designing and building the hardware interface, and writing the software to translate the code.

Component III: Portable Microcomputer

There are many microcomputers on the market which would work suitably within this system. Any computer which is RS-232 compatible is potentially compatible with the specially designed interface of component II. Due to an extremely limited budget, an inexpensive yet reliable computer was sought in this study. The Radio Shack TRS-80, Model I, with 16K memory was selected as it met these criteria. It has served for one year, under nearly constant use, with only two minor repairs required.

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* An industry standard for computer communications.

+ American Standard Code for Information Interchange.

In addition, parts and service are available nationwide for this Radio Shack equipment which is an important feature for a portable system to be transported into the field or to motel rooms. In addition an Epson MX-80 printer has been added to the system to produce hard copy results.

The role of the microcomputer in the system is two-fold. First, it should be portable enough for easy transportation between motel rooms or for placement in a field vehicle. In this way, it can be used daily to provide a hard copy printout of collected data, as well as supplying a number of data analyses. These presently include a rectangular plot of the dip value and direction data and an equal area net plot of either the poles or dip vectors of the discontinuity planes. Planned for the near future are program packages which will perform cumulative sums analyses (Piteau, 1971) plus plane and wedge failure analyses. The obvious advantage of having the microcomputer in the field is the attainment of preliminary evaluations of the slope data on the same day as collection.

The second role of the microcomputer is to act as a link to a large computer facility such as the one at Purdue University. Using a telephone coupler, the data collected in the field can be relayed by telephone to the larger facility for more detailed analysis. Purdue University currently has a number of sophisticated slope stability programs available which account for discontinuous weakness-planes, cohesion, contributions by fracturing intact rock, nonlinear failure planes and various statistical considerations.

For more details on the operation of microcomputers, the reader is referred to the operations manual for the TRS-80 and to the TRS-80 Technical Reference Manual.

Equipment Operation

Two data collection programs, written during this study, are available for the pocket computer. The first is the detailed line survey program which is used to collect all of the data shown on the sample field recording sheet (Fig. 1 this paper and in the FHWA Field Manual). The second program is a simple modification of the first and is used to collect only dip and dip direction data. In order to be compatible with the existing microcomputer programs developed during this study the pocket computer program supplies "dummy" information for all of the variables which are deemed not of interest and hence are not collected.

The following materials comprise the data collection package carried into the field:

1. A Radio Shack Pocket Computer, or Sharp PC-1211 Pocket Computer, and cassette interface;
2. A small cassette tape recorder;
3. A cassette tape containing the data collection programs;
4. Blank cassette tapes on which to record data.

5. A shoulder pack or sample bag in which to carry the tape recorder;

6. The basic geologic mapping equipment described in Rock Slope Engineering, Part G Field Manual, Page 2.

Procedure:

Load the program tape into the pocket computer following the directions outlined in the pocket computer manual. A listing of the two programs is provided in Appendix A of this paper. Switch the pocket computer to the "defined program" mode and press "Shift A". This will initiate the program. In the modified program the first prompt will ask for a line number. That only has to be entered one time for each set of 10 stations. The next prompt will be for the specific joint number for which data are about to be entered. The actual station number will become a combination of the line number and the joint number. For example, joint 3 on line 2 becomes station number 2003.

Proceed through all of the variables in likewise manner. If certain information is not desired the program may be altered to skip it and provide "dummy" information instead. At various intervals, the data are displayed again for review. If these are correct, simply press "ENTER" to continue onward. If they are not, a method for correcting data in the memory is available. This is explained in a subsequent paragraph. The pocket computer may be turned off at any time without losing its memory. When turned back on and "Shift A" is entered, it will indicate the last station recorded and what data are to be supplied next. "Shift A" may, in fact, be pushed any time to remind one of the present station number.

After 10 stations have been collected, the display will indicate that the allotted memory has been used up and the data should be transferred to the cassette tape. The "Shift X" command will turn on the tape recorder and transfer the memory in approximately fifty-nine, audible tone bursts. It may be a good idea to make two copies of the data on separate tapes following the same procedure for each. The next time "Shift A" is entered, the prompt "ClEAR MEM" or "END MEM" will occur as a reminder that the allotted memory is full and must be cleared. Afterwards, the "Shift A" command, in the defined program mode, will bring the computer back to the beginning of the program and the next ten stations may be entered.

The program includes a routine by which the stored data may be reviewed. The command "Shift S" will give the prompt "REV WCH ENT?" (Review Which Entry?). For example, joint #21 would be the first entry of the third group of ten stations (#21 to 30), so the number 1 should be entered. The command "Shift A" may be given at any time to resume data collection.

If an incorrect entry has been made, it may be corrected by using a "Shift C" command. The prompt "COR WCH ENT?" (Correct Which Entry?) will appear and should be entered as in the "Shift S" command above. The station number will be displayed. Press "ENTER" until the incorrect memory group (labeled A,B,C,D, or E) is reached. Enter the correct data when the appropriate prompt occurs. The "Shift A" command can then be used to resume normal operation.

The data tapes made in the field are analyzed by loading them into the microcomputer via the specially designed interface circuits. In most cases the analysis is fully automatic and takes about thirty minutes for one hundred stations. The analysis can typically be run while eating dinner and the day's work may then be reviewed in the evening.

After providing a hard copy printout of all of the collected data, the computer produces a rectangular dip vs. dip-direction plot (after Piteau, 1971) and then an equal-area net plot of either the discontinuity poles or the dip vectors. Examples of these printouts are shown on the following pages as Figures 5 and 6. The equal-area net plots are the same size as those provided in Hoek and Bray's text.

"Rock Slope Engineering" (1977)

The following types of analyses will be added to the system in the near future:

1. Cumulative Sums Analysis (after Piteau, 1971)
2. Equal Area Net Contouring
3. Cut Slope and Ø Circle plotting for additional graphical data analysis.

CONCLUSIONS

There are numerous advantages to this system. One is that an on-site preliminary computer analysis can be obtained before leaving the site. This analysis can pinpoint any data discrepancies or inconsistencies, thus showing a need for further work. Also, critical discontinuities can be identified quickly and hence, less time is directed toward measuring noncritical discontinuities on a subsequent day. Secondly, the more rapid nature of the data collection system will allow for more data to be collected in a given length of time than by conventional means. This will provide a more complete analysis and may prove to eliminate some of the statistical differences found to exist between the results of different investigators. Thirdly, the data are immediately in a form which may be read into the computer system without costly keypunching. Once the data have been loaded into the microcomputer, the microcomputer can either perform the preliminary analysis or transmit the data to a larger computer system for a more detailed analysis. The data transmission may be done by telephone from the field yielding a major time saving. All of these advantages add up to a system of rock-slope stability analysis which is less expensive, faster, and more complete than conventional means.

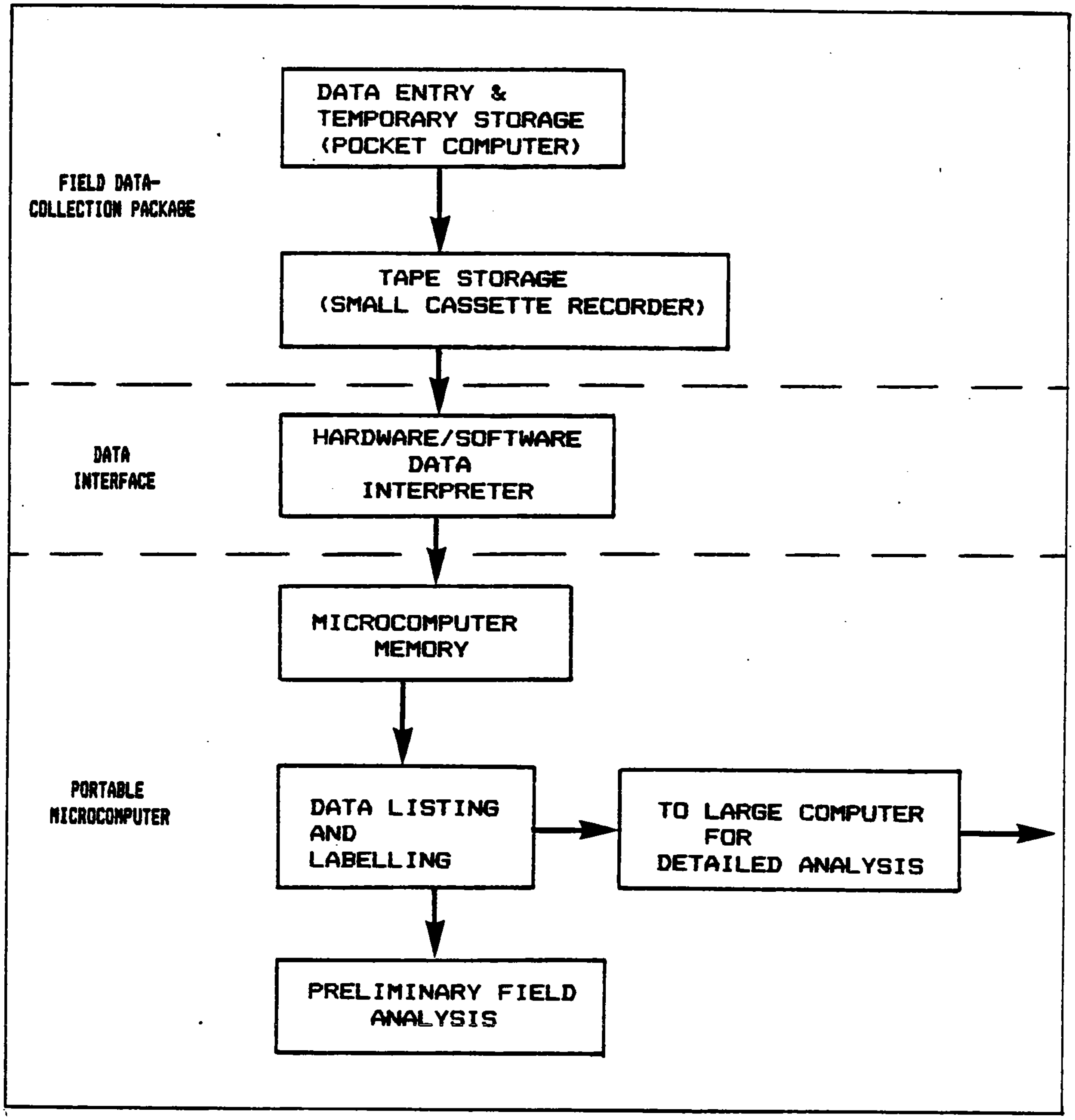


Figure 3. Flow Chart of the Data Collection and Evaluation System.

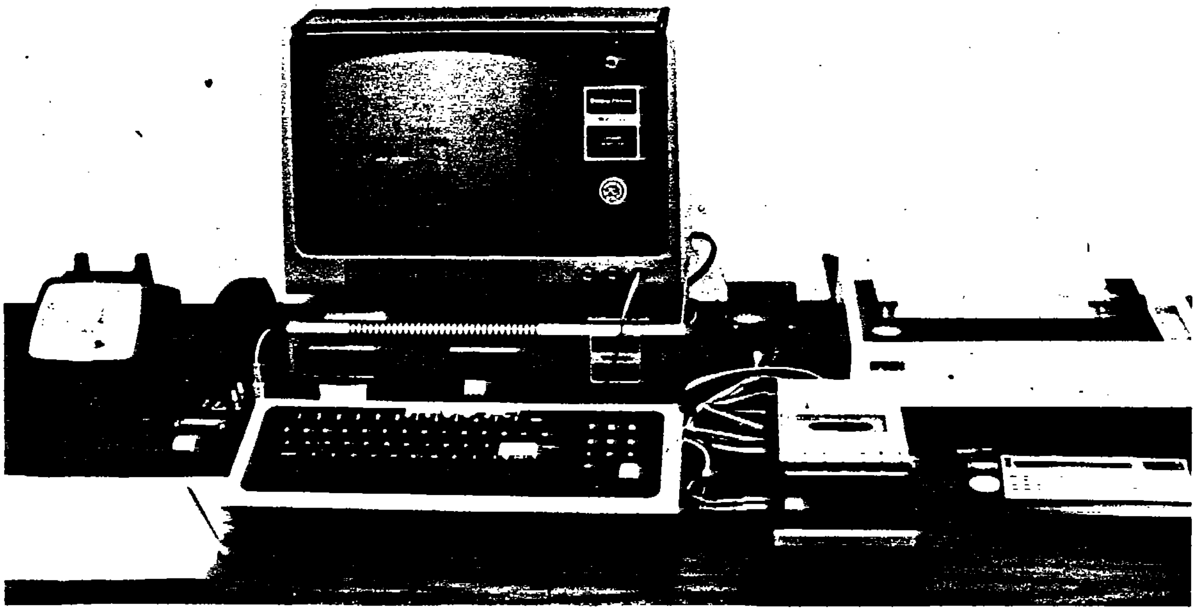


Figure 4. The Complete Data Collection and Evaluation System Developed at Purdue University.

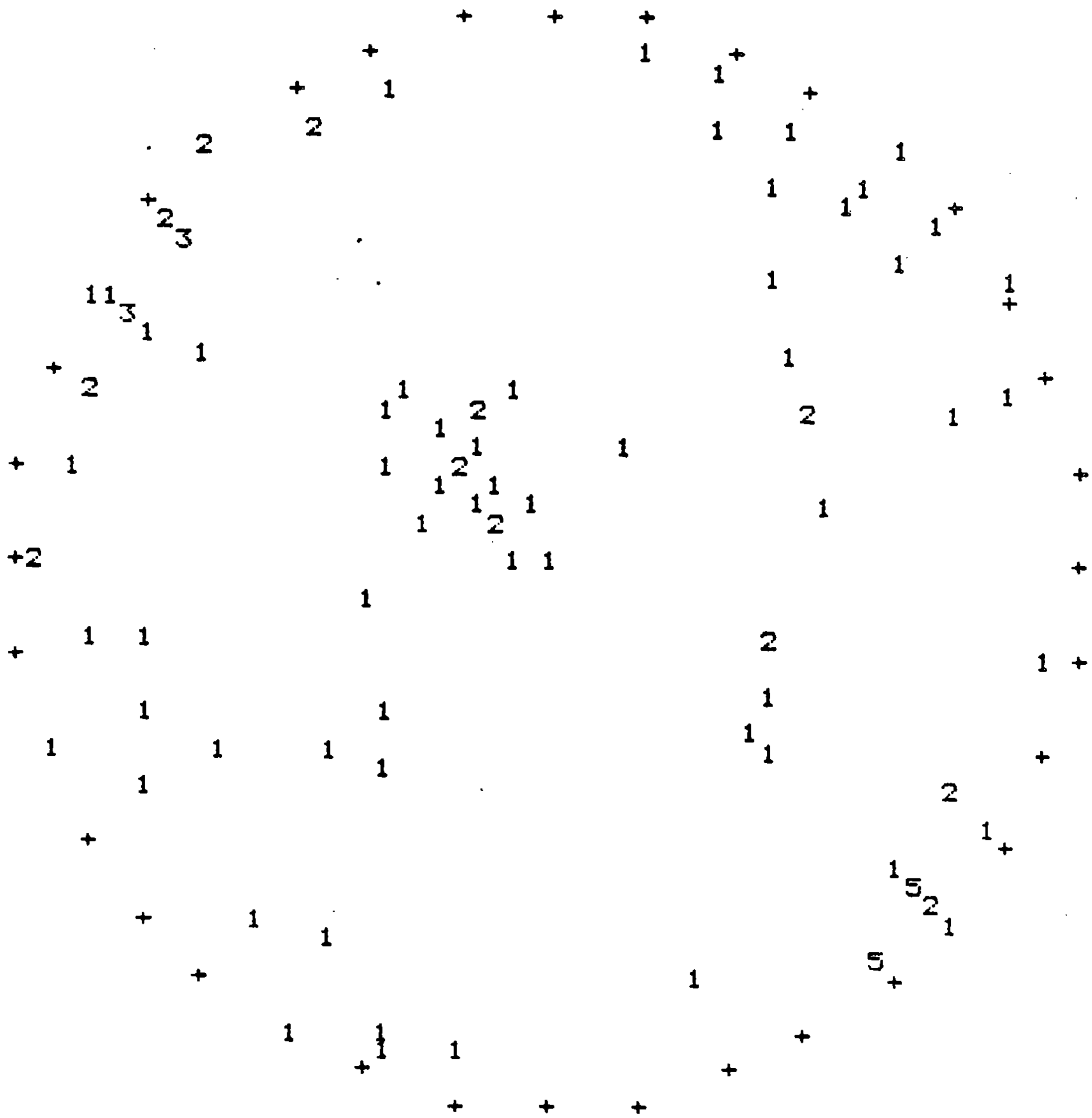
RECTANGULAR DIP PLOT

	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180
	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	1	0	0
15	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0
20	0	0	0	0	0	0	0	0	0	0	0	1	1	2	0	1	0	0	0
25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	0	1	0
30	0	0	0	0	0	0	0	0	1	0	0	0	1	0	0	0	0	0	0
35	0	0	0	0	0	1	0	0	0	0	0	0	0	1	1	0	0	0	0
40	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
55	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
65	0	0	0	0	0	0	0	0	1	0	0	0	1	0	0	0	0	0	0
70	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
75	0	0	0	0	1	0	1	0	1	0	0	0	1	0	0	0	0	0	0
80	0	1	1	0	0	0	0	0	0	0	1	2	3	3	0	2	1	0	0
85	0	0	1	0	0	0	0	0	0	2	0	0	1	2	0	0	0	0	0
85	0	1	1	0	0	1	0	0	0	0	1	0	1	2	5	0	0	0	0
80	0	0	0	1	1	0	0	1	0	0	0	0	0	5	0	0	0	0	0
75	0	0	1	0	1	1	0	0	0	0	0	0	2	1	0	0	0	0	0
70	0	0	0	1	0	0	0	1	0	0	0	0	0	0	0	0	1	0	0
65	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
60	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
55	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
50	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
45	0	0	0	0	0	0	2	0	1	0	0	0	0	1	0	0	0	0	0
40	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0
35	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360

90	0	0	0	1	0	0	0	1	0	0	0	0	1	0	2	0	0	0	0
90	0	0	0	0	1	0	1	0	0	0	0	0	0	1	0	0	0	0	0

TOTAL # OF STATIONS = 100

Figure 5. Output Sample: Rectangular plot for 100 stations.



OF STATIONS = 100

Figure 6. Output Sample: Equal area net polar plot for 100 stations.

It should also be noted that the usefulness of this system is not limited to rock slope studies alone. It could be a useful part of any study in which data for computer analysis is collected at a location which is not served by an electrical power source.

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APPENDIX A

Computer Programs for Pocket Computer Developed during this study.

1. Detail Line Survey

```
1 REM        ** POCKET COMPUTER PROGRAM **
2 REM          ** DETAIL LINE SURVEY **
3 REM          C.F. WATTS, PURDUE 1980
10 "A" GOSUB 190:IF C=0 THEN 80
20 I=INT ((80-C)/5): J=80-C-5*I: K=J+C : IF J=0 PRINT "FIN JN",INT (A(K+5)/E6): GOTO 85
40 PRINT"LF OFF JN",INT (A(K)/E6): IF J=1 PRINT "BEG. B": GOTO 110
50 IF J=2 PRINT "BEG. C": GOTO 120
60 IF J=3 PRINT "BEG. D": GOTO 130
70 IF J=4 PRINT "BEG. E": GOTO 140
72 PRINT "ERROR"
80 C=81: A(C)=9990000000: C=C-1
85 GOSUB 190:IF C<31 BEEP 3:PRINT "END MEM/SHIFT X":END
90 INPUT "A. JN#(4N)"; B: INPUT "TRTD(3N) ";D:INPUT "DIST(3N) ";E: A(C)=B*E6 + D*E3 + E
100 C=C-1: PAUSE "A10N",A(C + 1)
110 INPUT "B. RT(3A)HD(2A)ST(2A) ";A$(C)
115 C=C-1: PAUSE "B7A",A$(C+1)
120 INPUT "C. JNSP(2N)";B: INPUT "DIP DIR(3N)";D: INPUT "DIP(2N)";E: A(C)= B*E5 + D*E2 + E
125 C=C-1: PAUSE "C7N", A(C+1)
130 INPUT "D. JNL(3N) ";B: INPUT "JENDS(N) ";D: INPUT "FT(3N) ";E:INPUT "FTH(N) ";F
135 INPUT "FHD(2N)";G:A(C) = B*E7 + D*E6 + E*E3 + F*E2 + G
137 C=C-1:PAUSE "D10N",A(C+1)
140 INPUT "E. WTR(N) ";B: INPUT "RGS(N) ";D: INPUT "WIA(3N) ";E
145 INPUT "WL(2N)";F:A(C) = B*E6 + D*E5 + E*E2 + F
147 C=C-1:PAUSE "E7N",A(C+1):BEEP 2:PAUSE "NEW JN": GOTO 85
150 "S"GOSUB 190: INPUT "REV WCH ENT? "; Q: Q=(80-(Q-1)*5): PRINT A(Q): Q=Q-1: PRINT A$(Q)
160 FOR M=3 TO 5: Q=Q-1: PRINT A(Q): NEXT M: GOTO 150
170 "X" A(27)=9990000000:PRINT #"D";A(27):END
185 "C" GOSUB 190:INPUT "COR WCH ENT? ";Q: Q=(80-(Q-1)*5): R=C: C=Q: PRINT "JN ",INT  A(Q)/E6: GOTO 90
190 IF R=0 RETURN
195 C=R:R=0:RETURN
200 REM ** MEMORY SHOULD BE 57 **
```

2. Modified Detail Line Survey

```
1 REM        ** POCKET COMPUTER PROGRAM **
2 REM       ** MODIFIED DETAIL LINE SURVEY **
3 REM        ** JOINT ORIENTATIONS ONLY **
4 REM            C.F. WATTS, PURDUE 1980
5 "A" IF T=0 PRINT "DLS MODIFIED FOR JN ONLY"
10 IF C=0 THEN 80
12 IF C<31 PRINT "* CLEAR MEM *" : END
15 IF R<>0 GOSUB 190
20 I=INT ((80-C)/5): J=80-C-5*I: K=J+C : IF J=0 PRINT "FIN JN",INT (A(K+5)/E6): GOTO 85
40 PRINT"LF OFF JN",INT (A(K)/E6): IF J=1 PRINT "BEG. B": GOTO 110
50 IF J=2 PRINT "BEG. C": GOTO 120
72 PRINT "ERROR"
80 C=81: A(C)=9990000000: C=C-1
85 IF T=0 INPUT "LINE #= "; T
86 IF R<>0 GOSUB 190
90 INPUT "A. JN#(3N)"; B: D=0: E=0: A(C)=T*E9 + B*E6 + D*E3 + E
100 C=C-1: PRINT "A1ON",A(C + 1)
110 A$(C)= "AAABBCC"
115 C=C-1
120  INPUT "DIP DIR(3N)";D: INPUT "DIP(2N)";E: A(C)= B*E5 + D*E2 + E
125 C=C-1: PRINT "C7N", A(C+1)
130 B=1: D=1: E=1: F=1: G=1
140 A(C) = B*E7 + D*E5 + E*E3 + F*E2 + G: C=C-1
145 A(C) = B*E6 + D*E5 + E*E2 + F: C=C-1: IF C<31 PAUSE "END ALLOTTED MEM": PRINT "SET TAPE/SHIFT X"
147 PAUSE "*NEW JN*": GOTO 85
150 "S" INPUT "REV WCH ENT?"; Q: Q=(80-(Q-1)*5): PRINT A(Q): Q=Q-1: PRINT A$(Q)
160 FOR M=3 TO 5: Q=Q-1: PRINT A(Q): NEXT M: GOTO 150
170 "X" IF R<>0 GOSUB 190
175 A(30) = 9990000000: PRINT # "D"; A(30): END
180 "C" IF R<>0 GOSUB 190
185 INPUT "COR WCH ENT?";Q: Q=(80-(Q-1)*5): R=C: C=Q: PRINT "JN", A(Q)/E6: GOTO 85
190 C=R: R=0: RETURN
310 FILLERFILLERFILLERFILLERFILLERFILLERFILLER
320 FILLERFILLERFILLERFILLERFILLERFILLERFILLER
330 REM   ** USE FILLER AS REQUIRED TO BRING MEM TO 60 **
```

PREDICTION OF DEGRADABILITY FOR COMPACTED SHALES

by

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ABSTRACT

It may be desirable to mechanically degrade and compact shaley materials to high densities to (a) reduce the volume or (b) improve the stability of a fill composed of these materials. Prediction of mechanical degradability prior to placement operations has been achieved only in a qualitative sense, and on-site difficulties with improper material breakdown are all too common.

Research on Midwestern shales and siltstones has developed two simple procedures which may become standards for predicting mechanical degradability. These are (1) a conventional impact compaction test and (2) a point load strength test. The former test yields a density value for compaction control, and comparative gradations before and after compaction quickly show the change in mean aggregate size (index of crushing). The point load strength test is, of course, an extremely simple one, and the secant modulus derived from this procedure correlates well with the index of crushing.

The paper described the simple tests proposed for use in predicting mechanical degradability of shaley materials, and presents typical data obtained from several Midwestern rock members. These same materials have in the past produced millions of dollars of distress in embankments due to inadequate mechanical degradation during placement.

INTRODUCTION

Road cuts for highways constructed in the midwestern U.S. often encounter shale. Economic and environmental considerations generally make the use of the excavated material in nearby compacted embankment sections highly desirable. However, the poor strength and durability characteristics of many shales, along with inadequate construction procedures, have resulted in excessive settlement and slope failures of large shale embankments in several states (Chapman and Wood, 1974).

Problems in Indiana led to the initiation of research and development programs by the Indiana State Highway Commission through the Joint Highway Research Project at Purdue University (Wood, Sisiliano, and Lovell, 1978). Reports from these studies have been completed by Deo (1972) and Chapman (1975) on the classification of shales, Bailey (1976) and Hale (1979) on shale compaction and degradation characteristics, van Zyl (1977) on the storage and retrieval of existing data on Indiana shales, Abeyesekera (1978) on the shear strength parameters of compacted shales, Witsman (1979) on the compressibility of compacted shale, and Surendra (1980) on stabilization of compacted shales by additives.

The nondurability of shale is considered the major cause of the embankment problems. Nondurable shales will slake under the wetting and drying cycles of the service environment, and should be thoroughly broken down during construction to eliminate the presence of large voids within the compacted mass. Soft and nondurable shales are generally easy to break down during compaction. However, many shales in the midwestern U.S. are "hard" and difficult to degrade, requiring special compaction attention to increase the probability of a successful embankment service life.

The Importance of Shale Degradation Tests

The policy of thoroughly breaking down shales during embankment construction emphasizes the importance of understanding shale degradation resulting from compaction. Appropriate definition of the degradation functions would be most directly achieved through field compaction tests. Yet the expense of field tests and the limitations of present knowledge on shale degradation would reduce the effectiveness of any major field testing program. A standard compaction degradation test could be developed to generate the degradation functions in the laboratory, using an assortment of compaction variables. Ultimately, the results and experiences from the laboratory studies could be coupled with field compaction observations. When sufficient data are accumulated from both laboratory and field studies, the quantitative prediction of shale degradation during field compaction could be made from the results of laboratory testing only.

Work by Bailey (1976) established a basis for degradation studies of shale in the laboratory. Using Bailey's experience as guidelines, this research (Hale, 1979) was devoted to the development of a single standard test procedure and the application of the test to selected, troublesome midwestern U.S. shales. Correlations between the point load strength test and compaction degradation results were also developed.

Numerical Representation of Gradations

During the compaction process, fracturing, absrasion, and moisture effects break down individual shale pieces. The result is a compacted material with a gradation different from that of the uncompacted material. Therefore, a measure of the gradation change serves as an indicator of the amount of degradation that has occurred.

The index of crushing (IC) is a gradation index based on the summation of the weighted fractions of several size groups. Aughenbaugh et al. (1963) described the use of the IC as a measure of aggregate degradation during compaction. The percentage of the sample by weight within a size range is multiplied by a factor equal to the mean equivalent mesh size of that range. The summation of the values from each size group represents one gradation.

The actual IC value is computed as the difference between the numerical representation of the initial and final gradations and expressed as a percentage of the value from the initial gradation.

Upon inspection, the calculation of the gradation values used for the IC shows a great similarity to the calculation of the commonly used mean or average value in statistics. As explained by Harr (1977) the mean value of grouped data (K groups) is taken as

$$\text{mean} = \frac{\sum_{i=1}^{K} m_i \, f_i}{\sum f_i}$$

where m_i is the midpoint and f_i is the frequency of group i. With this explanation, the gradation values in the IC represent the mean or average aggregate size of the initial and final gradations. The IC is then a measure of change in the mean aggregate size.

As discussed by Bailey (1976) the IC enables the comparison of samples with dissimilar initial gradations. The successful use of the IC by Bailey, the ability to use the IC for samples with different initial gradations, and the concept of mean aggregate size led to the use of the IC as the primary measure of degradation for this study.

Selection and Description of Test Shales

Based on the results of a mineralogy study, three shales, New Providence, Osgood, and Palestine II, were selected for use in the testing program. New Providence shale lies at the base of the Valmeyeran (Osage) series of the Mississippian system. The shale is gray, has medium hardness, and is flaky. The Osgood shale is a member of the Salamonie dolomite and lies at the base of the Niagaran series in the Silurian system. The Osgood shale is blue-gray, hard, and flaggy. The Palestine shale is part of the Palestine sandstone formation in the Chester series of the Mississippian system. The Palestine shale is brown-gray, soft and flaky and can best be described as a transition between shale and sandstone. Further details of the shales are given by Hale (1979).

TESTING PROCEDURES

Sample Preparation

The excavation methods used to obtain the shale produced a number of large pieces. The large pieces were broken down with a carpenter's hammer to prepare samples suitable for testing. The broken shale was then dry sieved through a nest of sieves with mesh sizes of 38.1, 19.1, 9.52, 4.76, 2.38, 1.19, 0.59, 0.30, and 0.15 mm and a pan (U.S. Standard sieves 1 1/2 in, 3/4 in, 3/8 in, No. 4, 8, 16, 30, 50 and 100).

A 5.0 kg (11.0 lb) samples was prepared immediately before testing by blending the different sizes to fit the distribution of the equation:

$$P = (d/D)^{1}$$

where P is the percent passing any sieve, d is the sieve mesh size, and D is the top aggregate size (1 1/2 in). This gradation, called "A", provided a size distribution which emphasized the larger pieces but still included the finer sizes.

The shales were tested at their natural moisture levels to avoid the effects of either wetting or drying. Although the values of moisture content varied among the three shales, the moistures contents for samples from any particular shale were relatively uniform.

Impact Compaction

The compaction tests were similar to the Proctor-type compaction procedure in that a small faced hammer was dropped on the sample material a specified number of times. The equipment and compactive efforts were modified to satisfy the special needs of the testing program. A 15.24 cm (6 in) diameter steel CBR mold was used to accommodate the top aggregate size of the sample gradation. Bailey (1976) reported problems due to the additional degradation induced while removing the more tightly compacted shale samples of higher effort levels. To alleviate this problem and aid sample removal, all samples were compacted in a split CBR mold.

Different levels of compactive effort were controlled by the weight and drop of the hammer, the number of layers, and the number of blows per layer. After compaction the mold collar was taken off. Next, any excess material above the mold rim was gently removed, before weighing the sample for the density calculation. The compacted sample was then separated by hand and recombined with the excess material that had been previously removed. All of the shale was dry sieved through the nest of sieves used in the sample preparation. The index of crushing (IC) was determined once the initial and final sample gradations were known.

After considerable experimentation (Hale, 1979), an effort level of 790 $\frac{\text{kN-m}}{\text{m}^3}$ or 16,500 $\frac{\text{ft-lb}}{\text{ft}^3}$ was selected as the most appropriate one for a standard compaction degradation test. This effort was achieved by 30 drops of a 10 lb hammer from a height of 18 in. on each of 3 layers.

Use of the Standard Impact Test to Measure Shale Degradability

The samples were all blended to fit gradation A and were tested at their natural moisture contents. The compaction and subsequent sieving to measure degradation followed previously described procedures. Representative aggregate size distributions are illustrated in Figure 1.

A successful degradability prediction test for shales must be able to differentiate among various shales. The standard compaction procedure developed in this testing program demonstrated such a capability. The effects of compactive effort, initial gradation, and maximum size were studied earlier (Hale, 1979). It was demonstrated that a change in any one of these variables would change the degradation. Therefore, the sample and compaction variables must be held constant.

The 790 $kN\text{-}m/m^3$ (16,500 $ft\text{-}lb/ft^3$) effort level is reliable and should be retained for the degradability index test. The "A" gradation with its exponential cumulative gradation curve and 38.1 mm (1.5 in) top size, provides a reasonable model for the gradations which may be encountered during the construction of a compacted shale embankment. However, the entire "A" gradation is not completely necessary. Compaction tests were also performed on samples containing only the top four size groups of gradation A (38.1 - 19.1 mm (1 1/2 - 3/4 in), 19.1 - 9.52 mm (3/4 - 3/8 in), 9.52 - 4.76 mm (3/8 - No. 4 sieve), 4.76 - 2.38 mm (No. 4 - No. 8 sieve)). Using the partial "A" gradation simplifies the testing procedure. The use of only the four top sizes also emphasizes the role of the larger shale pieces in the degradation process and may therefore lead to a more distinct classification among shales. For these reasons, the partial "A" gradation is recommended when using the standard impact test to classify the degradability of a shale.

Moisture will also affect the degradability of a shale. The testing of all shales at a standard moisture level would provide a more accurate method of comparing the materials. However, a standard moisture level is difficult to establish for shales. Drying all shales to zero moisture content would establish such a moisture standard. However, Bailey (1976) reported the formation of cracks in samples oven dried for the point load strength test. Large variations in the strength index were attributed in part to these cracks. Based on these observations, the formation of cracks in the aggregates during the drying process could produce large variations in the degradability of a sample. Also, the degradability of a shale at zero moisture content would be difficult to correlate with the degradability of the same material at its natural moisture content during field compaction.

Complete saturation could also be considered as a standard moisture. However, the serious effects of moisture on shales (slaking and strength reduction) eliminate this alternative. The natural moisture level appears to be the most realistic standard. Although the natural moisture contents will vary somewhat among the shales, this level will allow the classification of shales for their in-situ moisture conditions. By using the natural moisture level, correlations between the index test and field degradation should be easier and the problems of moisture manipulation can be avoided.

Use of the Point Load Strength Test to Predict Shale Degradability

The uniaxial and triaxial compression tests are widely accepted methods of determining the strength of rock. However, the sample preparation necessary to obtain the required specimen size and shape generally limits the use of the test to the stronger rock materials which can withstand the preparation techniques and are not broken down by water. The specimen requirements greatly restrict the use of conventional compression tests for shales and other weak rocks.

Broch and Franklin (1972) suggested the use of a point load to simplify the testing of irregular specimens. They found that the specimen size and shape effects were more severe for irregular lumps than for cores or disks, thus making the test less precise for those conditions. Broch and Franklin concluded that despite the reduced accuracy, the point load strength (PLS) test on irregular lumps was a suitable method for strength classifications of rocks.

The point load index (I_s) is based on the formulation $I_s = P/D^2$ where P is the failure load and D is the initial distance between the loading points. The index and testing procedure were adopted by the International Society of Rock Mechanics in 1972. Additional work by Guidicini et al. (1973) and Bieniawski (1975) added support to the application of the point load index in rock mechanics, mining, and engineering geology.

Features of the Point Load Strength Test

The combined features of the PLS test are especially suited for testing shales. The elimination of special sample preparation allows the testing of weak or thinly laminated shales. The time saved by avoiding sample preparation, along with the short duration of the actual test, makes the testing of a large number of samples feasible. This capability is important considering the natural variations found in shales. The short time required to test an individual specimen also reduces the changes in sample moisture content which may occur during testing. The use of portable equipment allows the testing of shales during the subsurface investigation programs and at the construction site.

Samples for the PLS test require no special preparation. However, experience has shown that sample size has a major effect on the PLS index (Brook, 1977). The pieces selected for testing should be roughly equidimensional (Broch and Franklin, 1972). Since the fissility of a shale tends to produce flaggy or platey, rather than bulky, pieces, obtaining samples of uniform size becomes difficult.

To evaluate the size effect in shales, approximately 150 samples were prepared from the New Providence and Attica shales of Indiana for point load strength (PLS) testing. The samples varied in thickness from 5.0 mm (0.20 in) to 38.0 mm (1.5 in). The data from the tests were grouped into size ranges of 6.4 mm (0.25 in). As shown in Figure 2, the mean value of the PLS index from each size group decreased as the sample size increased.

Laboratory Testing

Samples of the test shales (New Providence, Osgood, and Palestine) were prepared for point load strength (PLS) testing. The results of the size effect study had indicated that the variation in the PLS index decreased as the sample size increased. Also, the higher failure loads associated with larger samples could be more accurately monitored from the loading device. For these reasons, the samples were prepared as large and uniform as possible.

The PLS test apparatus used in the study conformed to the standards of the International Society for Rock Mechanics (1972). The entire PLS apparatus was placed between the platens of a compression testing machine for loading. Samples with apparent bedding were placed in the PLS apparatus so that the load would be applied perpendicularly to the bedding planes. This was the direction of greatest strength for such shale types. After placing a sample between the points, a small seating load was applied, and an initial reading of the apparatus dial gage was used to calculate the sample thickness. The samples were tested at their natural moisture contents and subjected to a loading rate of 0.03 cm/min (0.01 in/min). Both load and sample deformation were monitored at regular time intervals using readings from the load gage of the compression machine and from the dial gage of the PLS apparatus.

Results and Discussion

The hypothesis adopted for this testing sequence was that the PLS test could provide a strength parameter indicative of the resistance of the material to degradation. However, PLS values were rather similar and did not correspond to the order or range of index of crushing (IC) values observed for the three shales.

With the rejection of the P/D^2 values, another parameter that could be obtained from the PLS test was necessary. Observation of the load-deformation relationships for each shale indicated that the slope of the resulting plot might provide a suitable parameter. Using the P/D^2 value as a stress term and Def./D as a strain term, a modulus form $[(P/D^2)/(\text{Def.}/D)]$ was established. A comparison of the modulus and degradation values is given in Table 1.

A regression analysis performed on the data produced an exponenetial curve with the equation below for the modulus expressed in psi,

$$\ell n\ IC = -3.289 \times 10^{-4}\ (\text{modulus}) + 4.10$$

Although the curve was based on limited data, predictions of the index of crushing (IC) from some limiting modulus values seem reasonable. The upper bound of the IC as the modulus approaches zero is appropriate for the set of conditions used in the standard compaction-degradation tests. The equation also gives reasonable lower levels of the IC for the higher modulus values possible in stronger shales.

CONCLUSIONS

The following conclusions are based on the results of the compaction degradation and point load strength tests on three Indiana shales.

1. The standard compaction degradation test may be used successfully to classify the degradability of shales. The partial "A" gradation is recommended to simplify the test and to place a greater emphasis on the degradation of the larger aggregate sizes. Modifications in the standard test may become necessary as more experience is gained.

2. Results from the point load strength tests indicate that the commonly used P/D^2 index is not a sufficient indicator of shale degradability. However, the modulus from this study provided a good correlation with degradation values and led to the formation of a reasonable prediction equation for the index of crushing.

Finally, the ability to classify shale degradability from the standard impact test or the point load strength test could ultimately lead to improved and more efficient design and construction procedures for building compacted shale embankments.

ACKNOWLEDGEMENTS

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Table 1. Comparison of Modulus and Shale Degradation Values

Shale	Modulus MPa (psi)	IC (%)
New Providence	9.44 (1369)	37.6
Osgood	18.49 (2681)	25.1
Palestine	3.38 (490)	51.8

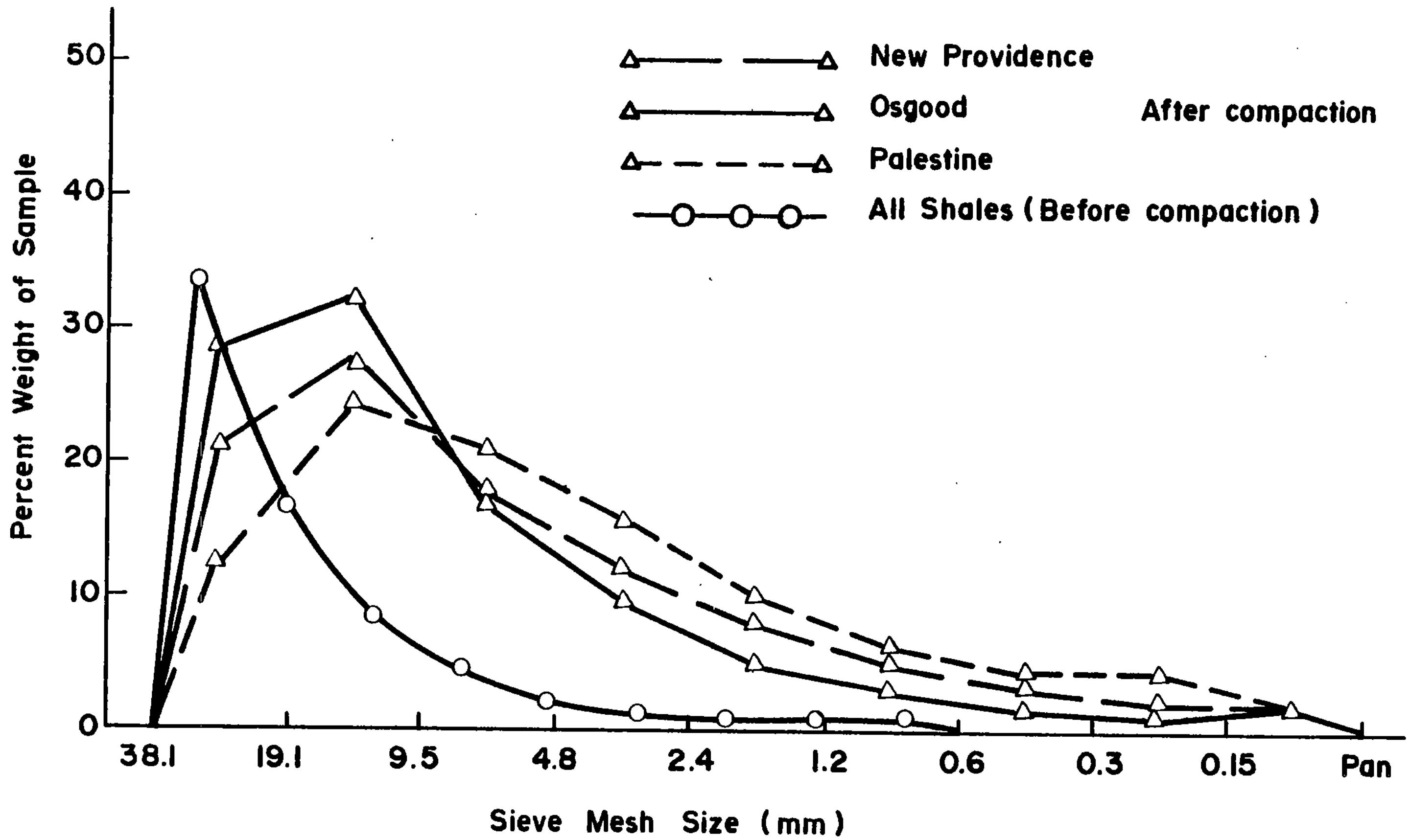

Figure 1: Aggregate Size Distribution for New Providence, Osgood, and Palestine Shale

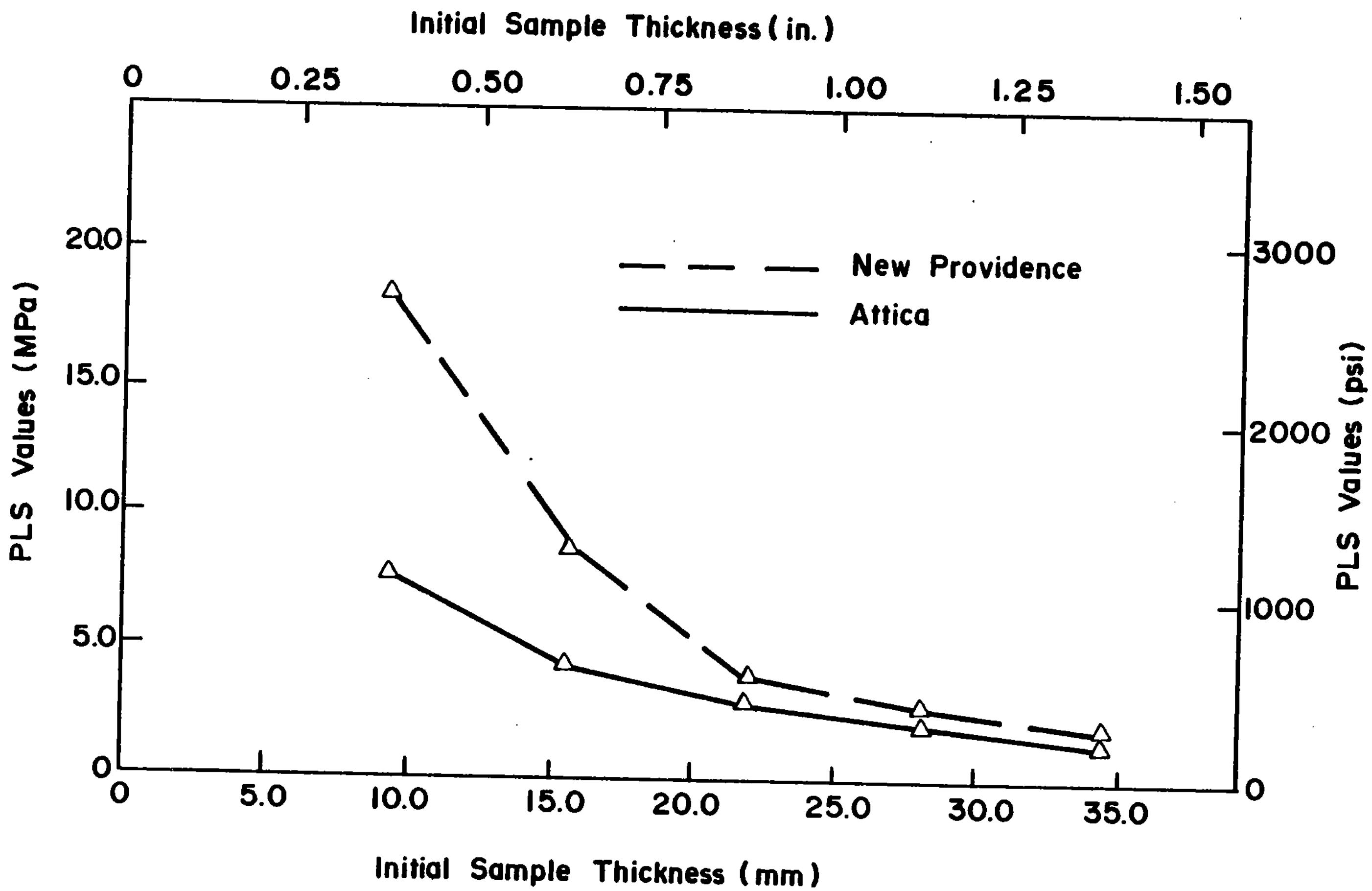

Figure 2 : Size Effect on Point Load Strength Values

A Review of the Progress of the Wyoming Heat Pipe Program

by

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I. Introduction

In the spring of 1976 the Wyoming Highway Department (WHD) and the University of Wyoming (UW) initiated a series of agreements to conduct a research program on the thermal control of bridge decks using the heat pipes. The principal objectives of the program were:

1. Generate an empirical data base on the insitu performance of gravity operated heat pipes installed on a bridge deck.

2. Using the experimental results, develop design algorithms suitable for use by highway engineers interested in utilizing heat pipes for snow and ice control.

3. Investigate practical problems associated with utilization of heat pipes in bridge decks including:

    a) Economics

    b) Expansion and contraction of piping

    c) Consideration of deck motion

    d) Logistics of heat pipe installation during the construction program

    e) Alternative materials suitable for heat pipe fabrication

In the past five years three different projects have been initiated under the program. Under the first agreement an experimental program was conducted to address the first and second objectives. A limited number of heat pipes and requisite instrumentation were incorporated in the construction of a new bridge (Sybille Creek Project). The performance of the heat pipes was monitored continuously for 2 1/2 years. Experimental data obtained served as the basis for development of some of the algorithms required to meet the second objective.

Based on the results of the Sybille Creek Project, a Wyoming Highway Department and Federal Highway Administration demonstration project, initiated in the spring of 1980, is currently nearing completion (Spring Creek Project). Objectives of this project include, demonstration of a practical heat pipe system for an entire bridge deck, securing additional experimental data and completion of a designers handbook for application of heat pipes to bridges.

A third project, started in the fall of 1980, involves installation of a heat pipe system in the approach ramps to the Union Pacific Railroad overpass on I-180 in Cheyenne, Wyoming (I-180 Project).

This paper will briefly review the Sybille Creek Project, describe the Spring Creek Project in some detail, and indicate the impact that these projects have had on the design of the I-180 heat pipe system.

II. Description of a Heat Pipe

A gravity operated heat pipe, as shown in Figure 1, consists of a closed vessel containing a working fluid in both the liquid and vapor states. When the heat pipe is operating, energy from an external source is conducted into the evaporator section located at the bottom of the heat pipe. This energy vaporizes some of the liquid and the vapor is convected up into the condenser where it condenses back into a liquid liberating the heat of vaporization to the condenser walls. The condensate flows back to the evaporator under the influence of gravity. When the vapor condenses on the walls of the container it liberates the heat of vaporization. Since the thermal energy is transferred in the form of latent heat, the heat pipe is an essentially isothermal device with extremely high conductance.

Note that the evaporator section must be hotter and spatially lower than the condenser. The vapor flow is maintained by the pressure gradient established due to the temperature difference between the evaporator and condenser. Although many fluids could be used as the working fluid, the high latent heat and low freezing temperature of ammonia make it particularly suitable for highway applications and it has been used on all Wyoming projects. The geometry of the heat pipes is relatively unconstrained. Container materials are limited by compatability with the working fluid, the environment of the external surfaces, and strength since pressures of the order of 2.1×10^4 pascals (100 psia) can be developed inside the container. All Wyoming heat pipes have been fabricated from mild steel tubing or ASTM A106 steel pipe.

III. Sybille Creek Project

Nine standard heat pipes (Figure 2) 24.4m (80 feet) long and 2.5cm (1 inch) outside diameter were installed transverse to the direction of traffic flow 5cm (2 inches) below the deck surface on 15cm (6 inches) centers. These pipes extend from depths of 15m (50 feet) in the earth up through the earth's surface and through the edge of the deck to the bridge centerline. Performance of the heat pipe system was monitored and recorded continuously at one minute intervals using a variety of instrumentation transducers and a digital data acquisition system. In addition the surface conditions on the deck and the adjacent roadway were recorded photographically at five minute intervals during daylight hours. A plan view of the experimental site is shown in Figure 3.

The Sybille Creek project has been thoroughly documented by Cundy[1], however; the principal results will be reviewed here. Although the upper surface of the heated portion of the deck did not remain unfrozen throughout the entire observed period, this surface experienced less than half of the total frozen time of the approach roadway surface. The heated section of the bridge deck was never observed to be preferentially iced relative to the approach roadway. A maximum power of 200 W/m^2 (63.5 Btu/hr-ft^2) and an average of 90 W/m^2 (29 Btu/hr-ft^2) were delivered by the heat pipes.

The temperature of the earth surrounding the heat pipes was depressed by as much as 7.0°C (12.6°F) which decreased the output power in the early

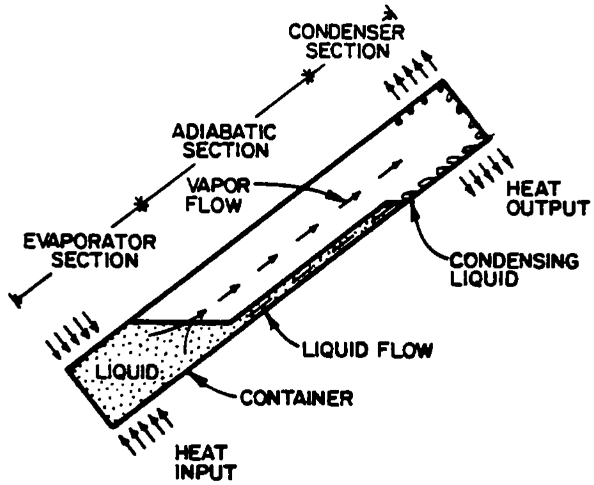


Figure 1. Schematic illustration of a heat pipe

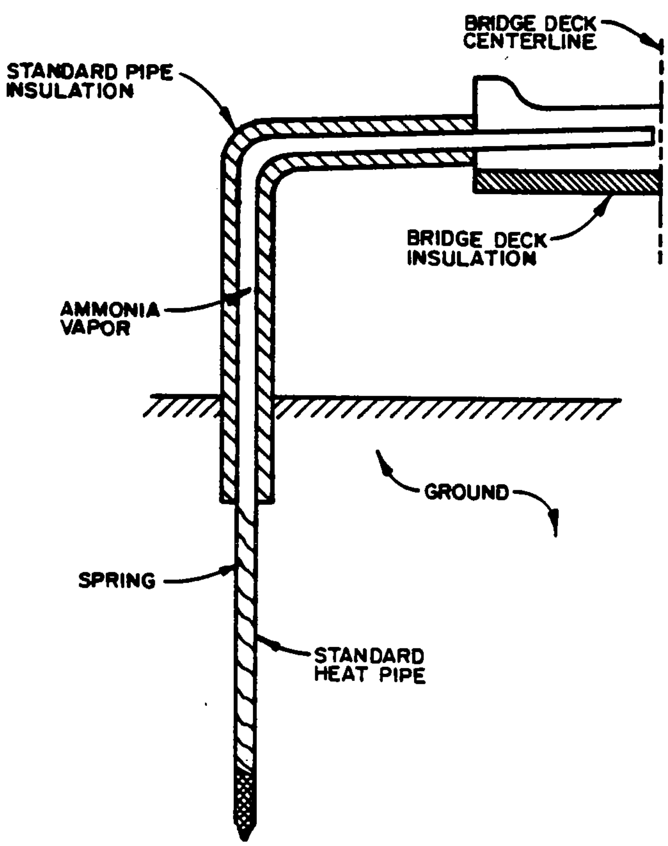


Figure 2. Sybille Creek standard heat pipe

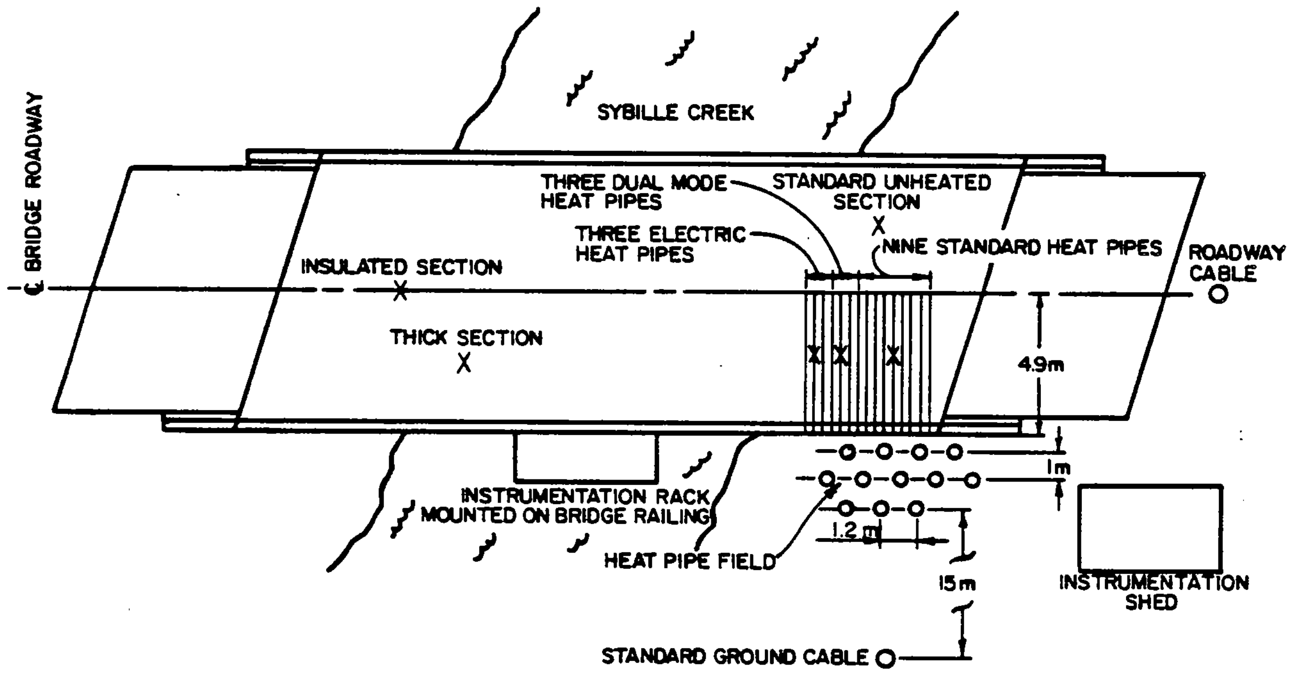


Figure 3. Plan view of Sybille Creek experimental site

spring months, however; the earth recovered to essentially the undisturbed ground temperature during the summer months.

Based on the results obtained, a variety of conclusions were drawn. It was concluded that:

1. Heat pipe systems are effective in eliminating preferential icing of bridge decks.

2. Surface heat fluxes considerably lower than those recommended by the ASHRAE Guide and Data Book(2) are adequate for highway applications.

3. Accurate computer simulations of the thermal response of both the unheated and heated portions of the deck could be made although the thermal model of the earth portion of the heat pipe system required further study.

4. In order to improve the cost/benefit ratio for the heating system, manifolding of several condenser elements on a single evaporator should be investigated.

IV. Spring Creek Project

The Spring Creek project was initiated in the Fall of 1979 with a heating system design agreement between the University of Wyoming and the Wyoming Highway Department. In addition to the usual test borings the Geology Department (WHD) drilled a 31m (100 foot) hole proximate to the proposed construction site. Core samples were taken on ten foot intervals and provided to the University for analysis. The University prepared a temperature instrumentation cable with thermocouples located on ten foot intervals which was placed into the hole which was in turn back filled with sand.

The site of the Spring Creek heat pipe installation is located in the western flank of the Laramie Mountains. The pipes are founded in the Permian Satanka formation. The Satanka consists primarily of reddish brown siltstone and silty shales with minor amounts of gypsum. The Satanka is overlain by alluvial deposits of silt, sand and gravel varying in thickness from 1.5m (5 feet) to 4.6m (15 feet).

Tectonically, the area is traversed by several small faults resulting in the bedrock being highly fractured. Weathered fractures in the upper 1.2m (4 feet) to 1.5m (5 feet) of the Satanka bedrock provide the aquifer for the upper groundwater.

The Satanka in this area is underlain by the Casper formation of Pennsylvannian age. The Casper sandstone is the main source of water for the City of Laramie and to avoid affecting this aquifer the depth of the heat pipes was restricted.

The properties of the core samples as determined by UW personnel may be contrasted with the properties previously determined for the Sybille

Canyon site in Table 1. Subsurface temperature measurements were recorded periodically for approximately one year prior to actual construction. The data are presented in Figure 4.

In addition to these field studies two laboratory experiments were conducted to generate data for use in the design of the heating system. The two problems addressed were the manifolding of a number of condensers, and the effect of nonuniform temperature on a long condenser.

Dynatherm Corporation installed heat pipes with two condensers on a common evaporator in a test slab in Virginia. These heat pipes failed and it was generally concluded that it was due to inadequate internal cleaning of the pipes. The question of whether manifolded condensers with the condensers exposed to different temperatures or perhaps a temperature distribution would operate stably delivering energy to all the condensers was addressed in these experiments. Three condensers, each 1m (3 feet) long were manifolded together and each condenser equipped with a water jacket as shown in Figure 5. A different water flow rate was provided for each jacket. It was found that the heat pipe did operate stably delivering energy to all three condensers.

The effect of nonuniform imposed temperature on a long condenser was tested using a 16.3m (35 foot) long condenser equipped with three water jackets, which were subjected to differing flow rates. Once again the heat pipe was found to be stable delivering energy throughout its length.

With the properties of the subsurface materials known and the questions of manifolded condenser utilization resolved the thermal design of the heating system was addressed.

V. Power and Energy Requirements

This study was initiated by calculating the powers and energies required to prevent a surface which is insulated on the underside from ever falling below 1°C (model A) or from ever falling below 1°C during precipitation events (model B) where the frozen precipitation was assumed to be melted and drained as rapidly as it fell in both cases. These are deterministic calculations which characterize the system's environment in terms of power spectra and total energies. A detailed description of the surface heating model A is available in reference (1).

In the 1950's W. P. Chapman and his colleagues developed a similar snow melting model and his results are used as the basis for the power and energy requirements that are recommended by the American Society of Heating, Refrigerating and Air-Conditioning Engineers[2] in the design of snow melting systems. However, the expressions he used for the various modes of heat transfer are not as comprehensive as those included in the present study. Chapman had to select particular events from historical weather records for detailed calculations whereas the present study involved continuous hourly calculations over a twenty-four year period.

A system designed to just prevent the preferential freezing of one surface relative to another will not of course require the capacity pre-

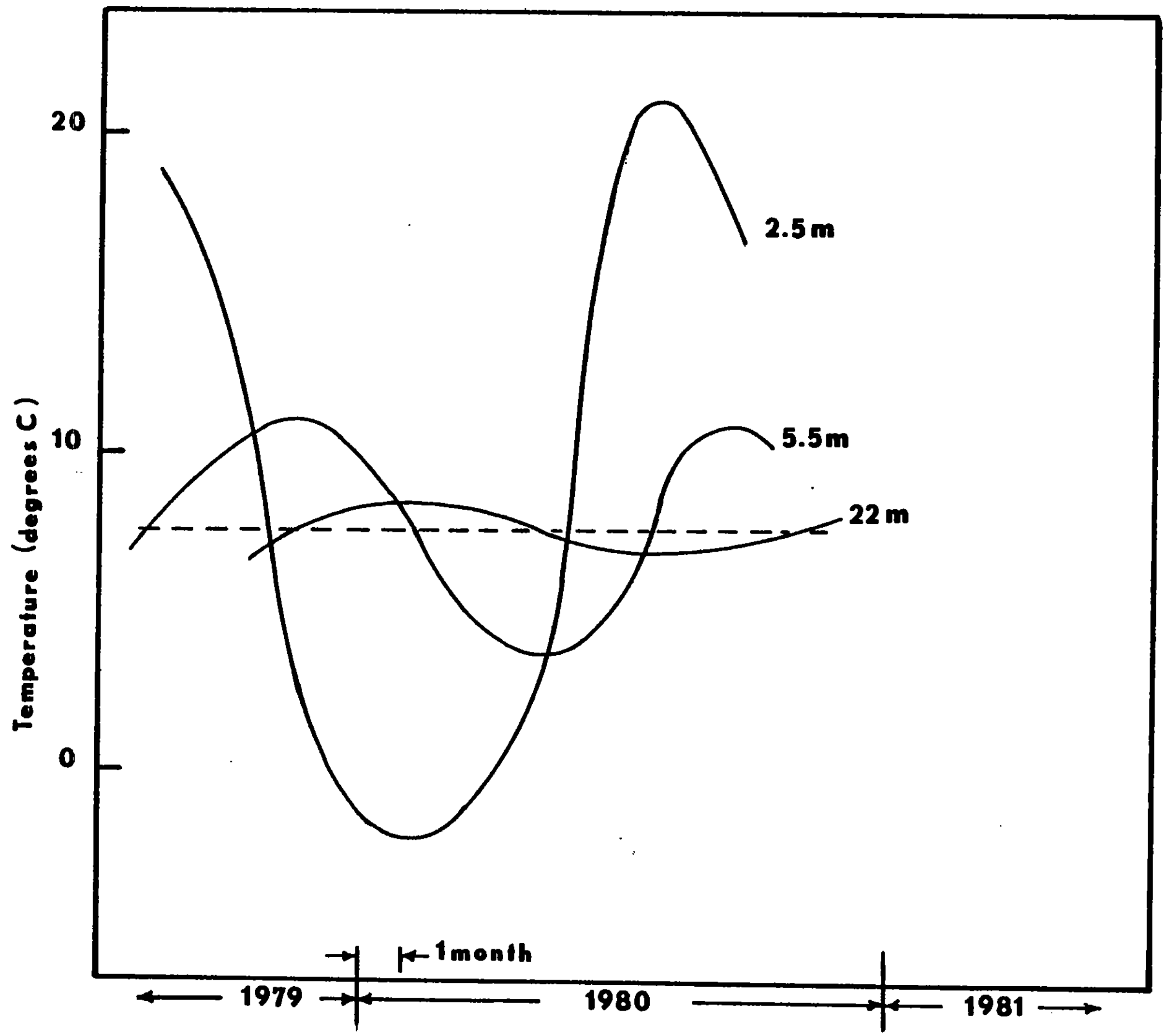


Figure 4. Seasonal temperature variation with depth as a parameter

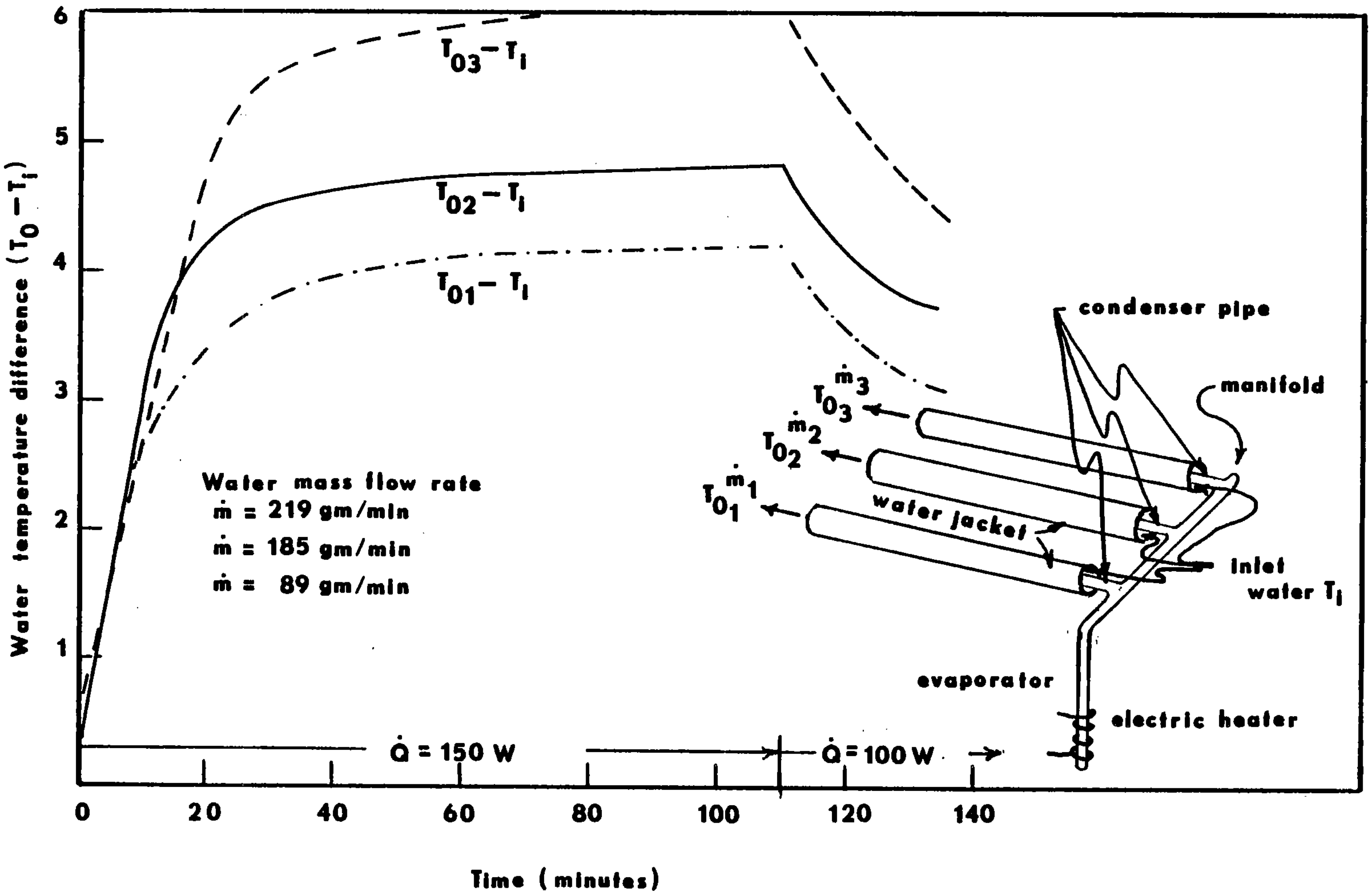


Figure 5. Temperature response of manifolded condensers

Table 1.

Thermal properties of subsurface materials at the Sybille Canyon and Spring Creek sites.

Property	Spring Creek	Sybille Canyon
Thermal Conductivity (k)		
W/m °C	2.16	1.21
Btu/hr ft °F	1.25	.70
Specific Heat (c_p)		
J/Kg °C	654	675
Btu/lb °F	0.155	0.160
Density (p)		
Kg/m^3	2520	2419
lb/ft^3	157	151
Thermal Diffusivity (d)		
m^2/hr	4.7×10^{-3}	2.7×10^{-3}
ft^2/hr	0.051	0.029

dicted by the above models and will be, in this sense, an underdesigned system. Analytical performance predictions of any underdesigned system are very problematic since it is difficult to accurately calculate the difference in the surface conditions between an unheated surface and a surface that is not sufficiently heated to prevent both snow accumulation and refreezing.

An empirical estimate of a system's performance in terms of both prevention of preferential icing and snow melting capability can be made by comparing the estimated heating characteristics of a proposed system with those required by models A and B and relating these to the observed performance of an existing system in a comparable environment. In this case, the performance of the Sybille Canyon prototype system and Cheyenne weather data were used as the empirical basis to estimate the performance of the Spring Creek system. Cheyenne weather data was used since Laramie historical weather data was not available. This section will describe Cheyenne's pertinent weather characteristics and the results of the two heating models.

The weather data required to implement the above analysis were obtained from the National Oceanic and Atmospheric Administration, Environmental Data and Information Service, National Climatic Center, Asheville, North Carolina in the form of SOLMET (Hourly Solar Radiation-Surface Meteorological Observation) and hourly precipitation data tapes. The SOLMET merges all the meteorological data recorded at Cheyenne Municipal Airport (WBAN No. 24018) with a regression estimate of the hourly surface solar radiation which is based upon recorded cloud cover data.

A summary of the pertinent environmental parameters in the snow melting models is given in Table 2 along with the corresponding total energy required (model A) and energy required during the precipitation events (model B) for the 24 winters between 1952 and 1976. A review of this table indicates a large variability in many of the listed parameters. The yearly average solar radiation was 196.3 $\pm$ 5 w/m^2 but it should be noted that this is derived quantity; the average annual air temperature was 7.8 $\pm$ .6°C (46°F); the average wind speed was 6.0 $\pm$ .3 m/sec (13.4 mph); the average total time at or below 0°C was 152 $\pm$ 11.3 days (3648 hours); a modified freezing factor, which was defined by the following integral:

$$\int_{o}^{1\ \text{Year}} \Theta\, u\,(\Theta)\, dt$$

where Θ = ambient air temperature -1°C, u is Heaviside's unit function

$$u(\Theta) = \begin{cases} 0 \text{ when } \Theta \geq 0^\circ C \\ 1 \text{ when } \Theta < 0^\circ C \end{cases}$$

and dt is the differential time. The integral had an average of 452.6 $\pm$ 236.5 degree centigrade days; and the average number of hours of measurable frozen precipitation was 59.7 $\pm$ 38.2 hours which is only 16% of the average total time recorded as having snowfall (380.4 $\pm$ 207 hours) since most of the snowfall was recorded as trace amounts. The ASHRAE Guide states that a snow melting system in Cheyenne, should be designed for 138 hours of snowfall greater than 0.01 inches of water equivalent per hour which is in fact the maximum measurable snowfall value that appears in Table 2. The record snowfall of the 1979-1980 winter does not appear in this table.

The ASHRAE Guide also states that on the average 46.4% of the winter hours (Nov. 1 to March 31) have no measurable snowfall and are above 0°C while 49.8% are below 0°C with this latter period having a mean temperature of -5.83°C (21.5°F) and a mean wind speed of 6.84 m/sec (15.3 mph). These ASHRAE statistics were not calculated using the SOLMET data base but a cursory check indicates that they are probably compatible.

The annual average energy required to hold a surface at 1°C was calculated to be 292.1 $\pm$ 157 kwh/m^2 for the 24 winters examined, but Table 2 suggests that Cheyenne does not have an average winter in this sense. The winters are either very severe in that they require 400 kwh/m^2 or more as 10 out of the 24 winters did, or the winters are relatively mild in that they require less than 235 kwh/m^2 as did 1/2 of the winters examined. A similar division occurs in the energy required to hold a surface at 1°C during the precipitation events where between 40 and 90 kwh/m^2 was generally required during the severe winters and less than 20 kwh/m^2 would have been sufficient for most of the mild winters.

The design suggested in the ASHRAE Guide requires a total energy of 505 kwh/m^2 of which 64 kwh/m^2 is required during the measurable precipitation events. These values appear to be in reasonable agreement with the maximum values that appear in Table 2.

Winter	Total Energy Required (kwh/m^2)	Energy During Precipitation (kwh/m^2)
1961-1962	533	52
1956-1957	457	90
ASHRAE[2]	505†	64

The average spectrum of required powers for the twenty winters between 1952 and 1972 is presented in Figure 6 along with the spectrum for the worst winter (1961-1962) of the 24 Cheyenne winters studied, and the ASHRAE Guide design spectrum for measurable precipitation events. This figure indicates that there is little difference between the average year and the worst year SOLMET spectra but there is a large difference between the ASHRAE and SOLMET power spectra for the precipitation events. For instance, the SOLMET data imply that a 100 w/m^2 system could keep the surface above 1°C 48% of the time requiring power and melt the snow as fast as it fell around 28% of the time as compared to only 10% of the time when the ASHRAE design spectrum is used.

The ASHRAE data and the SOLMET data agree that the peak required power is around 1600 w/m^2 (see Table 2 and Figure 6) but it is felt that the ASHRAE spectrum is not representative of Cheyenne's weather. Also shown on Figure 6 is an envelope of the spectra for Dodge City, Salina, New York, Madison, Minneapolis and Chicago based on data calculated using models A and B as well as ASHRAE Guide data. This figure indicates that all the cities including Madison and Minneapolis have very similar power spectra which are much lower than the ASHRAE Cheyenne spectrum.

The power spectrum for Sybille Canyon for the winters of 1977-1978 was calculated from the experimental field data and the spectra for both years are also very close to the average Cheyenne spectrum given in Figure 6. The required energies (model A) for these two years are 346.2 kwh/m^2 and 498.6 kwh/m^2 respectively. The 1978-1979 winter is seen to have been a very severe one -- comparable to the 1961-1962 winter in Cheyenne which was the worst winter recorded on the SOLMET data base.

In summary, the Sybille Canyon test site and Cheyenne are very similar in their power spectra and energy requirements. The recorded performance of the Sybille heat pipe system should therefore give a good indication of how a system with similar heating capabilities would perform in an environment similar to Cheyenne's.

† Idling energy plus Class III melting energy.

Table 2 Summary of Weather Characteristics for Cheyenne

Year	Av. Temp. (°C)	No. Days 0°C or Below	Deg-Days 1°C or Below	Av. Wind Speed (m/sec)	Meas. Snow (Equiv. mH_2O)	Hrs. of Snowfall	Hrs. of Meas. Frozen Precip.	Av. Solar Radiation (w/m^2)	Total Energy Required (kwh/m^2)	Energy During Precip. (kwh/m^2)	Peak Power (w/m^2)
1952-1953	7.87	167	596.8	6.18	.2510	562	127	197.8	431.3	82.50	1607.9
1953-1954	8.77	144	500.2	5.76	.1303	465	73	198.3	351.2	56.07	1604.5
1954-1955	7.89	160	682.3	6.34	.0832	412	65	197.6	470.6	54.09	1214.8
1955-1956	8.08	152	729.3	6.03	.0968	570	73	199.6	453.5	42.78	965.3
1956-1957	7.35	153	615.1	6.56	.2523	644	138	185.7	456.7	90.43	1207.1
1957-1958	7.70	157	557.6	5.88	.1349	686	130	188.8	380.4	56.93	704.6
1958-1959	7.60	156	657.0	6.42	.1175	654	98	192.9	426.7	57.75	828.2
1959-1960	7.58	163	756.3	6.40	.0826	679	88	194.1	466.8	39.63	829.3
1960-1961	8.15	153	572.6	6.26	.1632	618	102	192.8	397.3	40.28	787.3
1961-1962	6.48	161	902.9	5.93	.0916	632	74	194.5	533.4	52.03	1011.9
1962-1963	8.01	148	671.5	5.99	.0723	433	58	198.9	439.9	52.21	1154.1
1963-1964	7.85	158	714.2	6.37	.0587	465	63	193.3	467.0	35.25	897.8
1964-1965	8.86	149	356.7	6.25	.0148	238	20	193.8	235.0	15.95	1240.1
1965-1966	7.94	134	183.0	5.78	.0129	122	20	197.1	116.7	5.90	666.4
1966-1967	8.52	133	136.2	5.81	.0594	162	22	194.6	101.7	13.08	1304.0
1967-1968	7.62	158	188.3	5.60	.0245	178	20	199.7	120.9	11.19	1424.6
1968-1969	7.79	136	184.7	5.59	.0271	138	18	194.0	116.1	8.04	1030.4
1969-1970	7.62	161	197.3	5.80	.0413	194	38	200.9	128.3	17.53	648.1
1970-1971	7.32	156	203.8	6.16	.0581	178	44	196.9	136.0	21.41	871.9
1971-1972	7.70	147	186.7	6.06	.0277	154	26	197.8	122.2	9.16	840.6
1972-1973	6.13	169	269.9	5.72	.0523	247	44	190.4	167.5	5.72	1046.3
1973-1974	7.07	128	429.4	5.86	.0548	317	52	210.4	194.3	30.48	744.2
1974-1975	8.32	163	379.0	5.34	.0303	247	23	197.2	188.6	16.50	1386.6
1975-1976	7.72	138	190.8	5.70	.0168	135	16	203.3	108.5	13.52	946.4
Average	7.75	152	452.6	5.99	.0814	380	59	196.3	292.1	34.52	1040.1
St. Dev.	0.62	11	236.5	0.31	.0664	207	38	5.0	156.9	24.36	282.9

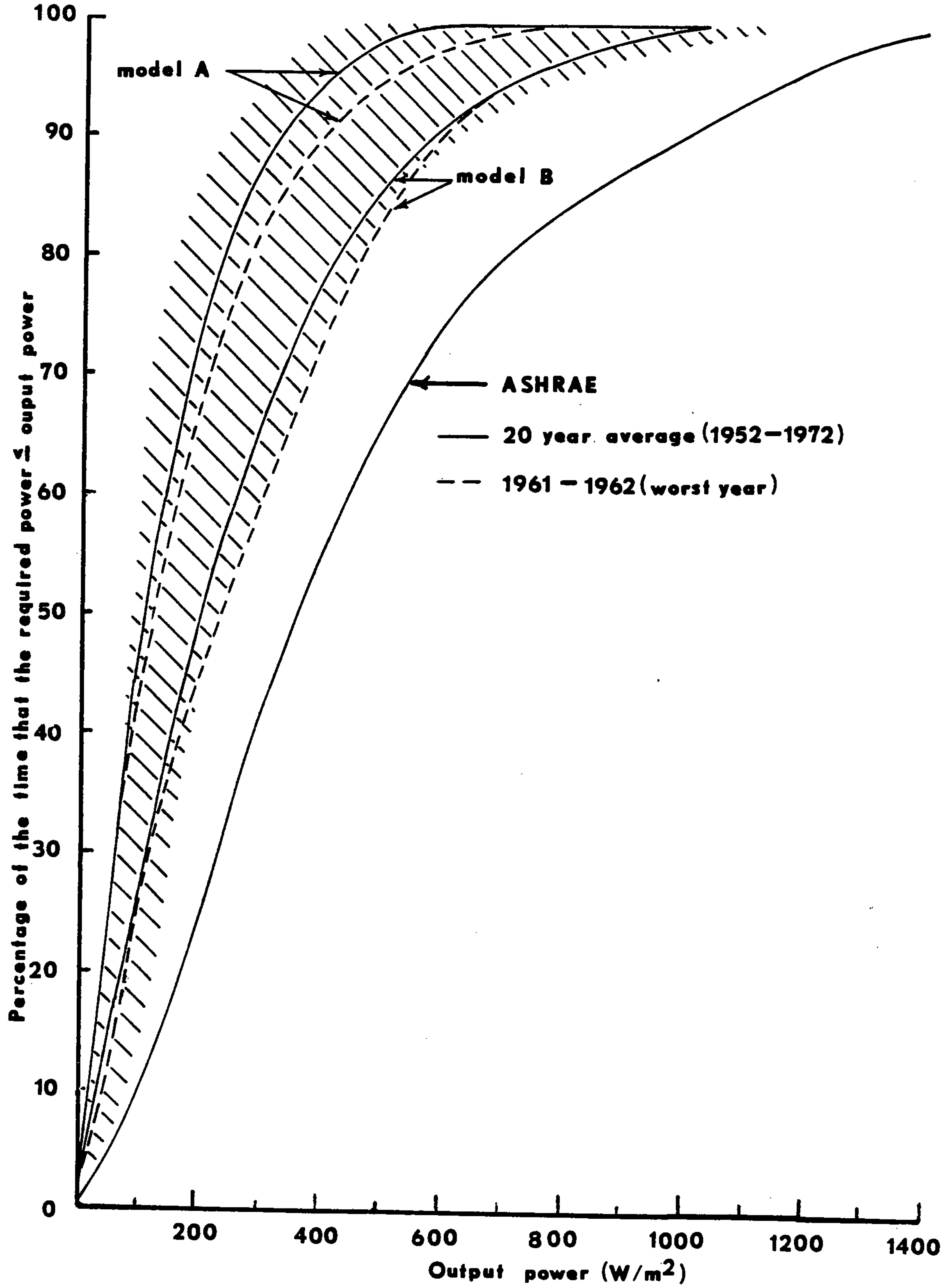


Figure 6. Power spectra for Cheyenne and various other cities (envelope)

VI. Thermal Design of the Heat Pipe System

Thermal Design of the heat pipe system is based on a calculation of the overall conductance between the ground and the top surface of the bridge deck. An electrical analogy is used and each heat pipe is represented using an electrical network.

The pipe wall resistance R_p is given by the cylindrical conduction relation

$$R_p = \frac{\ln (D_o/D_i)}{2\pi\ k\ \ L}$$

Internal thermal resistances due to evaporation and condensation of the ammonia are given as

$$R_e = \{ \pi D_1\ L_e\ h_e \}^{-1}$$

and

$$R_c = \{ \pi D_i\ L_c\ h_c \}^{-1}$$

where

$h_e \simeq 360$ Btu/hr ft^2 ^oF $\{2044\ w/m^2\ {}^oC\}^*$

and $h_c \simeq 1400$ Btu/hr ft^2 ^oF $\{7949\ w/m^2\ {}^oC\}^*$

The power losses in the system involve heat lost from the lower surface of the deck and in the piping between the evaporator and condenser. It is assumed that adequate insulation will be provided so that $\dot{Q}_1 \simeq 0$. Thermal resistance of the deck is a function of the diameter and spacing of the embedded pipes, depth of cover and thermal conductivity of the concrete, as given by

$$\dot{Q}_d = S\ L_c\ k\ (T_c - T_s) = U_d\ (T_c - T_s)$$

where

S = Shape factor

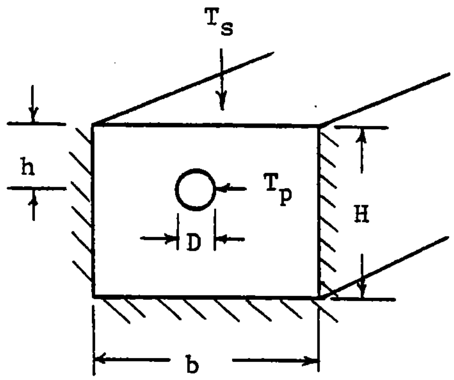
L_c = Length of condenser

k = Thermal conductivity of deck

U_d = Deck conductance

Calculation of the shape factor involves the Jacobian elliptic functions sn, cn and dn and the equation

$$S = \frac{2\pi}{\ln\left[\dfrac{\frac{4h}{D}\ \mathrm{sn}\left(\frac{Kh}{H}\right)}{\left(\frac{Kh}{H}\right)\mathrm{cn}\left(\frac{Kh}{H}\right)\mathrm{dn}\left(\frac{Kh}{H}\right)}\right]}$$



$\frac{K'}{K} = b$ and $D < 2b,\ h$

The resistance between the ground and evaporator is given by the relation

$$R = \frac{\ln\left(\dfrac{4\ L_c}{D_c}\right)}{2\pi L_e\ K}$$

VII. Implementation of the Design

Analysis of a manifolded heat pipe design for use on the Spring Creek bridge led to a condenser design shown in Figure 7. The use of 2 long fingers and two short fingers is based on a trade-off on the length of evaporator pipe which could be installed at the Spring Creek site. The site is underlain by the Casper sand stone, the main source of water for the City of Laramie and it was agreed that the heat pipes would not penetrate this formation. Therefore, the evaporator pipes were limited to lengths of 31m (100 feet).

Each corner of the bridge deck was heated by 15 evaporators arranged on a 3m (10 feet) spacing. (As shown in Figure 8) This design allowed for manufacture of the manifolded condensers, header pipes and evaporator in a fabrication shop. The manifolds were shipped to the site and installed on the top rebar mat. Subsequently the evaporators were placed in the holes and backfilled with grout, following which the header pipes were installed. All the steel utilized for heat pipe fabrication was coated with fusion bonded epoxy and the headers running from the evaporators to the condenser manifolds were insulated with three inches of urethane foam which was coated with butyl paint. Prior to insulating the header pipes each heat pipe was evacuated, leak tested, and charged with 5 pounds of ammonia.

A series of photographs showing the construction sequence is provided in Figure 9.

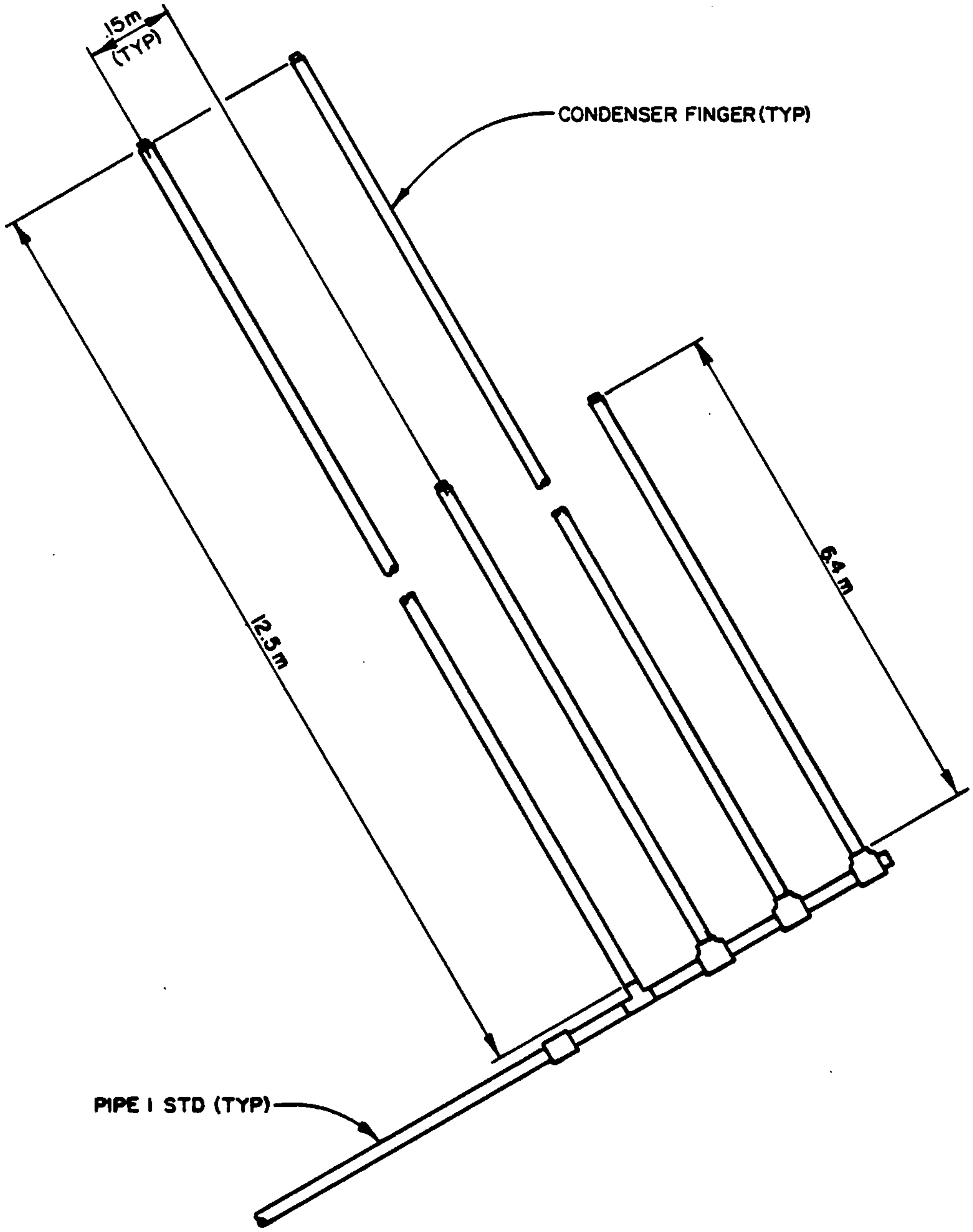


Figure 7. Condenser Design

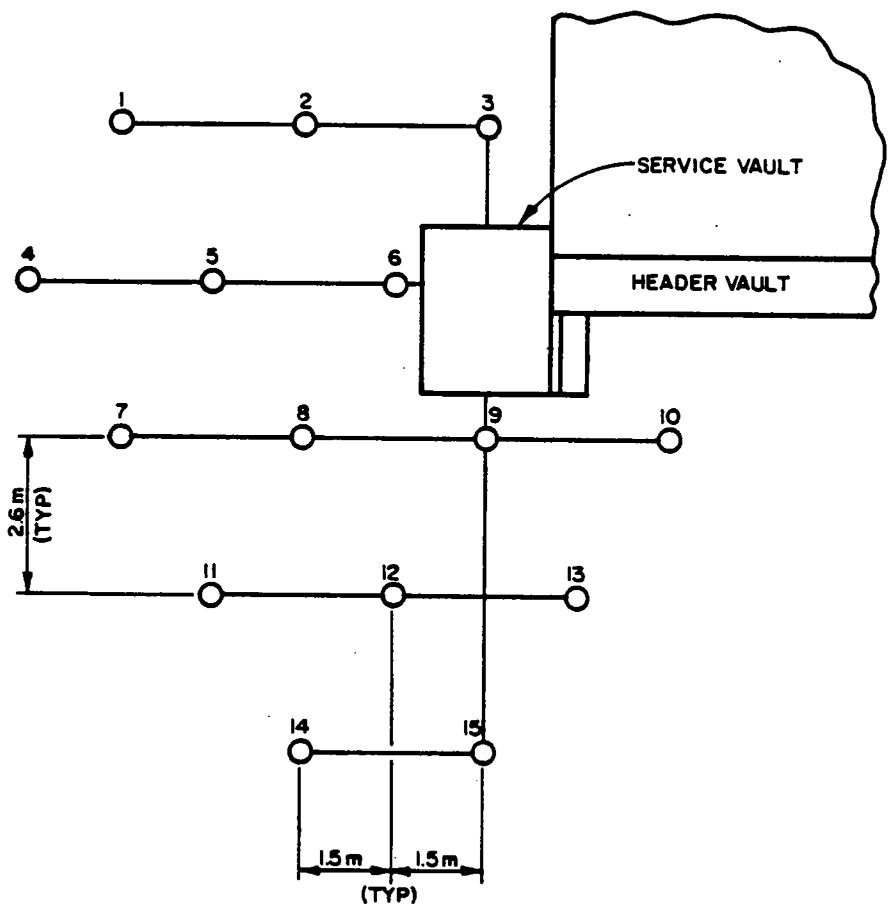
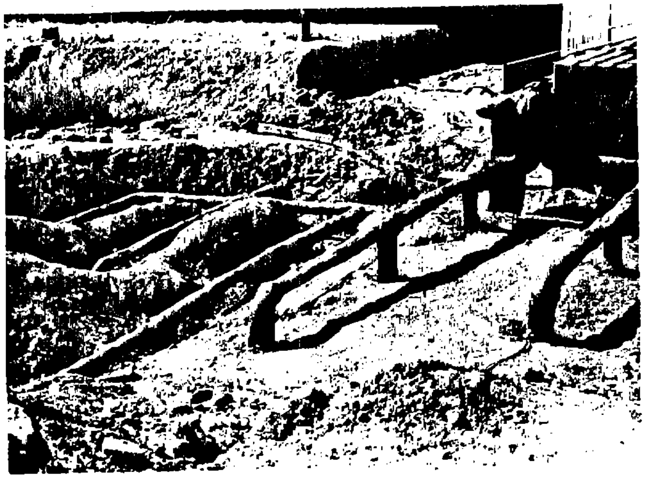
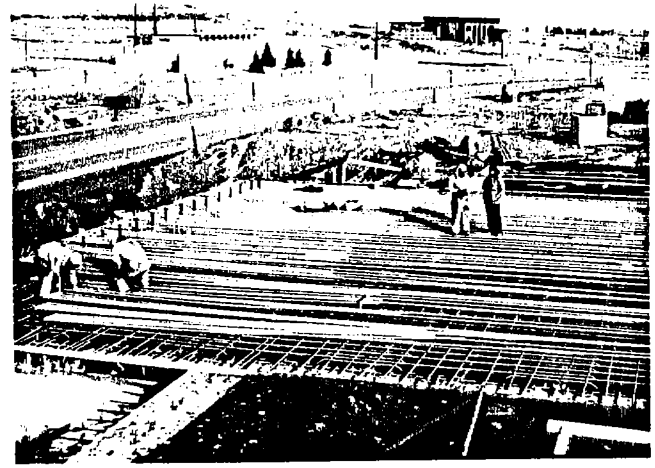


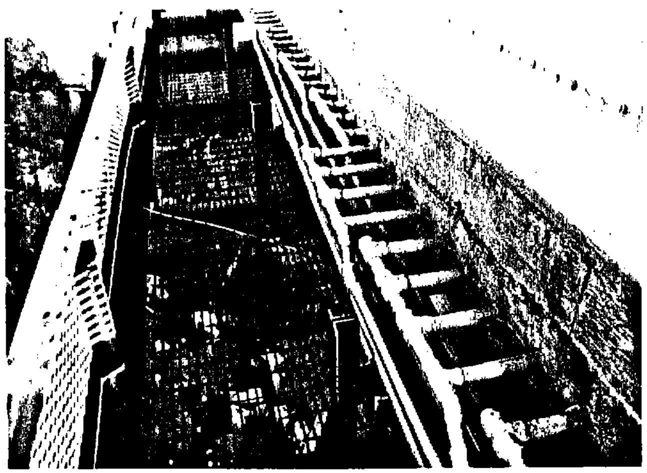
Figure 8. Hole Arrangement



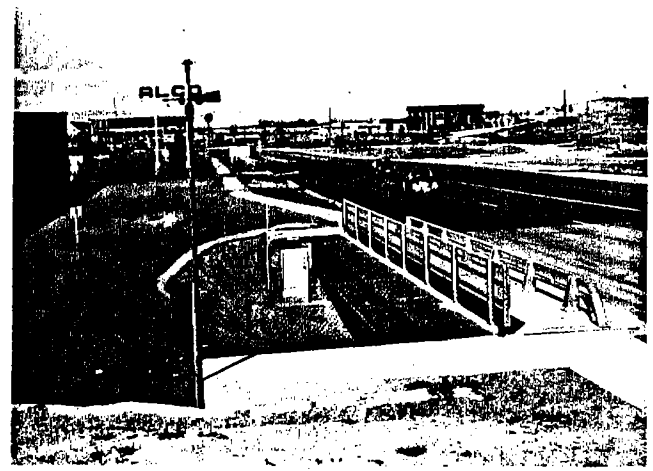
Header pipes running from evaporators to service vault



Placement of condenser manifolds on deck



Manifolds and headers in header vault



Completed structure

Figure 9. Construction sequence on Spring Creek Bridge

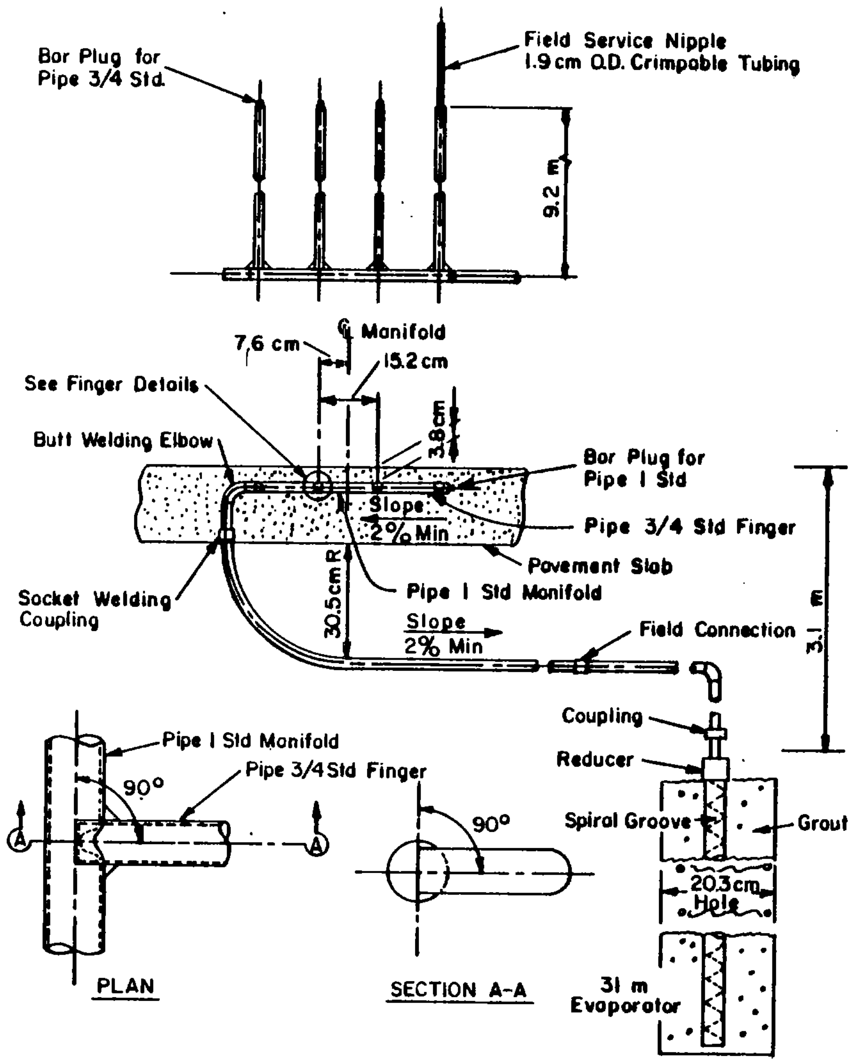


FIGURE 10. TYPICAL CHEYENNE HEAT PIPE DETAILS

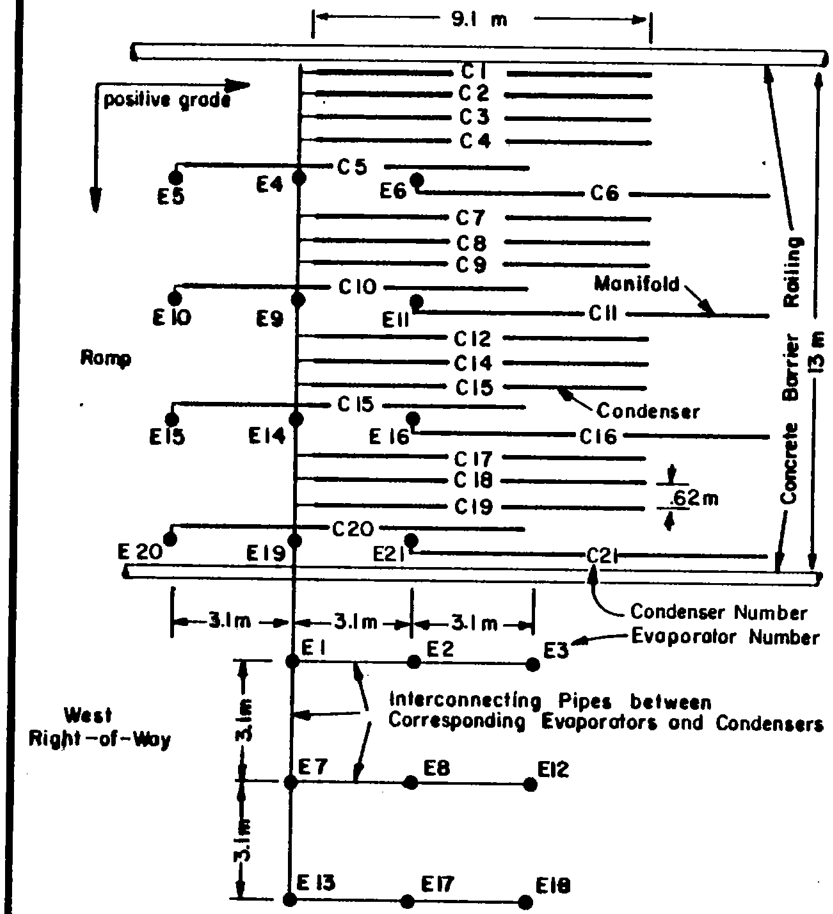


FIGURE 11. TYPICAL RAMP MODULE SHOWING EVAPORATOR, CONDENSER AND INTERCONNECTING PIPE LAYOUT.

VIII. Instrumentation

In order to quantify the thermal performance of the heat pipe system the site is heavily instrumented. Several evaporator pipes were instrumented with thermistors located on ten foot intervals. A remote ground site and the approach roadway were instrumented with thermistors located on various spacings down to depths of 31m (100 feet). In addition, 6 sites on the deck were instrumented with thermistors.

Weather data including wind speed, wind direction, air temperature, humidity, barometric pressure, precipitation, and incoming solar radiation will also be monitored. The data will be recorded on 10 minute intervals utilizing a computer based data acquisition system.

With completion of construction scheduled for May 1981, the first winter of data acquisition will be 1981-1982.

IX. Cheyenne, I-180 Railroad Overpass

The Cheyenne I-180 Railroad Overpass will incorporate heat pipes on two northern ramps which have a 7% grade. The ramps are not elevated structures but rather are filled. A manifolded heat pipe design has been developed using the results of the previous two studies. A typical heat pipe is shown in Figure 10. Since there is insufficient earth to couple to directly under the pavement evaporators will be placed in the berm area between the two ramps. A modular arrangement which heats a 9.2m (30 foot) length of the 13m (42 foot) wide pavement has been developed and is depicted in Figure 11.

X. Summary and Conclusions

During the past five years the Wyoming Highway Department has conducted three construction projects which incorporated gravity operated heat pipes. These projects have demonstrated that heat pipes systems are effective in eliminating preferential icing in spite of the fact that they are underdesigned and do not melt all of the snow that falls as rapidly as it falls.

It has been demonstrated that heat pipe systems can be incorporated in a normal construction schedule without undue impact on the contractor's normal proceedure. Although a variety of improvements in the design and manufacture of heat pipe systems have been achieved the cost per square foot of heated surface remains high principally due to the cost of manufacturing and placing the evaporator pipes. Approximately 30% of the system cost is involved with drilling the holes and grouting the evaporator pipes.

The high cost of the system will limit its applicability to especially hazardous locations where a total cost/benefit analysis indicates that such a system is warranted. In view of the fact that commercial power is not used and the system is virtually maintenance free the capital cost can be amortized over the design life of the installation.

Additional techniques for decreasing the system cost will be explored in the near future. These include using the condensers as structural members to replace reinforcing steel and alternate techniques for coupling the condensers to the earth energy.

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THE APPLICATION OF INDUCED POLARIZATION IN HIGHWAY PLANNING, LOCATION, AND DESIGN

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ABSTRACT

Small deposits of iron pyrite in minute crystal form and dispersed extensively in bedding planes are found in the Anakeesta shale formation in the Appalachian Mountains. These deposits can cause serious environmental problems when exposed in highway cuts and fills. Problems encountered with the disturbance of low-grade iron pyrite deposits include severe damage to aquatic life in streams, rapid deterioration of pipe culverts, severe staining, and the growth of undesirable algea. The cure is costly, time consuming, and less than foolproof, and mitigation is rarely acceptable. Avoidance is best, but damages can be alleviated with appropriate design.

A number of remote sensing and other techniques were evaluated in an effort to find a means of economically, accurately, and expeditiously locating low-grade deposits and establishing their vertical and longitudinal boundaries. Good results were achieved using the induced polarization process along a proposed highway corridor from Tellico Plains, Tennessee, to Robbinsville, North Carolina.

The various techniques and methods considered included airborne and surface-mode magnetometers, which are very good for locating large, high-grade, mineable iron ore deposits magnetic in nature but are not applicable to locating small, low-grade pyrite deposits; nor are they adaptable for use in very rough, mountainous terrain. The electromagnetic processes considered were not very good for locating the low concentrations of sulfide minerals. Imaging techniques are good for determining soil types and plotting formations but are virtually useless in pinpointing small, specific underground mineral deposits.

This paper discusses the induced polarization process, its application to detecting iron pyrite deposits along proposed highway alignments, and its application to other secondary geological explorations for highway location and design studies, such as the location of faults, determining the depth of various geological formations, and formation description. This paper also reviews reasons for locating these low-grade iron pyrite deposits, potential environmental problems that these deposits may cause, some solutions to the problems, some mitigation procedures, and means of avoiding the damaging deposits.

INTRODUCTION

Low concentrations of sulfide minerals are serious cause for concern when they occur in environmentally sensitive areas and happen to fall in the path of proposed highway construction. The Appalachian Mountains of East Tennessee and Western North Carolina are noted for deposits of both rich and low-grade ores of iron pyrite. The Anakeesta shale formation, which occurs extensively in the Tellico Mountains between Tellico Plains, Tennessee, and Robbinsville, North Carolina, is the geologic formation containing the low-grade deposits of minute crystals of iron pyrite or iron sulfide (FeS_2) which are too small to detect with the natural eye. When exposed to the elements and permitted to weather, the crystals oxidize quickly to a weak solution of sulfuric acid (H_2SO_4) and probably iron oxide (Fe_3O_4), both damaging to acquatic life. In an area experiencing acid rain and producing tanic acid as the nature of its vegetation, the addition of even a weak solution of sulfuric acid can upset the balance. Toss in a natural trout stream, one or two avid trout fishermen, Trout Unlimited, disappearing inverts of asphalt coated corrigated metal pipe, some upset environmentalists, and a judge, and iron pyrite can become a real headache.

Acid drainage and its adverse environmental impact is very familiar to residents of Appalachia, especially to those of the coal mining regions and the Ocoee River in the Copper Basin which is in the vicinity of this project. The environmental problem may have existed all along with highway construction; but if the environment was a problem, the problem must have gone unnoticed, or the needs could have been so great that they offset the adverse impact. Also, most of the highways are old enough that any problems which may have occurred could have healed over and disappeared. The seriousness of the problem apparently culminated with the continuation of the construction of the proposed forest highway connecting Tellico Plains to Robbinsville.

Once sulfide minerals are disturbed by highway construction and leaching begins, remedial measures can be very expensive. Remedial measures incorporated in the design process are much less costly. Detection of the deposits and the utilization of design procedures incorporating a combination of avoiding the deposits and remedial measures may be the most economical and expedient approach and will help the highway industry maintain some element of creditability.

Detection or location of the sulfide mineral deposits prior to the final design and possible disturbance during construction is crucial to the adequate protection of the environment. To be effective and acceptable, detection must be both accurate and as inexpensive as possible. Any detection method used in the Appalachian region also must be adaptable to rough mountainous terrain covered with dense vegetation and to some fairly complicated geology. With these criteria in mind, the induced polarization and resistivity detection method was tested for the first time in highway work in a successful effort to locate the low-grade iron pyrite deposits which had caused pollution problems on previous projects.

ENVIRONMENTAL PROBLEMS ASSOCIATED WITH SULFIDE MINERALS

Acid mine drainage is a problem familiar to many as is acid rain, but acid drainage from highways has not been considered a widespread problem, although problems have been experienced at some locations, especially in the Appalachian Mountain range. The problems are related to essentially the same basic source--decomposition of compounds containing sulfur of which iron pyrite is about the number one actor in the Appalachian Mountains. The problems occur generally when deposits of low concentrations of widely disseminated, minute crystals of iron pyrite contained in somewhat porous rock are disturbed and left exposed to weathering conditions as they are in highway cuts and embankments not protected by blankets of impermeable materials. The telltale signatures of the presence of the iron pyrite are the rust-colored stains on the cut slopes and on the exposed broken rocks in fills, the deterioration of the inverts of pipe culverts, the deposition of iron oxide in streams or ditches, the dying of aquatic plants and animals, and the appearance of algae which require the acid-iron rich environment for growth. In some locations such as the Copper Basin, galvanizing on steel guard rail has a life span of only two to five years, probably as a result of acid rain but sometimes apparently as a result of splashing.

The need to locate and deal with sulfide minerals effectively is certainly a matter of concern for highway planners and designers. Although environmental problems have had to be considered only in recent years, they now must be handled effectively. Geologists are the only group which can effectively deal with this particular problem. They should be called on beginning with corridor and route selection and should be kept active until the highway facility is operational. The need to know if problem mineral deposits exist begins with the corridor and route selection process. Geologists should be called on to make such preliminary determinations during the conceptual stage. If deposits are obvious, they of course can be avoided. It is the elusive deposits that are bothersome, but these usually can be located more efficiently in the more difficult terrains during the preliminary design stages after preliminary center lines and profiles have been developed. If communications are good between planners, environmental sections, location sections, and geologists, suspect areas can be delineated in the early stages and can receive proper attention as the proposed route is developed more fully.

Once iron pyrite bearing formations have been exposed on highway cut sections and in embankments, the process which breaks down the pyrite into iron oxide and sulfuric acid is difficult and expensive to control. Usually a combination of remedial measures are necessary. The increased acidity of streams should be neutralized. For best trout survival and reproduction, particularly in a natural habitat, the pH level must be kept between about 6.5 and 8.5. (A pH level of 5 can be tolerated to some extent by the trout of the Appalachian range where streams tend to be on the acidic side.) At the lower tolerable pH ranges and below, expensive treatment facilities may be required to neutralize the acid, and monitoring stations will have to be maintained. Sodium hydroxide (NaOH) or lime, usually quick lime (CaCO) or hydrated lime ($Ca(OH)_2$), may be used as the neutralizing agent.[1] When these compounds are dissolved in water, heat is generated. This may necessitate temperature monitoring also, especially in trout streams of marginal quality. For trout reproduction, stream temperatures must drop into the range of 40 degrees to 50 degrees F for a few months, and for continued healthy survival, stream temperatures must not exceed about 70 degrees F.

The same chemical compounds used to neutralize the acid will cause the precipitation of the iron oxide as the pH level begins to increase above about 6. Precipitation is most unattractive in the stream bed. In order to control appearance and to remove the solids at the point of neutralization as required by the Environmental Protection Agency, the water may have to be passed through a retention pond capable of maintaining a detention time approaching about 30 minutes. In a situation like this, location of the treatment facility is critical, especially if a retention pond is necessary. Another matter which must be considered is the possible requirement of aeration to provide free oxygen in the outfall, to provide for mixing, and to provide for complete oxidation.

In order to cut down on the leaching from the cuts and fills, massive doses of lime may be applied to the slopes. If the cut slopes are near vertical, little else can be done except to direct the drainage to the treatment facilities. Fill slopes may present the most serious leaching problems if the fills were constructed with pyritic rock without choking the voids and sealing the slopes with an impervious material. This condition will permit rain water to run freely through the fill and to dissolve the pyrite in massive quantities. A fair remedial measure may be to envelope the embankment section in a thick, impervious layer of clay.

The deterioration of pipe inverts caused by acid drainage also requires complicated remedial measures. About the only solution is to reline the deteriorating pipe culverts with vitrified clay or plastic pipe. This very effectively reduces the original pipe size which in turn may cause ponding at the inlet end and may result in overflow around the culvert and through the fill, thus dissolving iron pyrite in the embankment material and further defeating its purpose of containing acid water. If ponding occurs, protection must be provided against seepage through the fill. Inlet structures must be constructed carefully to direct all drainage into the vitrified or plastic liner. The outlet end may require an anchoring device and a paved ditch if it emerges up in the side of the fill.

Remedial measures always seem to leave something to be desired, usually are expensive, and usually require constant maintenance and monitoring. A better approach is to try to avoid the pyrite deposits which can cause problems or to incorporate protective measures into the design. In many cases, troublesome deposits, if adequately located, can be avoided by slight line shifts or by changes in the profile grade line to avoid cutting into the deposit. It may not always be feasible or practical to avoid the deposits. Then economics and design problems may necessitate other considerations such as designing the cut slopes so they will support a thick blanket of impervious clay at least over the pyrite zone. Rock containing the pyrite may have to be disposed of either in embankments or waste areas which can be enveloped on the top, bottom, ends, and sides with an impervious clay layer. Usually if handled in the design phase, treatment facilities can be avoided, and the cost will be considerably less.

INDUCED POLARIZATION METHOD OF USE AND EQUIPMENT

Surface and airborne geophysical survey methods were considered for this project. Because of the rough, steep terrain and the dense vegetation with trees up to and probably exceeding 100 feet in height, airborne techniques with towed receivers seemed impractical. It was decided to use the surface induced polarization method which was developed in the early 1950's and had been used extensively for the exploration of low grade, 1% to 2%, sulphide mineral deposits.

Polarization as used in this context means to block or build up and refers to the phenomenon which is induced when current is applied to the moist soil just below the ground surface and is blocked at the interface of a metallic surface by an opposing current (see Figure 1). The mode of conduction of the current applied to the moist soil is ionic in nature. This applied current flows through the solutions which fill the pore spaces of the rocks. When this current reaches the face of the pyritic mineral, the ionic current is converted to electronic current or is blocked until it builds up sufficiently to give up or receive the electrons contained in the crystal lattice in the metallic particles of the pyritic formation. The blocking action, or the resistance to the flow of current, increases with the time that the induced direct current is allowed to flow through the conducting medium which results in the build up of ions at the metallic interface. Eventually, an excess of ions build up at the interface which appreciably reduces the amount of current flow through the metallic particles. The Coulomb or electromotive forces between the charged ions at the polarization zone force the ions to return to their normal position when the induced flow of direct current is cut off. This reversal of charge or transient flow of charged particles creates a small current flow which can be measured as a voltage at the ground surface as a decaying potential difference. This "pulse transient" induced polarization method of detecting polarization can be achieved either by applying the current for a period of time and measuring the decay once after the cut off or by averaging several decays when applying the current in alternate directions in a series of pulses. These measurements are taken in the "time domain" with the geophysicists looking for areas where current flow is maintained for a short time after the applied current is terminated. By reversing or alternating the flow of current repeatedly at frequencies of from DC to a few cycles per second before the polarization occurs, the effective resistivity of the system will change as the frequency of the switching occurs permitting measurements to be made in the "frequency domain." The apparent resistivity value is measured as the frequency of the applied current is altered. The presence of metallic minerals is indicated by changes in the apparent resistivity. A correlation can be developed between the type of material present below the ground surface and recorded data,[2,3,4,5] including measured resistivities, frequencies, and applied voltage.

The resistivity of the material encountered or its ability to conduct electrical current is the key to the interpretative process. Since graphite, for example, is an excellent conductor and seems, as commonly expressed, to "eat" the current, graphitic shales are easily detected. Some graphite bearing shale formations were encountered on the project. Decreased or low resistivity and increased polarization were considered to be indicative of contact with a pyrite deposit.

The induced polarization technique functions in accordance with Ohm's Law, i.e., voltage equals current times resistance ($V = IR$). Current is passed into the ground by the transmitter at specific preset frequencies. The receiver measures an output voltage. Since the earth is not really a true resistor, inductance and capacitance are present, and impedance is referred to as the apparent resistivity. Capacitance and inductance are frequency dependent. Voltage measurements are actually taken at the higher

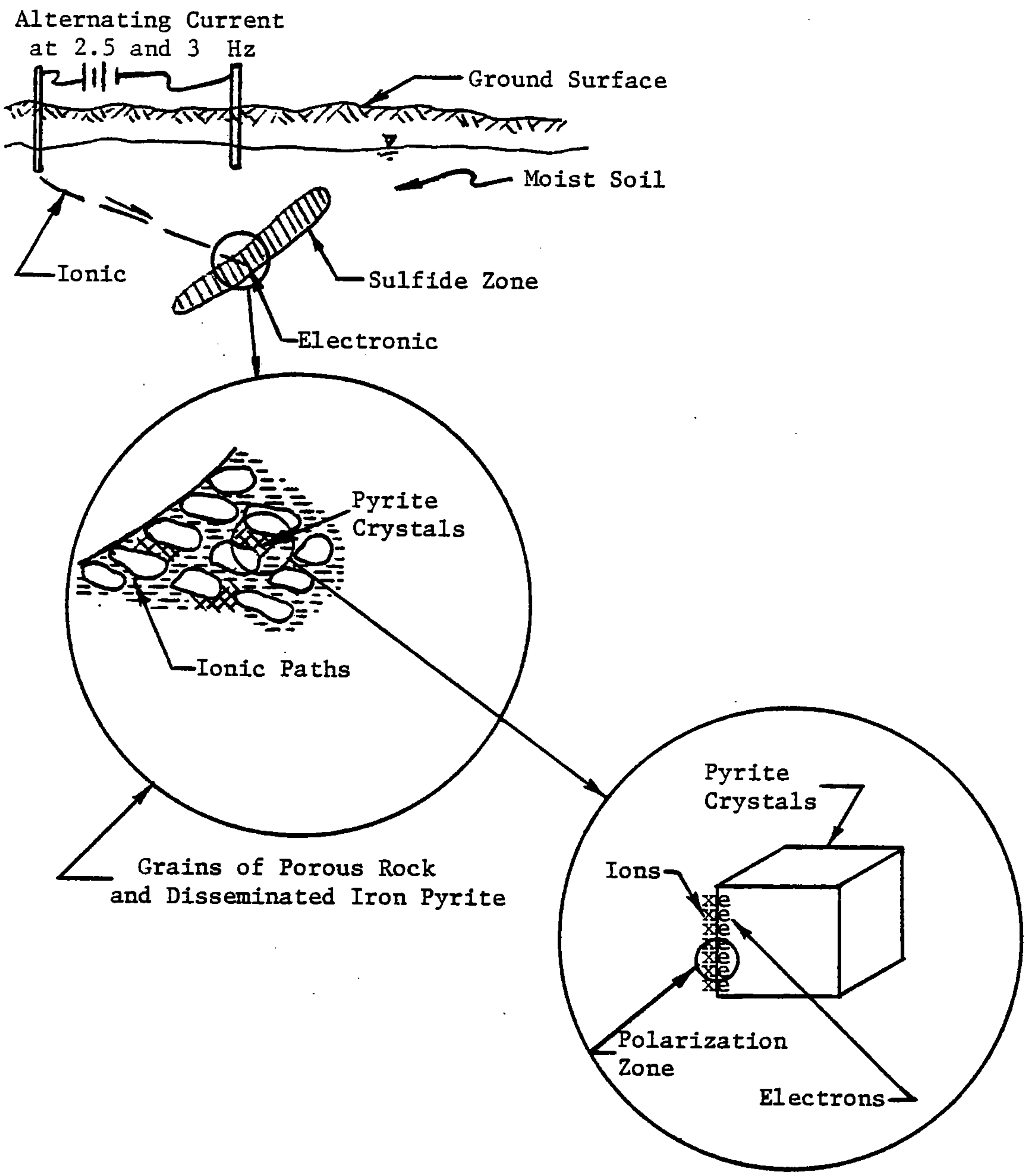

Figure 1. Diagramatic sketch of the induced polarization process.

frequency; then the frequency is lowered to obtain a frequency effect measurement which is related to the polarization of the subsurfaces.

Figure 2 illustrates the configuration of the equipment and the spacing of the dipole-dipole array used in the reconnaissance survey made along the center line of the proposed highway. The equipment used to conduct the survey was divided into two categories--one for the transmitting electrodes and one for the receiving electrodes. A number of hollow, steel stakes about 30 inches long were used as the electrodes and were equipped with quick connections, usually alligator clips. The electrode connections were formed using heavy-guage, well-insulated copper wire. Small tools consisted of such things as a small sledge hammer, an ax, a small supply of water for occasional use with the electrodes, an extra supply of wire, a tool kit for adjusting the equipment, walkie-talkies, and a few others.

The transmitting circuit contained the transmitter (see Figure 3) which was equipped for varying the frequency of the applied current, and for maintaining a constant voltage through the electrode to the moist soil. The transmitter was very sensitive to moisture and condensation caused by frequent showers, very high humidity of about 90 percent, and hot days and cool nights or extreme changes in temperature. The transmitter, weighing about 40 pounds, could not be left in the field over night because of the moisture problem and because of wild hogs. The moisture problem was eventually solved with a desiccant such as silica gel. The geology of the area, rough terrain, soil conditions, soil moisture, the high currents required, and the resistance experienced prevented the use of a battery pack adaptable to the transmitter, and the current had to be supplied by a gasoline-operated generator which weighed about 50 pounds and required about two gallons of gas for a ten-hour operating period.

The receiving dipole consisted of the receiver (see Figure 4) for measuring the voltage, two copper sulfate pots used for the dipole electrodes, and the wire. The soil beneath the copper sulfate pot had to be moist, and the copper sulfate level was checked daily.

A four-person crew was probably optimum with everyone taking turns carrying the equipment. However, a larger crew seemed desirable at the end of a ten-hour day when the vehicle was over five miles away, over a ridge, and about 1,000 feet higher or lower in elevation. The day always began with a rough ride, usually a long hike into the line, and a careful calibration of the receiver to the transmitter while the first station setup was underway.

TECHNOLOGY TRANSFER OF INDUCED POLARIZATION TO THE HIGHWAY FIELD

The problems with water quality encountered when highway cuts penetrated into rock strata containing widely disseminated minute crystals of iron pyrite was cause for some serious concern. With many miles of highways still needed in the Appalachian region and with some highways nearing the final design stage, an accurate, economical, and expedient means of locating the worrisome, low-grade sulfide mineral deposits was very much in demand. The use of core drilling rigs is very expensive, disruptive to a sensitive environment, and food for some watchful eyes.

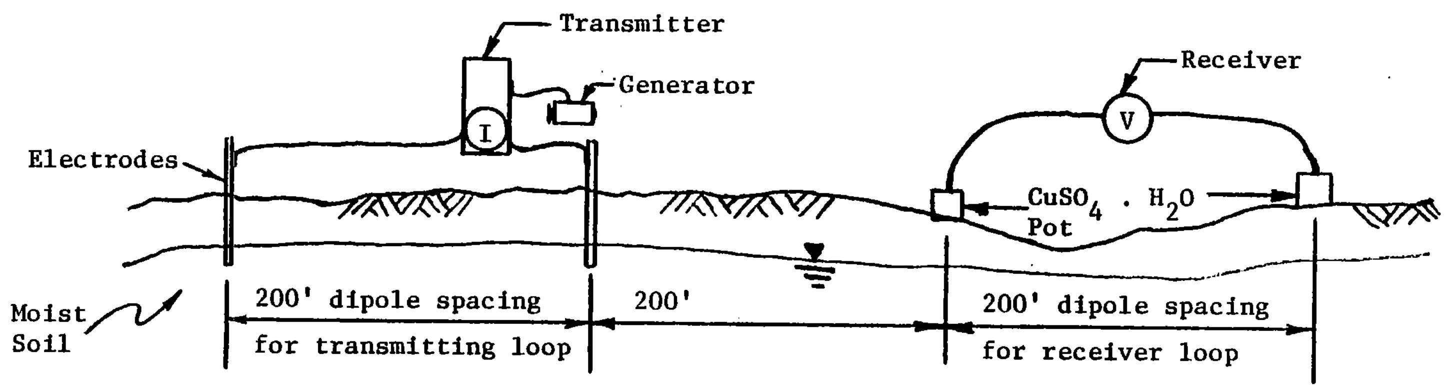

Figure 2. Electrode configuration with dipole-dipole array. The constant spacing of 200 feet was used on the reconnaissance survey.

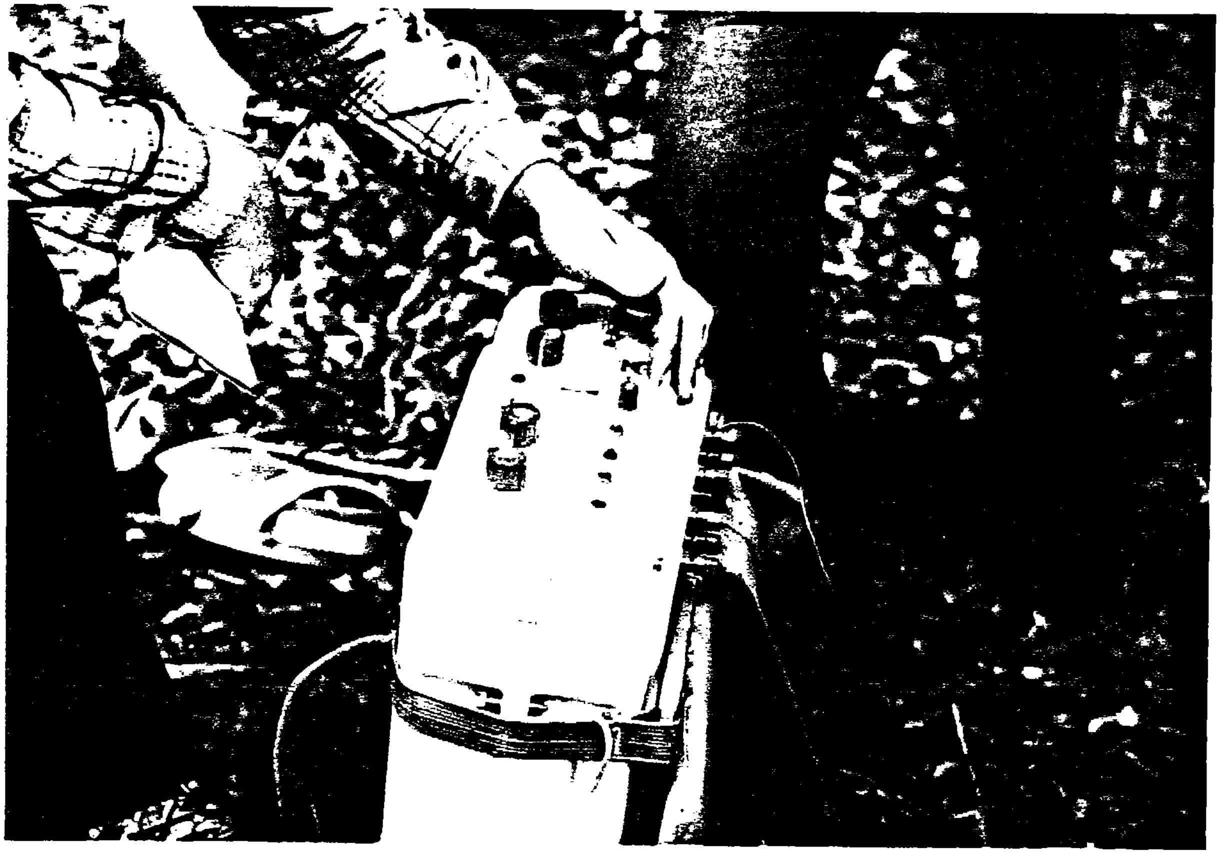

Figure 3. Induced polarization transmitter with frequency control and alternating input current regulator.

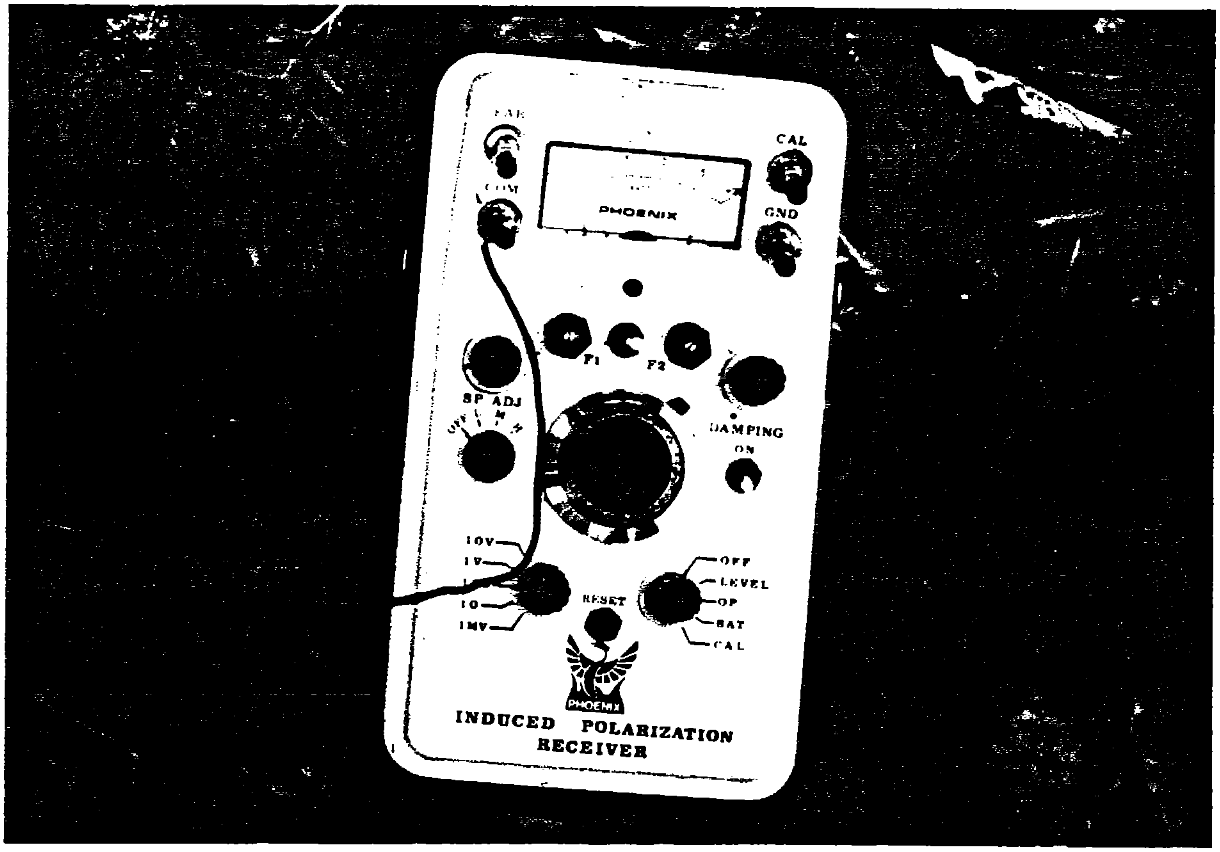

Figure 4. Induced polarization receiver for measuring output voltage for calculating the resistivities ranging from 20 ohm meters to greater than 80,000 ohm meters at frequencies of 2.5 and 3 Hz.

Many miles of the Tellico-Robbinsville project were virtually inaccessible to drill rigs. At accessible points, the drill rigs would have had to be moved in on skids with tracked vehicles or cables or set with helicopters. Water for the drills would have had to be brought in by tankers or pumped in for as much as two miles up 45 percent grades. Skidding would have required a great deal of clearing, and some clearing would have been required even if drills were set by helicopter. Every possible cut section would have had to be cored, and even then some deposits could have been missed. With these problems, remote sensing seemed to be a possibility, and a great deal of consideration was given to the various types available.

The photographic and nonphotographic imaging techniques of remote sensing can be used in mineral exploration and detection, but these methods are keyed entirely to interpretive methods that rely on surface indicators. These techniques also must be coordinated with seasons due to the dense foliage, although vegetation types are sometimes indicators of soil condition and minerals present. Some types of remote imaging techniques are good for determining soil types and plotting geologic formations. However, anything that lies below the surface does not lend itself very well to detection by these methods.

The electrical and magnetic geophysical exploration and prospecting methods were considered next. Almost all of these techniques can be used in the moveable surface mode or airborne mode. These methods are used extensively in the search for oil and minerals, and the electromagnetic techniques are known to be very good for locating iron pyrite. Much consideration was given to the use of aircraft for the search for the scattered, low-grade, iron pyrite deposits plaguing the completion of the forest road across the Tellico Mountains just south of the Great Smoky Mountains. Either airplanes or helicopters can be used with these methods, but they usually carry a towed bird trailing about 200 to 500 feet behind the aircraft and containing all or part of the instruments. The aircraft must also fly from 200 to 400 feet above the ground. Anyone familiar with the terrain, weather conditions, and forest types common to this area know why the airborne systems were set aside in favor of the moveable surface mode. Due to the rough, steep terrain, occasionally severe adverse air currents, occasional quick thunder storms, and the high, dense vegetation, the moveable surface mode was finally considered the most practicable.

With the decision made to remain on the surface, the next major decision concerned the best method to use. The choices seemed to be either electromagnetic methods or the induced polarization method, both of which are good for finding sulfide mineral deposits and for defining geologic features. However, electromagnetic methods are limited to locating massive sulphide deposits of about 15 percent by volume or above. The induced polarization technique is sensitive enough to locate deposits of approximately one percent of the highly disseminated minute crystals which cause the most serious problems in the Appalachian Mountains. The induced polarization technique has a number of other favorable aspects also. However, after helping carry the equipment around over 17 miles of rough mountains between 3,000 and 5,000 feet in elevation, some doubts about the wisdom of the decision did emerge. The favorable final results, however, made the effort worthwhile.

No evidence could be found of the use of induced polarization in the highway field prior to this undertaking. However, for the well-identified and defined objective, an induced polarization survey seemed to be the most practical approach. Since induced polarization was so widely recognized and acceptable in geophysical exploration and was known to be capable of delineating anomalies indicative of low-grade iron pyrite deposits, the exercise seemed to be one of technology transfer or a demonstration rather than research. Therefore, a very small demonstration project was set up as a test application along a two-mile section of center line for which core samples had been taken and geological maps and profiles had been prepared by very capable geologists. A reconnaissance line was run using an electrode configuration with 200-foot spacing and a dipole-dipole array (see Figure 2). At locations where anomalies indicative of iron pyrite were detected, a detailed survey was made using a minimum 25-foot spacing expanded on 25-foot intervals and a maximum 200-foot spacing. The survey was conducted using the induced polarization and resistivity equipment which could be moved from station to station along the ground surface. The results were excellent and compared very favorably with the analysis of the extracted cores. Some deposits of the sulfide minerals also were located on the induced polarization survey which were not located by the core drilling process. After the successful demonstration, the project was extended to include 17 more miles of survey.

One additional feature of the induced polarization method is its ability to indicate the lateral and vertical limits of the deposits. Detailing is the most expensive and time consuming part of the survey, but, because of the narrow transverse widths involved, the exact location and horizontal limits of the spots needing detail work can be defined on the reconnaissance survey; thus the amount of detail work needed can be limited. If a preliminary profile of the proposed line is available, then any deposits falling entirely within embankment sections can be eliminated from the full detailed survey.

Absolutely essential to the use of induced polarization techniques is the use of qualified personnel. The interpreter or geophysicist must be knowledgeable by education and experienced in the area of geophysical exploration and geology with a good comprehension of the area being surveyed and with a complete understanding of the induced polarization equipment being used. The problems encountered as a result of disturbing sulfide mineral deposits are very serious environmentally, and efforts to locate deposits accurately are not places to conduct a training program for a team of inexperienced key personnel. The survey team must be chosen on the basis of proven experience and ability.

INTERPRETATION AND RESULTS OBTAINED

The reconnaissance survey procedure used is described as a surface mode, moving set-up, induced polarization and resistivity survey using the frequency domain. The transmitting and receiving dipoles were spaced 200 feet apart to obtain only the N = 1 induced polarization and resistivity measurement (see Figure 5). Both sets of dipoles were moved continually along the survey line at 200 foot intervals which provided a measurement of apparent resistivity and polarizability within a half sphere with a radius of 100 feet or to a depth of 100 feet. When areas of decreased resistivity and increased polarizability were located, detail work was recommended or conducted to determine the location and depth to the top of the anomalous source. The detail survey employed 25-foot or 50-foot dipole settings with readings to

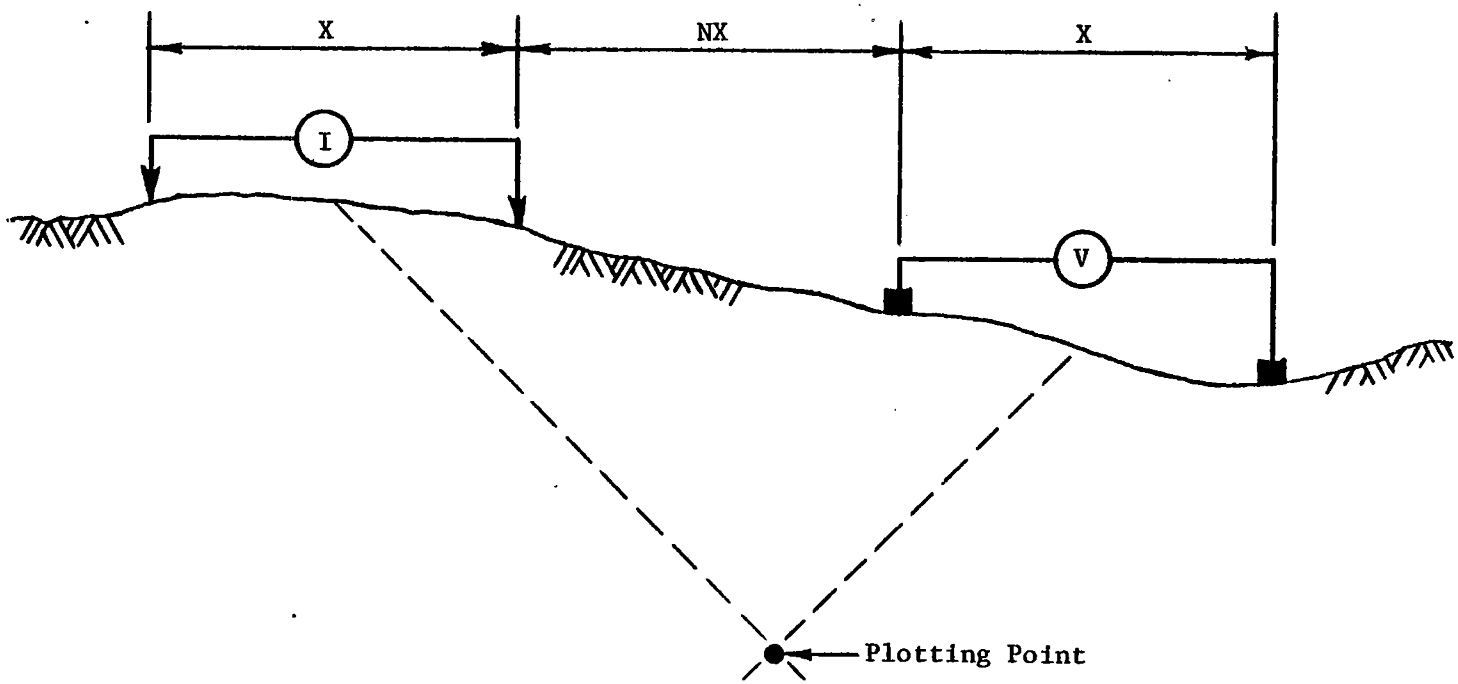

Figure 5. Electrode configuration with 200-foot dipole array. X = 200 feet and N = 1 for reconnaissance work. X = 25 feet or 50 feet and N = 1, 2, 3, and 4 for detail work.

N = 3 or N = 4 which provided for a more precise determination of depth and width of the sulfide mineral deposit. As the data were interpreted, profile maps were developed showing definite, probable, and possible induced polarization anomalies; these were indicated by bars as illustrated in Figure 6. The bars represent the surface projection of the anomalous zones as interpreted from the location of the transmitter and receiver electrodes when the anomalous values were measured.

Since the induced polarization measurement is essentially an averaging process, as are all potential methods, it is frequently difficult to exactly pinpoint the source of an anomaly. No anomaly can be located with more accuracy than the electrode interval length. Using 200-foot electrode intervals, the position of a narrow sulfide body can be determined only to lie between two stations 200 feet apart and 100 feet deep. In order to locate sources at some depth, larger electrode intervals must be used, with a corresponding increase in the uncertainties of location. Therefore, while the center of the indicated anomaly probably corresponds fairly well with the source, the length of the indicated anomaly along the line should not be taken to represent the exact edges of the anomalous material since the exact edges will lie somewhere between the dipole spacing used and the set ups which incorporate the first and last reading indicating an anomaly.

Metal factor (MF) anomalies, percent frequency effect (PFE) and resistivity are shown on detail data plots, but only resistivity and PFE anomalies are shown on reconnaissance plots. The PFE results indicate polarizable areas without taking into account the resistivity of the areas. The MF or "metallic conduction factor" is obtained by combining the PFE and the resistivity.

The PFE is a ratio of the apparent resistivity (ρ) at the low frequency (f_1) or switching cycle and the apparent resistivity at the higher frequency (f_2)[5].

$$\text{Apparent resistivity at low frequency at station a} = \left[\frac{\rho(f_1)}{2\pi}\right]_a$$

$$\text{Apparant resistivity at high frequency at station a} = \left[\frac{\rho(f_2)}{2\pi}\right]_a$$

$$\text{Apparent frequency effect} = (fe)_a = \left[\frac{\rho_a(f_1)}{\rho_a(f_2)} - 1\right]_a$$

The MF is obtained by combining the PFE and the resistivity.

$$MF = \frac{fe \times 100}{\rho(f_1)/2\pi} \times 1{,}000 = \frac{\left[\frac{\rho_a(f_1)}{\rho_a(f_2)} - 1\right] \times 100}{\rho_a(f_1)/2\pi} \times 1{,}000$$

$$= \left[\frac{1}{\rho_a(f_2)} - \frac{1}{\rho_a(f_1)}\right] \times 2\pi \times 10^5$$

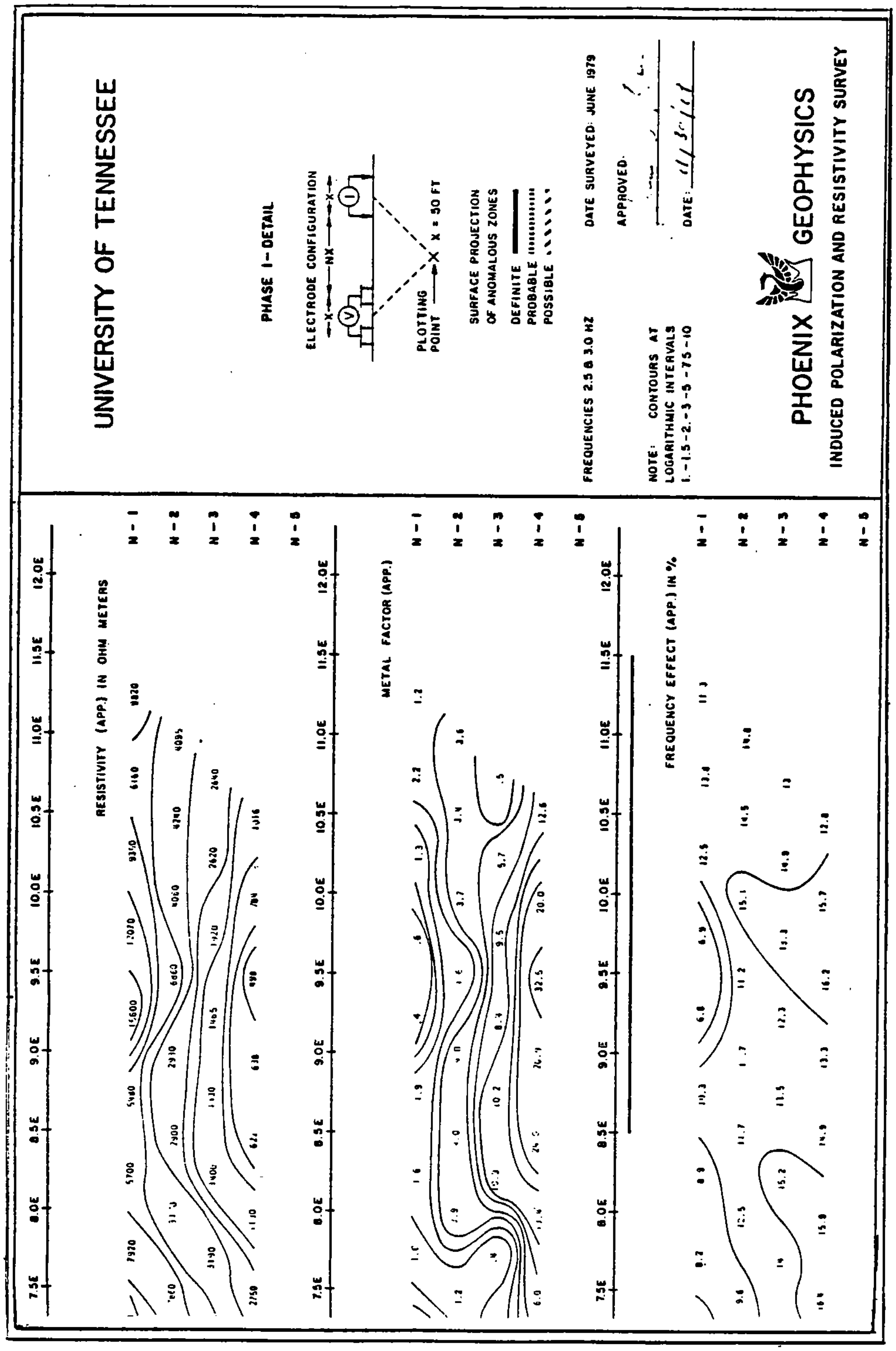

Figure 6. Typical output plot of induced polarization data showing the resistivity plot, percent frequency effect, and the metal factor plot, with symbols for indicating the location of pyrite deposits.

For an in-depth review of the theory and mathematics associated with the development and interpretation of the induced polarization and resistivity process, the reader is referred to references 6, 7 and 8.

A good conductor (low resistivity) that is strongly polarizable (high PFE) will give a well-defined or "definite" MF anomaly. Less well-defined MF anomalies are designated as probable or possible. The PFE and MF parameters are complementary. The relative importance of each type of information depends upon the particular geophysical environment encountered and the type of target expected. For example, a mineralized silicified zone will give a "definite" MF anomaly. Alternatively, an oxidized ore zone may only give a "weak" PFE anomaly but a "definite" MF anomaly.

The plot of the reconnaissance data usually shows only the apparent resistivity and PFE. The variation in resistivity is related to the variation in rock units; low resistivity indicates conductive material and/or highly permeable rock, such as carbonaceous slates or phyllites. High resistivity indicates tightly compacted rocks, such as quartzite or sandstones. A high PFE response of 10 or above suggests a very high concentration of sulfides or polarizable material; conversely a low PFE response indicates low or nonexistent sulfide content. A combination of low resistivity and high PFE can be assumed to represent areas where sulfide mineralization would readily dissolve if exposed in a road cut or fill.

On this project, a section of low resistivities was encountered in the first 2,000 feet, but the PFE response was not continuous, indicating some points of high sulfide content. The PFE response dropped dramatically at some locations, and the resistivity remained low, resulting in the conclusion that the rock unit in these areas were very permeable but contained very little sulfide mineralization. At several other locations, the apparent resistivity was moderately high, approximately 5,000 ohm meters, and the PFE was anomalous. These areas probably contained several narrow massive sulfide veins or disseminated sulfide mineralization zones across most of the 200-foot intervals. Only a detailed survey of these locations would validate the interpretation. The detailed survey conducted in the demonstration phase proved the importance of conducting detail work at locations where the reconnaissance survey indicated high sulfide content or shorter interval profiling in areas of anomalous response.

High PFE readings were received along the entire line but at varying depths, with some areas exhibiting highly polarizable material at or near the surface. Near the central portion of the line, a high PFE response was recorded at depths greater than 50 feet. Cuts in the areas of shallow response would undoubtedly expose the pyritic material, but probably no significant amount of pyrite would be encountered in the central portion.

There was a wide variation in apparent resistivity for the entire proposed highway center line, ranging from 20 ohm meters to greater than 80,000 ohm meters. The areas of low resistivity and high PFE were symbolized on the data plots by solid bars above and below the line respectively (see Figure 6). Where these bars coincide, sulfide mineralization occurs in conductive and/or extremely permeable rock and most likely would present problems if disturbed by cutting or if the material were used unprotected in embankments. The depth and width of these areas could not be determined with any precision from the reconnaissance data; detailed surveys in these areas would be required if they appeared to fall within cut sections with the profile grade penetrating the pyrite deposit. Several areas were located which exhibited a high sulfide content as symbolized by a solid bar below the line and indicated by moderate to high resistivity readings. The high resistivity indicates a less permeable, more dense rock unit which, if exposed to weathering, may not be subject to serious leaching or release of sulfides into solution.

OTHER POSSIBLE BENEFITS

The benefits derived from the use of induced polarization and resistivity surveys for locating troublesome deposits of sulfide minerals, hopefully, has been well-demonstrated above. Other benefits which may be of use in highway construction include the potential for locating areas of very sound rock, structurally and environmentally acceptable for use in roadway work for rock blankets, riprap, fills, or aggregates. Very dense, compact rock with good structural characteristics are indicated by apparent resistivity measurements above 10,000 ohm meters through several continuous dipole intervals and a PFE response less than 2 percent. The induced polarization and resistivity technique will give readings to depths of about 600 feet which is probably within the penetration range of most quarrying operations. Faults and large fractures are detectable using the induced polarization survey method, as are cavities and voids containing moisture. Faults may be indicated by no return signal or by no readings at the receiver if all the equipment is operating properly. Under certain conditions negative readings can be obtained, indicating that the circuit may be broken between the transmitter and the receiver. This might be indicative of faulting, fracturing, or some strong discontinuity or of a force field in the underlying strata strong enough to overcome the applied current and to reverse the flow which may be indicative of some geologic phenomenon. It was realized early in the project that good geologists, fully knowledgeable of the induced polarization equipment and its capabilities, could determine a great deal about what geologic formations were occurring below the surface and could approximate depths fairly well. This knowledge would be very useful in preliminary design work, a factor to be considered if the induced polarization equipment also were being used for the purpose of locating iron pyrite deposits in the early stages of design considerations. Another potential use is for locating natural sand-gravel deposits in areas where they may be present.

SUMMARY AND CONCLUSIONS

Sulfide minerals, particularly iron pyrite, can cause some serious problems when deposits of low concentrations of minute crystals contained in rather porous rock are disturbed in highway construction and permitted to weather. Remedial measures, after the weathering and breakdown of iron sulfide begins, can be expensive and difficult. Locating the troublesome deposits and avoiding them or incorporating into the design the methods of dealing with them seem to be the most logical, economical, and practical means of avoiding problems associated with the exposure of sulfide mineral bearing rocks.

The feasibility of locating troublesome iron pyrite deposits using induced polarization and resistivity surveys was well-demonstrated on the proposed forest highway connecting Tellico Plains, Tennessee, and Robbinsville, North Carolina. This was the first time that induced polarization techniques had been applied to highway work. The technology for such use had been proven and in existence since the early 1950s. Excellent results were obtained in this demonstration, and induced polarization surveys have been conducted on some other projects since. Some other benefits of the use of induced polarization surveys in highway work also were demonstrated to be feasible. However, some research into interpretative techniques and correlations between data generated and geologic formations should be conducted before actual field applications are made.

Induced polarization surveys may be conducted most profitably after good preliminary center lines and profile grade lines have been developed but before lines and grades are committed to such a stage that only costly shifts and adjustments can be made. If some preliminary geological studies have been conducted, including some on site reconnaissance, the survey time required may be kept to a minimum and having to recall the induced polarization survey team may be avoided, an important factor if private firms are used. In this concept, the induced polarization process shows good promise as a planning, location, and design tool in the highway field. As with any technique, the capability of the process must not be exceeded nor the value of a qualified geologist operator and interpreter minimized. Induced polarization is no genie, although it will find gold.

ACKNOWLEDGMENT

This demonstration project was funded by the U.S. Department of Transportation, Federal Highway Administration, Region 15. The authors wish to thank and commend Gary Klinedinst, who was responsible for administering the contract; Ken Hall, a very capable geotechnical engineer who was the contract monitor and who spent many days in the field working closely with the crew; and I. Paul Sunwoo, an aggressive engineer with innovative ideas who brought the authors together with his Region 15 supervisors in order to attack an old problem with a new approach.

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EVALUATION OF THE ACID DRAINAGE POTENTIAL OF CERTAIN PRECAMBRIAN ROCKS IN THE BLUE RIDGE PROVINCE

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ABSTRACT

In humid areas of the U.S. a toxicity associated with acid can arise from oxidation of iron-disulfides. Because present law prevents potentially deleterious materials being left exposed creating sources of pollution, a need exists for evaluating materials disturbed in the course of construction.

Acidic drainage from construction in the Blue Ridge when little or no precautions were taken caused severe adverse impacts on biota of several streams. Rocks of the Blue Ridge with the highest potential for creating acid problems are within the Great Smoky Group of the Precambrian, Ocoee Supergroup. Either of two lithologies dominating the group, metasandstone or argillite (slaty), may contain sufficient concentrations of iron-disulfides to be deleterious. Lithologic distinction is no sure-fire means for predicting acid potential; however, argillaceous rocks most often tend to present the problem.

Pre-construction geophysical surveys and/or geochemical analyses of cores delineate many potential problems; however, the sulfides are irregularly distributed and some rocks possess a neutralization potential. In one case a preliminary volumetric estimate of material to be encapsulated was 10,500 yd^3; a second appraisal was 52,500 yd^3; and finally through co-construction evaluation 31,500 yd^3 was buried. If continuous evaluations had not been made, some material might have been unnecessarily buried, or perhaps, indiscriminantly used in construction.

Preliminary on-site evaluations were microscopic estimates of sulfide mineral content, and when the sulfide content was estimated 1%, the material was analyzed to determine its acid-base balance. A decision could then be made as to the materials' disposition.

Acid drainage resulting from weathering of disturbed pyritiferous rocks is an all to well known problem associated with the coal industry - especially in the humid regions of the country. The coal industry has devoted a great deal of research and development toward reducing or eliminating the problem of acid drainage. Geochemical analyses of overburden predict potential acidity as well as levels of potentially toxic trace elements that may become mobilized by increased acidity.

Also, regional stratigraphic studies - especially along some of the panoramic highway cuts of the Kentucky Interstate System - have demonstrated some relationships between depositional environments of the coal-bearing strata and their potential for acid generation. Hence, when the geology is well known, adverse environmental impacts may be reliably predicted and at least minimized.

Generation of acid by weathering of rocks is not unique to the coal-bearing strata of the Carboniferous. Any sulfidic rock - especially those sedimentary rocks formed in euxinic environments containing iron-disulfides are subject to the problem. Certain Late Precambrian metasedimentary rocks of the Blue Ridge province were originally formed under ancient euxinic conditions and contain sufficient iron-disulfides to create adverse conditions when subjected to accelerated weathering through excavation.

Reports have been made of several case histories of acid-related problems in the Blue Ridge caused by disturbance or indiscriminant use of pyritiferous Precambrian rocks. One of the earliest reported was by Huckabee, et al in 1963. Here the demise of aquatic biota in Beech Flats Creek in the Great Smoky Mountains was attributed to indiscriminant use of pyritiferous rock as rip rap along the stream embankment. More recently problems were encountered during construction of a leg of the Tellico Plains, Tenn.-Robbinsville, N.C. Scenic Highway.

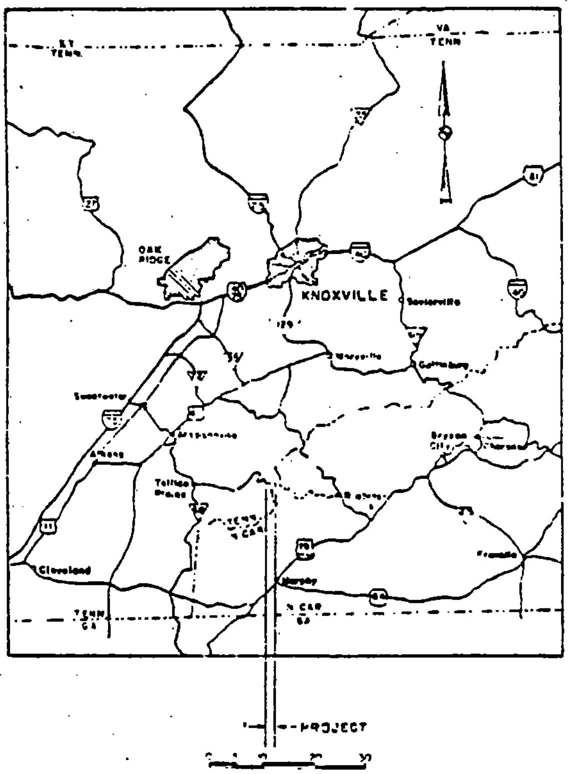
Biotic and chemical analyses by the Tennessee Division of Water Quality control and the Federal Highway Administration (FHWA) on several "blue ribbon" trout streams draining this leg of the project and flowing into the North River revealed severe water quality deterioration related to increased levels of acidity. Tracing the source of the problem by sampling upstream the cause became clear - indiscriminant use of pyritiferous rocks in fills and embankments and some over-blasting by the contractor. A temporary mitigative measure, trickling NaOH into the headwaters of affected streams, was instigated; however, this measure was not without problems and the technique was prohibitive, cost-wise, as a perpetual solution of the acid problem.

Some techniques dealing with the acid problem can be borrowed from the coal industry. Although "Induced Polarization" studies as have been described at this meeting are useful for first-cut appraisals, one major limitation of geophysical methods of this sort is their inability to detect materials containing neutralizing elements which can counteract potential acidity. This is indeed a very important factor when considering costs for hauling volumes that are safe for ordinary fill or those volumes that must be encapsulated. It is my observation that nothing can supplant co-construction evaluation of rocks as they are exposed. This was perhaps demonstrated during construction along the last 2.4 miles of the Tennessee portion of the Tellico Plains - Robbinsville Scenic Highway (Fig. 1).

This project located in the Blue Ridge Physiographic province ascends the Unicoi Mountains of the Unaka Chain - southwest of the Great Smoky Mountains - in Monroe County, Tennessee. The project begins near the Falls Branch Scenic Area of the Cherokee National Forest, extends along Sassafras Ridge, and ends at Beech Gap on the Tennessee-North Carolina state line.

The rocks disturbed in the course of this project are classified as metasedimentary units within the Great Smoky Group of the Ocoee Supergroup. Figure 2 is a generalized stratigraphic column showing the relationships of these rocks. These rocks have undergone several episodes of deformation which accounts for their metamorphism and the overall structural complexity in this region. In general the major structural elements strike northeastward between N 48^{o}E and N 72^{o}E and dip southeastward ranging from 38^{o} to vertical.

Three lithologic assemblages dominate the stratigraphy of the project area. Two are predominantly metasandstone: one consisting of metasandstone or metasiltstone interbedded with argillite, and the other consisting of massively bedded metasandstone. The third is argillite. Sulfide occurrences are the highest in the argillites and in the metasandstones that are interbedded with argillite.



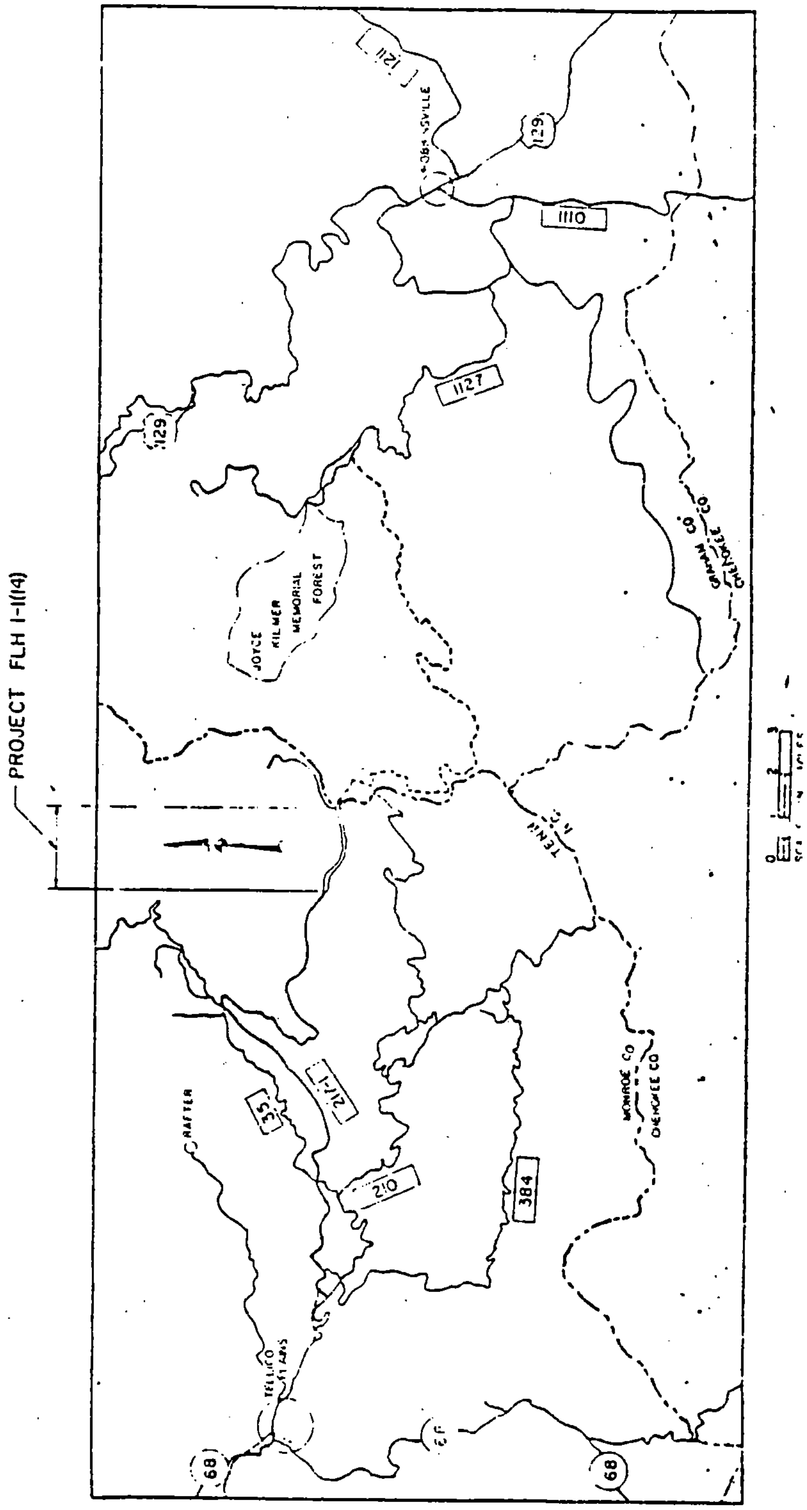


Figure 1. Location of the Tellico Plains - Robbinsville Scenic Highway-Project FLH 1-1 (14).

<table>
<tr><td></td><td colspan="3">King, et al (1958)</td><td></td><td colspan="2">Hurst & Schlee (1962)</td></tr>
<tr><td>Cambrian</td><td colspan="3">Chilhowee Gp.
Cochran Cgl. & Higher</td><td rowspan="7">Correlation Uncertain</td><td colspan="2">Murphy Gp.
Nantahala Fm. & Higher</td></tr>
<tr><td rowspan="6">Precambrian</td><td rowspan="6">Ocoee Supergroup</td><td>Walden Cr. Gp.</td><td></td><td colspan="2">?</td></tr>
<tr><td rowspan="4">Great Smoky Gp.</td><td>Unamed Ss.</td><td rowspan="4">Great Smoky Gp.</td><td>Dean Fm.</td></tr>
<tr><td>Anakeesta Fm.</td><td rowspan="3">Hothouse Fm.
Hughes Gap Fm.
Copper Hill Fm.</td></tr>
<tr><td>Thunderhead Ss.</td></tr>
<tr><td>Elkmont Ss.</td></tr>
<tr><td>Snowbird Gp.</td><td></td><td colspan="2">?</td></tr>
</table>

Figure 2. Generalized Stratigraphic Section. Nomenclature is that which has been used in areas adjacent to the project.

The lithologic unit herin designated as argillite is primarily a dark, silty, and argillaceous rock that has been altered to slate, phyllite, or even schist. Carbon and iron-disulfide present in the rock produce weathered outcrops which are dark and rusty in color. Although the rusty iron-oxide staining of the rocks may occur in the presence of iron sulfides, all oxidized iron is not related to pyrite. The unit is similar to the Anakeesta described by King (1968) in the Great Smokies.

The metasandstone unit is a gray sandstone consisting mainly of quartz, orthoclase, plagioclase, micas, and small amounts of undisintegrated rock clasts which has been metamorphosed. Pyrite may occur in this unit when it is massive, but only as very disseminated trace amounts. The metasandstone occurring as interbeds with slate is more prone to larger amounts of the disseminated iron-disulfides. Graded bedding is observable in the coarser-grained portions of this lithologic unit. The unit is similar in description to King's (1968) Thunderhead.

The metasandstone/argillite unit consists of interbeds of variable thicknesses of the two lithologies described above. The fact that the metasandstone often contains significant amounts of sulfide minerals even though it is much lighter in color than the argillite has caused past problems of differentiating potentially deleterious rocks from those that are not. Without any real analyses dark rocks have usually been categorized as "bad" and light rocks as "good." Hence, in a number of situations some dark colored rocks are blamed for what light rocks are producing in terms of potentially toxic conditions.

All three of the units are believed to have intertonguing relationships. The repetition of the argillite unit along the project alignment may be in part due to structural relations, but because of the presence of the metasandstone/argillite unit which is definitely intertonguing, an intertonguing interpretation seems more plausible.

The sediment represented by the argillite is interpreted to have accumulated as organically rich mud in depositional basins having low levels of free-energy and oxygen. Such conditions allowed the fine sediments to accumulate and the contained organic sulfur to be reduced to form sulfide minerals. This rather quiet depositional environment which gave rise to the sediment comprising the present slates was periodically disturbed when sand, now comprising the metasandstones, was washed in by a relatively higher energy sediment transporting agent. The graded bedding in some of the metasandstones suggest that they may be resedimented materals transported by turbidity currents from some intermediate depositional site to which they had been transported by a high-energy agent from a granitic source area.

Pyrite is likely to be found in any of the described rock types, although its origin is primarily attributed to the dark carbonaceous argillites. The sulfides may have been mobilized during deposition or during deformation and metamorphism. Again, no clear distinction regarding sulfide content can be made on the basis of rock color.

The acid drainage that is associated with certain Precambrian rocks is attributable to the oxidation of iron-disulfides. Although there is no totally acceptable definition of acidity, it is most commonly expressed as pH - a measure of the free hydrogen ions dissolved in water. Figure 3 is an example of the oxidation process whereby free hydrogen ions, hence acidity, is generated. Based upon this relationship the potential acidity of a rock can be stoichiometrically calculated in $CaCO_3$ equivalents, if the amount of pyritic sulfur is known. For example, material with 0.1% pyritic sulfur yields an amount of H_2SO_4 that would require 3.125 tons of $CaCO_3$ to neutralize 1000 tons of that material.

Besides having the potential for acid production most rocks contain some bases in the form of alkali carbonates, silicates, or exchangeable bases that can serve to neutralize acidity. Samples can be treated with standardized acid, then titrated to

- $FeS_2 + 7/2\ O_2 + H_2O = Fe^{++} + 2\,SO_4^{=} + 2H^+$
- $Fe^{++} + 1/2\ O_2 + H^+ = Fe^{+++} + 1/2\ H_2O$
- $Fe^{+++} + 3\ H_2O = Fe(OH)_3 + 3H^+$
- $FeS_2 + 14Fe^{+++} + 8\,H_2O = 15\,Fe^{++} + 2\,SO_4^{=} + 16H^+$

Figure 3. Acid Produced By Pyrite Weathering.

determine the amount of bases present in $CaCO_3$ equivalents. This then is the neutralization potential of that rock which can be balanced against the acid potential. Figure 4 is an acid-base account for a rock core taken on the project.

From the acid-base account potentially toxic material was defined as any rock material having a net deficiency (more acid than base) of 5 or more tons of $CaCO_3$ equivalent per 1000 tons of material. This division between toxic and non-toxic material is arbitrary, but has been used successfully in working with rocks were acid generation has been a problem (Sobek, et al, 1978). Potentially toxic rock materials require special handling and some mode of isolation to protect the environment from the acidity.

Approximately 30,000 yd^3 were recommended for encapsulation within three burial sites on the project: about 4,000 yd^3 in a false-cut located at 954 ±, about 11,000 yd^3 at 995 ±, and about 15,000 yd^3 at 965 ±. Approximately 10,000 yd^3 were blended and used within fill areas. A summary of those sections of the project where potentially deleterious rocks were encountered is given in Table I. Table I also indicates the recommendations made with regard to the disposition of the material. As can be seen, in certain areas where the volume (thickness) of potentially acid-producing rock was not great, or the acid potential was not too high, and it was contiguous to a sufficient volume (thickness) of rock material with a neutralizing potential, the materials were blended together with agricultural lime and placed with care into the center of selected common fills.

The work on this leg of the highway project is in its final stages, and the jury, so to speak, is still out on the decision as to how efficacious the work on reducing or eliminating pollution has been. It may take 5 or more years to fully evaluate whether the results are commensurate with the efforts.

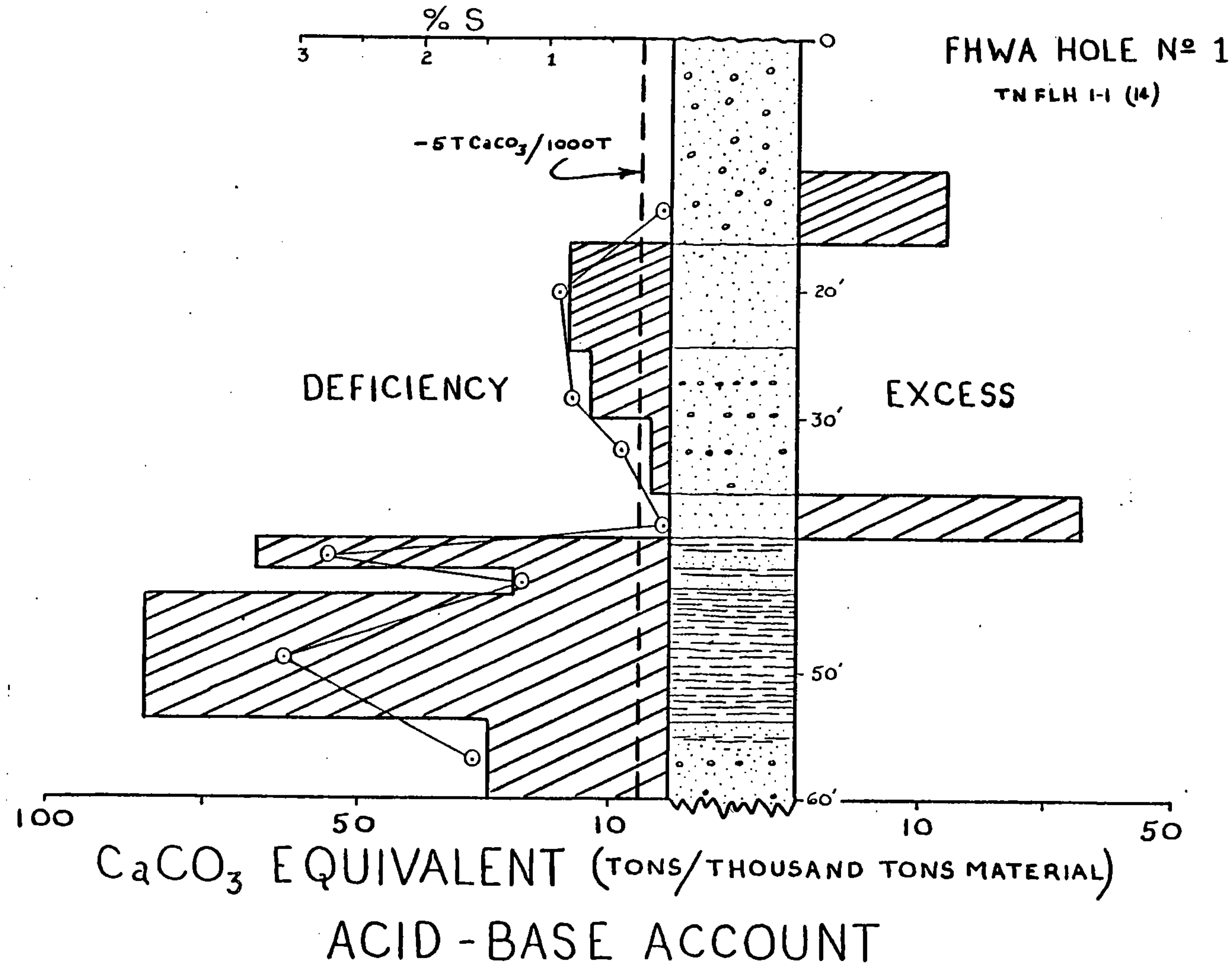


Figure 4. Acid-Base Account for FHWA Borehole No. 1, 958 + 00.

TABLE I

A Summary of the Character and Disposition of Potentially Deleterious Rocks Encountered Along Project FLH 1-1 (14).

Station to Station		+Lithotype	Estimated Volume yd^3	%S	Acid Pot. T $CaCO_3$ per 1000T	Neut. Pot. T $CaCO_3$ per 1000T	*Disposition
956 + 00	959 + 00	1	7,100	1.58	49.38	6.5	B
959 + 00	969 + 50	1	2,000	0.40	12.50	4.81	F
988 + 00	989 + 50	1	3,600	1.28	40.00	5.18	B
996 + 50	999 + 00	2	11,000	1.69	52.81	9.65	B
1053 + 75	1054 + 50	3	1,800	1.10	34.17	16.25	B
1063 + 00	1064 + 50	3	650	0.86	27.42	5.90	B
1071 + 50	1076 + 25	3	8,800	0.53	16.77	6.39	F
1076 + 25	1078 + 00	3	3,500	0.85	26.53	9.97	B
1082 + 75	1084 + 00	3	2,500	0.74	23.04	6.55	F

\+ 1 = Mostly Argillite
2 = Interbedded Argillite and Metasandstone
3 = Mostly Metasandstone

* B = Disposed at Burial Site
F = Blended and used in Common fill.

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Some Geotechnical Aspects - Early Planning Along Corridor K, Appalachian Development Highway; Section Between Andrews and Almond, North Carolina

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North Carolina Department of Transportation
Area Highway Geologist

INTRODUCTION

Study along a corridor designated to run between Cleveland, Tennessee and Sylva, North Carolina was initiated soon after passage of the Appalachian Regional Development Act in early 1965. This paper deals with one section of that proposed highway lying between Andrews and Almond, North Carolina. After periodic consideration of several alternate routes, between the years of 1965 and 1979, the decision was made to concentrate environmental and geotechnical studies along a corridor designated as alternate 8. This proposed corridor lies between and connects the towns of Andrews, Robbinsville, Stecoah and Almond, North Carolina. The decision to concentrate on this alternate was partially based on a combination of environmental and geologic considerations. See Figure 1 for a map showing all study alternates and the geology of the area.

In addition to potential slope stability problems in major cuts which would be required in a project of this magnitude, concerns were voiced over potential acid run-off due to disturbance and exposure of rocks with high sulfide content.

BACKGROUND

The Geotechnical Unit of the NCDOT became involved in the proposed project during the summer of 1977. Early reports submitted were based primarily on research, conversation, and on personal observation and experience. Slope stability of existing cut slopes in the relevant rock formations were of prime importance. Regional structural trends and mineralogy were also important considerations, since the rocks in this area are frequently anisotropic in their strength characteristics. The rocks are characteristically bedded in nature and possess two or more joint systems which may lead to either planar or wedge failure. Mineralogy is important in two respects: (1) indication of speed and depth of weathering; and indication of residual soils types and their relative stability, and (2) significant environmental damage can occur if highly sulfidic rocks are exposed to weathering processes. This damage may take two forms: (1) weathering processes may produce sulfuric acid in quantities adequate to create acid conditions in small streams receiving drainage from the rock, resulting in kills of fish and other life forms, and, (2) if iron is present, it becomes soluble under acid conditions and may be precipitated downstream as brown scum. A potentially serious effect which is non-environmental in nature is heating produced by the oxidation process, which may produce granulation of the rock and significant settlement in embankments.

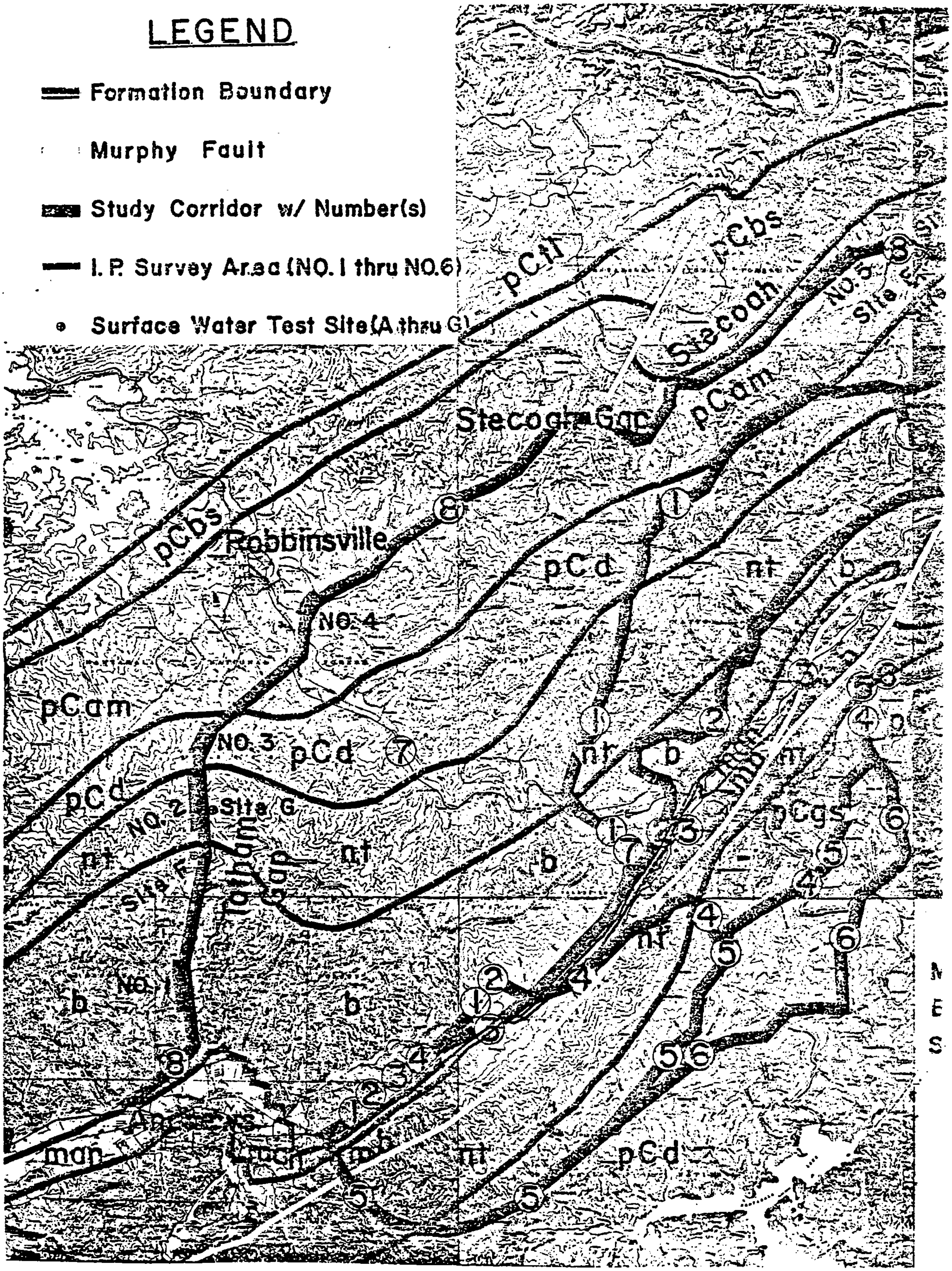
LEGEND
Formation Boundary
Murphy Fault
Study Corridor w/ Number(s)
I. P. Survey Area (NO. I thru NO.6)
Surface Water Test Site (A thru G)
pCtl
pCbs
Stecoah
NO.5
Stecoah Gap
pCam
Robbinsville
pCd
nt
b
NO. 4
NO. 3
NO. 2
Site G
Site F
Tatham Gap
NO. I
pCgs
Andrews

Figure 1

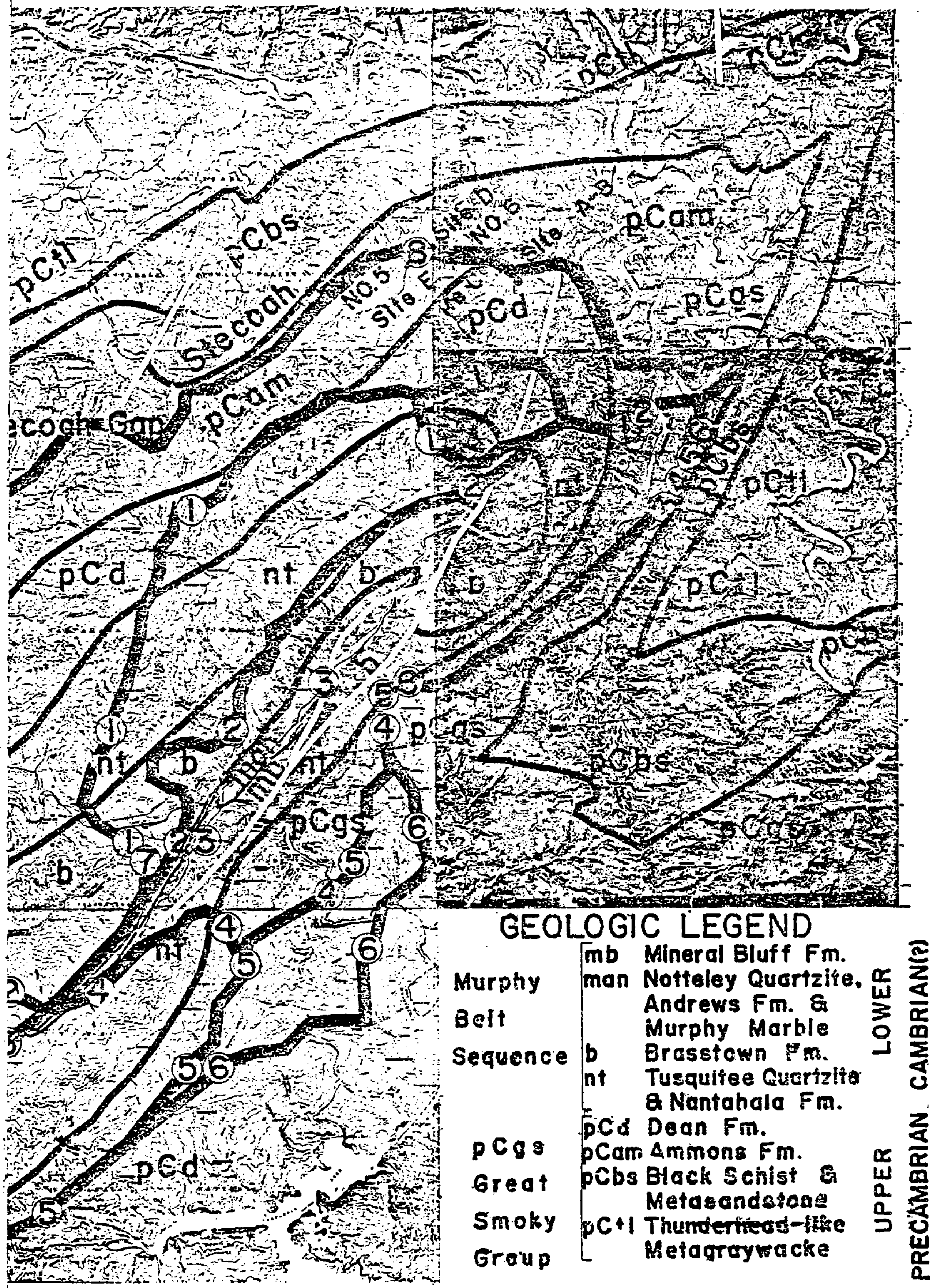
GEOLOGIC LEGEND
Murphy Belt Sequence
mb Mineral Bluff Fm.
man Notteley Quartzite, Andrews Fm. & Murphy Marble
b Brasstown Fm.
nt Tusquitee Quartzite & Nantahala Fm.
pCgs Great Smoky Group
pCd Dean Fm.
pCam Ammons Fm.
pCbs Black Schist & Metasandstone
pC+l ~~Thunderhead~~-like Metagraywacke
LOWER CAMBRIAN(?)
UPPER PRECAMBRIAN
Stecoah
Stecoah Gap
pCam
pCbs
pCd
pCgs
nt
b

Figure 1

Figure 2

GEOLOGIC COLUMN

MURPHY BELT SEQUENCE	mb	Mineral Bluff Formation	LOWER CAMBRIAN(?)
	man	Nottely Quartzite, Andrews Formation and Murphy Marble	
	b	Brasstown Formation	
	nt	Nantahala Formation	
GREAT SMOKY GROUP	pCd	Dean Formation	UPPER PRECAMBRIAN
	pCam	Ammons Formation	
	pCbs	Black Schist and Metasandstone	
	pCtl	Thunderhead-like Metagraywacke	

An unusually wide range of geologic and topographic conditions are involved along the proposed corridor. A total of seven mappable rock units, some of which consist of two or three differing rock types, are involved on one or more of the proposed alternates.

One of the prime reasons for the environmental sensitivity of this project lies in the fact that all corridors except Alternate 8 are involved to some extent with the Nantahala Gorge, an extremely rugged and beautiful gorge, and major tourist and white water enthusiast attraction. It is, in fact, one of the most popular centers for white water rafters and canoeists in the eastern United States.

The major structural feature of the area is the Murphy Fault, which exercises strong control over the geology and the physiography of the region. The Murphy Fault is a thust fault with a maximum throw of approximately one mile. The point of maximum throw lies near Andrews. The fault has thrust the southeastern limb of the regional geosyncline over the northwestern limb.

The following paragraphs briefly describe the major characteristics of the formations present in the study area. Formations are discussed in order of increasing age (see Figure 2). As previously indicated, the regional structure is geosynclinal; the axis of the syncline is approximately the same as the course of the Nantahala River between Hewitt and Wesser and maintains the same trend outside the gorge. The following seven units are involved; some units include more than one formation and one unit has not been identified at this time as to formation (black schist and metasandstone).

(1) Mineral Bluff Formation

This formation consists predominantly of pelitic schists with intercalated quartzites, sandy lenses and calc-silicate granofels.

The schists and phyllites have a prominent secondary foliation which coordinates with the primary schistocity to create a high slide potential.

Weathering is moderate to deep with moderate potential for slope stability problems. There appears to be no signifcant potential for environmental impact from the sulfide standpoint.

(2) Man Unit (includes the Notteley Quartzite, the Andrews Formation and the Murphy Marble)

The Notteley Quartzite is a very hard orthoquartzite, mineralogically 70% to 90% quartz with some feldspar. This formation is unlikely to be present in the study area, except in isolated occurrences.

The Andrews formation consists of alternating marble and cross-biotite schists with the schists predominating. The formation contains considerable pyrite in pelitic layers. Also present are iron sulfides which were mined extensively during the 1920's. The rapid weathering characteristics of the calcareous beds promote stability problems in some cut slopes. Furthermore, the formation presents serious sulfide contamination potential.

The Murphy marble consits of both dolomitic and calcitic marble with substantial talc interbeds. This formation is being mined actively for crushed stone at Hewitt.

(3) Brasstown Formation

This formation consists predominantly of greenish gray to dark gray cross-biotite schists but contains some micaceous quartzites.

Structurally characterized by prominent schistocity planes, the Brasstown Formation exhibits two or more major joint sets which can serve as release surfaces for planar failure on schistocity planes.

The formation is moderately resistant to weathering, but is probably weathered along schistocity planes to significant depths.

The schists and quartzites are mineralogically "clean", containing only trace quantities of sulfides.

(4) Tusquitee Quartzite and Nantahala Formation

These formations consists predominantly of sulfidic schists with much graphite; some metasiltstones with interbedded white metaquartzites are also present. Structurally, the formations are moderately competent in the fresh state and exhibit moderately slow weathering characteristics, except along schistocity planes. Stability in cut slopes is good if schistocity planes are favorably oriented or can be handled through design or retention methods. However, the schists and metasiltstones have high potential for sulfide comtamination when excavated.

The quartzite beds lend stability to cut slopes and have little or no potential for sulfide contamination.

(5) Dean Formation

The Dean formation consists predominantly of muscovite-quartz schists with much gray fine to coarse-grained interbedded metasandstone. Cut slopes in fresh rock should remain relatively stable, however, the formation is frequently deeply weathered. When weathered, the soils are moderately to very slide-prone, especially in the more micaceous zones. Mica content ranges between 0% and 60%. Sulfides, while almost certainly present to some extent, do not appear to pose a substantial hazard.

(6) Ammons Formation

This formation consists predominantly of metasandstone with abundant metasiltstone and muscovite schist. Slopes are fairly stable when cut in fresh rock; the formation, however, is deeply weathered over much of its area of exposure and slopes in the weathered material are moderately to very slide-prone. This is due to high mica content and/or the sandy, weakly cohesive nature of the soils.

While sulfides are probably present, the mode of occurance and low percentages are generally such as to make the potential for significant levels of leaching quite low. The highest potential for a sulfide leachate problem occurs in the Horse Branch member.

(7) Black Schist and Metasandstone

This formation is not likely to be encountered in the project area. It is very similar to the Anakeesta formation and has been erroneously correlated with the Anakeesta in the past. The formation consists predominantly of a black to gray muscovite schist with abundant pyrrhotite and graphite interbedded with a gray metasandstone composed of quartz, feldspar, and subordinate biotite. Minor units of para-amphibolite, tremolitic marble and bedded, nodular, calc-silicate granofels are also present. This unit has considerable potential for slope stability problems due to its schistose nature and mineralogy. There is also considerable potential for acid drainage problems due to sulfides.

SCOPE OF WORK

Late in 1979, it was determined that Alternate 8 was to be the corridor used for the proposed highway. Since this decision, the investigation has been concentrated in the area of Alternate 8.

Subsequently, a decision was made to conduct an induced polarization (I.P.) study along Alternate 8. As the initial step in this survey, a recommendation was made suggesting five study sections along this route. In mid 1980, an on-site conference involving all concerned personnel was held to finalize study intervals.

I.P. SURVEY ALONG ALTERNATE 8

The intent of the selection process was to conduct a survey in each potentially different rock unit. The following intervals and corresponding rock formations were selected.

Baseline Number	Station Interval (-L-)	Formation
1	220-270	Brasstown
2	375-340	Nantahala
3	440-462	Dean
4	590-612	Ammons
5	1157-2206	Grassy Branch Ammons
6	1282-1316	Horse Branch Ammons

A contract was let early in June to Phoenix Geophysics of Denver, Colorado to conduct the surveys. Work was begun on August 22 and completed on August 30.

The I.P. survey was conducted in two phases. Phase I included surveys on a reconnanissance mode in areas 1, 2, 3, 4, and 6. Phase II included a detailed mode survey on several short survey lines included in area 5 (Lines 5A through 5F) and detail testing along any line intervals determined to be possibly anomalous during Phase I.

Reconnanissance mode surveys were conducted on a pole-dipole configuration with a 200-foot electrode spacing. Detail surveys were conducted on a dipole-dipole configuration with an initial spacing of 50 feet between electrodes (N=1); other readings were taken with N=2, 3 and 4.

Definite anomalous I.P. responses were obtained near both ends of Baseline #2 and possible anomalous I.P. responses were noted on Baselines #4, #5 and #6. In the cases of Baselines #4 and #6, detailed surveys did not pick up the anomalies. The possible anomalies on Baseline #5 were interpreted to reflect the strong topographic effect of the existing cut slopes. As a result of this testing and on the basis of the report submitted by Phoenix Geophysics, it was decided to conduct a field drilling program to determine the reason for the positive responses or anomalous zones along Baseline #2.

Theory of the Induced Polarization Method

The geophysical parameter measured by the Induced Polarization method is the result of the polarization of metallic or electronic conductors in a medium of ionic solution conduction. This phenomenon occurs wherever electrical current is passed through an area containing metallic minerals such as base metal sulfides. Normally, conduction of a current through the ground is through ions in the ground water, i.e. by ionic conduction. The group of minerals commonly described as metallic, however, have specific resistivities which are normally responsible for changes in conduction of electrical currents through the ground. The induced polarization effect takes place at those interfaces where the mode of conduction changes from ionic in the solutions filling the interstices of the rock, to electronic in the metallic minerals in the rock. The blocking action or induced polarization, which depends upon the chemical energies necessary to allow the ions to give up or receive electrons from the metallic surface, increases with the time that a DC current is allowed to flow through the rock; i.e. as ions pile up against the metallic interface the resistance to current flow increases.

From an alternate viewpoint, it can be seen that if the direction of the current is reversed repeatedly before polarization occurs, the effective resistivity of the system will vary with the frequency of the switching. This is due to the fact that the amount of current flowing through each interface is dependent on how long the current has been flowing in one direction. This geophysical parameter is designated as the percentage frequency effect.

The values of the percentage frequency effect or F.E. are a measurement of polarization in the rock mass. However, since the degree of polarization is related to the resistivity of the rock mass, it is found that the metal factor values or M.F. are more useful than F.E. in determining the amount of polarization present. The M. F. (metal factor) values are obtained by normalizing the F.E. value for varying resistivities. In effect, normalization of the F.E. value removes the resistivity of the rock as a variable. Since the resistivity is reduced to a constant, the M.F. reflects only the degree of polarization and, consequently, the amount of polarizable material present. Metal factors greater than 10 are normally considered to indicate significant polarization.

The metal factor (M.F.) is calculated by means of the following formula:

$$\text{M.F.} = \frac{(\text{F.E.}) \times 100}{\frac{p(f_1)}{2\pi}} \times 1000 = \frac{\left[\frac{p(f_1)}{p(f_2)} - 1\right] \times 100}{\frac{p(f_1)}{2\pi}}$$

$$\text{M.F.} = \left[\frac{1}{p(f_2)} - \frac{1}{p(f_1)}\right] \times 2\pi \times 10^5$$

Where $p(f_2)$ is the resistivity at frequency f_2, $p(f_1)$ is the resistivity at frequency f_1 and F.E. is the precent frequency effect.

The greatest application for the I.P. method has been in the search for disseminated metallic sulfides of less than 20% by volume. However, it has also been used successfully in the search for massive sulfides in situations where, due to source goemetry, depth of source, or low resistivity of surface layers, the E.M. (electromagnetic) method cannot be successfully applied.

In normal operations the I.P. method does not differentiate between the economically important metallic minerals such as chalcopyrite, chalcocite, molybdenite, galena, etc., and other metallic minerals such as pyrite. Other electronic conducting materials which can produce an I.P. response are magnetite, pyrolusite, graphite, and some forms of hematite.

Resistivity values can be roughly interpreted to indicate rock density and probable rate of release of soluble minerals. Resistivities of approximately 5000 ohm-meters or higher indicate relatively dense rock and relatively slow rates of release. Low resistivity indicates rock types which tend to weather more rapidly.

Discussion of Study Areas

A discussion of each base line, giving relevant values for the important parameters, follows.

Along Baseline No. 1, underlain by rocks of the Brasstown Formation, no anomalous zones detected. Resistivities ranged between 1,827 and 12,625 ohm-meters; F.E.'s between 3.4 and 6.1 and M.F.'s between 0.4 and 2.7 The M. F.'s observed are well below the generally accepted value of 10 considered to be indicative of significant polarization.

Along Baseline No. 2, which traverses the Nantahala Formation two definite anomalous zones were noted; one near the beginning of the survey and the other at the end. In the first anomaly, resistivities ranged between 433 and 886, F.E.'s between 13.2 and 16.8 and metal factors between 15 and 39. In the second anomaly, which was much stronger, resistivities ranged between NR (no reading; a current sink) and 136; F.E.'s between NR and 20; and metal factors between NR and 1,889. Along other intervals of this baseline, i.e., the non-anomalous zones, resistivities ranged between 6,285 and 14,429; F.E.'s ranged between 6.7 and 9.0; and metal factors between 0.6 and 1.4

Along Baseline No. 3, underlain by the Dean Formation, no amomalous zones were defined. On this baseline, resistivities ranged between 1,242 and 6,270; F.E.'s between 3.7 and 7.0; and M.F.'s between 0.8 and 5.6. The low resistivities correlate well with the typically deep weathering already noted in the formation discussion earlier in this report.

Baseline No. 4, located over the undifferentiated Ammons formation, indicated one possible anomalous zone at station 16+00. As a result, a detail survey was conducted in this vicinity, which did not pick up the anomaly. Over the remainder of Baseline No. 4, resistivities ranged between 1,990 and 4,577; F.E.'s between 3.0 and 5.6; and M.F. between 0.7 and 3.4. Again, the low resistivities are given credibility by the typically deep weathering observed during reconnaissance of the study area.

Area No. 5 is underlain by the Grassy Branch member of the Ammons Formation.

Due to the nature of the proposed construction (widening of the existing roadway), the survey lines in this area were run along approximate cross-section lines at right angles to Line -L-. Survey lines were run across selected cut faces at five sites.

This formation is not considered to be significantly sulfidic, however, evidence of considerable iron staining on existing cut faces indicates the presence of pyrite.

The detail survey conducted on each of the five lines indicated a possible anomalous zone which appeared near station 13 and was interpreted to be the result of the topography of high cut slopes.

Table Showing Survey Results - Area V

Line	Approx. -L- Sta.	Resistivity Range	F.E. Range	M.F. Range
5A	1156+00	149-4262	2.7 - 7.8	1.7 - 30.0
5B	1159+00	955-5811	2.1 - 8.9	1.1 - 4.2
5C	1172+00	324-5798	1.6 - 9.3	0.8 - 8.1
5D	1174+50	451-5303	2.3 - 8.8	0.8 - 8.4
5E	1205+00	592-7066	2.0 - 7.4	0.4 - 6.8

Depth of weathering in this formation as exhibited in the existing cut slopes is typically shallow, gradational and erratic. Unsurveyed slopes in the interval showed depths of weathering exceeding 40 feet.

Baseline No. 6 is located over the Horse Branch member of the Ammons formation west of Calfpen Gap.

This rock unit is considered to be potentially sulfidic, especially in the vicinity of its contact with the Nantahala formation. Our survey was conducted to the west of that contact.

The I.P. survey picked up a possible anomaly near station 22 and another near station 26. These anomalies were predicted, based on a high F.E. of 10.8 and the low resistivity of 58 at station 22 and an F.E. of 6.7 and a resistivity of 52 at station 26. These numbers give M.F. values of 185 and 130 at stations 22 and 26, respectively. A detail survey was conducted in this vicinity, which did not pick up the anomalies. Along the remainder of Baseline No. 6, resistivities ranged between 433 and 3,809, F.E.'s between 3.4 and 7.3 and metal factors between 1.6 and 29.

In accordance with the recommendation contained in the Phoenix Geophysics report, it was decided to conduct a drilling program along the anomalous sections of Baseline #2 in order to define the source of the anomalous I.P. responses. Drilling was deemed unnecessary in the other areas on the basis of the statement in the consultant's report, as follows: "The other areas surveyed should not encounter sulphide mineralization of sufficient quantity to be more hazardous than that which has already been exposed along Area V".

CONDUCT OF THE DRILLING PROGRAM

The proposed core drilling was begun late in 1980 and completed early in 1981.

Borings 1 thru 5 were advanced to a depth of approximately 100'; borings 6 thru 9 were advanced to depths of 25', 28', 31' and 60', respectively. The shallower borings were drilled in order to more closely define the contacts between the two rock types revealed by the first five holes. See Figure 3 for a planimetric map showing the culture, baseline, boring locations and I.P. data.

The Mineral Research Laboratory, a branch of N.C. State University, prepared selected core for analysis by Dr. Don Byerly, University of Tennessee, who performed the sulfide testing.

The results of the sulfide testing showed no obvious correlation between two rock types present and the sulfide content.

The megascopicably identifiable rock types observed during the investigation included: (1) Light to medium gray fine-grained metasiltstone with metaquartzite laminae, metallic sulfides and other secondary mineralization. The sulfides occur in three modes; disseminated, concentrated along the laminae and as mineral coatings on open and healed joint surfaces and, (2) a dark gray banded slate with a phyllitic sheen containing sulfides in more distinct masses which tend to concentrate along the numerous fracture surfaces, both open and healed. The first type is dominant; occurring throughout Borings 1, 2, 3, 5, 6, 7, and 9 and below approximately 42 feet in Boring 4. The second rock type occurred only in the upper 42 feet of Boring 4 and in the relatively shallow (31 feet) Boring 8. Megascopic identification of the other secondary minerals was not possible. However, the test results strongly indicate significant amounts of carbonate in the form of calcite are present.

An apparent trend was detected in the sulfide and carbonate percentages. The higher total sulfur percentages tended to occur nearer the surface, while the carbonates tended to increase with depth. Exceptions to this apparent trend occur as follows: high carbonate percentages were present in Boring 6 at

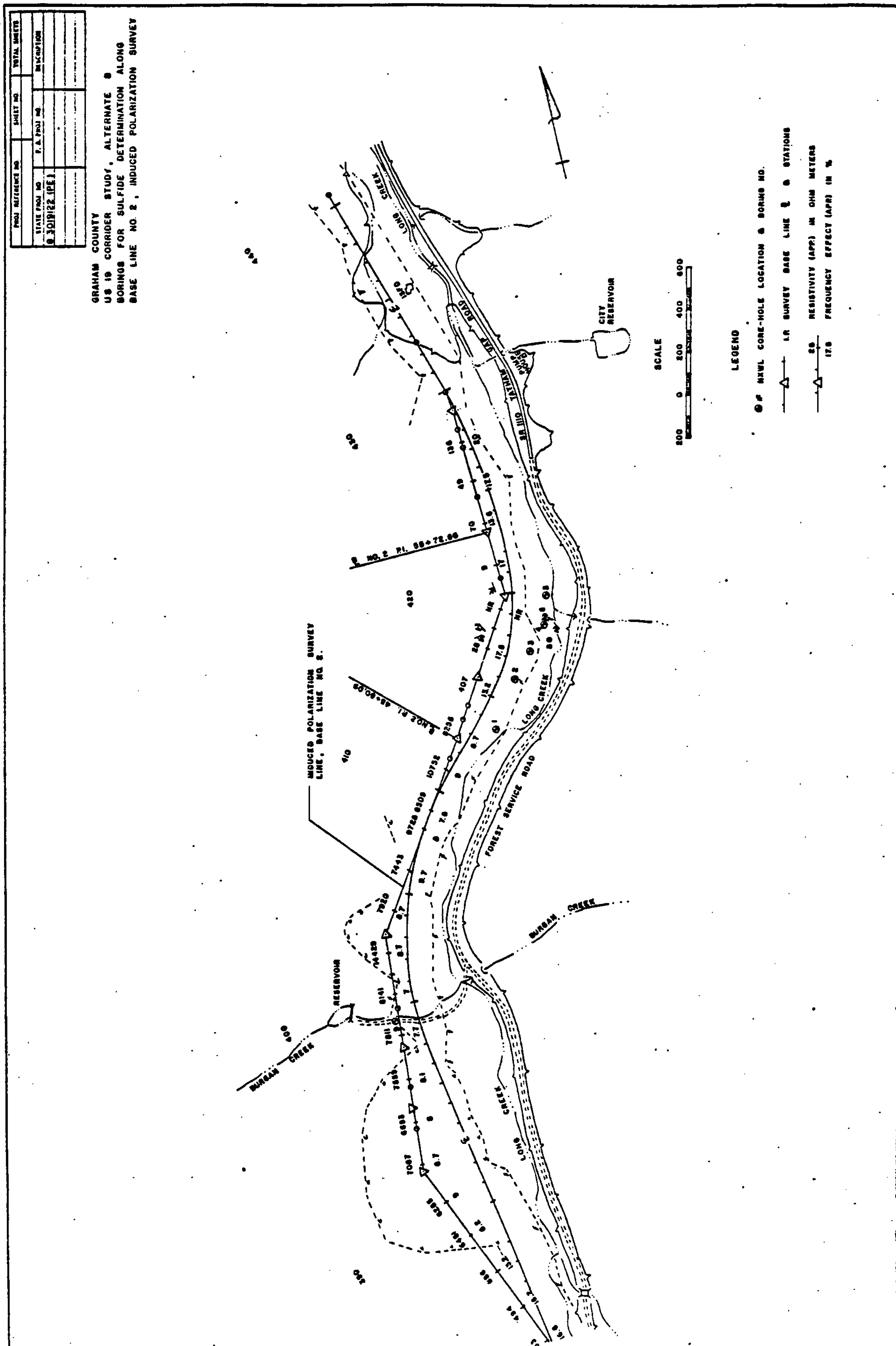
GRAHAM COUNTY
US 19 CORRIDER STUDY, ALTERNATE B
BORINGS FOR SULFIDE DETERMINATION ALONG
BASE LINE NO. 2, INDUCED POLARIZATION SURVEY
CITY RESERVOIR
SCALE
LEGEND
NXWL CORE-HOLE LOCATION & BORING NO.
I.R SURVEY BASE LINE & STATIONS
RESISTIVITY (APP.) IN OHM METERS
FREQUENCY EFFECT (APP.) IN %
LONG CREEK
FOREST SERVICE ROAD
BURGAN CREEK
RESERVOIR
INDUCED POLARIZATION SURVEY LINE, BASE LINE NO. 2

Figure 3

a depth of 15.8 to 19.0 feet and in Boring 8 at a depth of 25.0 to 28.1 feet. These occurrences are in the two differing rock types.

In the table below are the results of the sulfide testing. The number in the sample identification code indicates the hole number from which the sample came.

Sample No.	Depth Interval (in feet)	+Percentage Total Sulphur	Acid Potential T $CaCO_3$/1000T	Neutralization Potential T $CaCO_3$/1000T	*Balance +/-
1A	22.3-25.3	0.88	27.5	1.3	- 26.2
1B	50.6-53.6	0.54	16.88	72.0	+ 5.12
2A	34.8-37.5	1.45	45.31	11.5	- 33.81
2B	92.0-94.8	1.25	39.06	10.9	- 28.16
3A	24.5-28.5	0.79	24.69	22.28	- 2.41
3B	50.1-53.1	0.8	25.0	65.63	+ 40.63
3C	79.5-81.3	1.52	47.5	9.73	- 37.77
4A	17.1-19.9	2.03	63.44	11.85	- 51.59
4B	34.6-37.5	1.72	63.75	17.53	- 36.22
4C	56.4-59.2	0.98	30.63	16.58	- 14.5
4D	85.2-88.1	0.09	2.81	119.38	+116.57
5A	20.7-23.5	2.10	65.63	7.93	- 57.7
5B	42.7-45.6	1.74	54.38	15.75	- 38.63
5C	67.4-70.3	1.74	54.38	6.15	- 48.23
6A	15.8-19.0	0.77	24.06	65.13	+ 41.07

* This figure represents the arithmetic difference between the acid potential and the neutralization potential. (-) indicates a net deficiency (more acid than base). (+) indicates an excess in base account. The arbitrary division between toxic and non-toxic is now set at -5T $CaCO_3$/1000 T of material.

\+ % Total Sulfur is used. It is very unlikely that any organic or sulfate sulfur is present in these metamorphic rocks; it may be assumed, therefore, that all this sulfur is pyritic sulfur. It should also be noted that the percentage given is for <u>sulfur</u>, not <u>sulfide</u>; this percentage would approximately double if translated into sulfide percentage.

Further testing has been arranged with the Mineral Research Laboratory; however, complete results are not yet available. Four composite samples have been selected, on the basis of the sulfide testing, for chemical assays of the elements Ca, Mg, Mn and Al. Sample numbers listed correspond to the numbers assigned for previous sulfide testing. Grouping was done as follows:

Group No.	Common Characteristic	Sample No.
I	High Neutralization Potential	1B 3B

		6A	
II	High Acid Potential	4A 5A 5C	
III	Low Acid Potential	4C 7A 9A 9B	
IV	Moderate Acid Potential	1A 2A 2B	3C 4B 5B

In addition to the chemical assays on the acid soluble fraction of the above sample groups, commercial assays have been arranged for Cu, Zn, Pb, Au and Ag on groups II and IV. Also, sulfide flotation concentrates are to be made on these groups. At this time only the chemical assays for the four groups have been run. The results are summarized in the following table.

Analysis, Percentage *

Group #	CaO	MgO	Mn	Al_2O_3
I	2.17	2.44	0.06	3.99
II	1.61	1.48	0.03	2.75
III	0.66	1.84	0.04	3.88
IV	0.72	1.72	0.03	3.54

* percentages of original head sample

Remarks: (1) Analyzed acid soluable portion for Ca, Mg, Mn and A.

Surface Water Testing

In order to establish base conditions for future reference on streams likely to be affected by construction along Alternate 8, arrangements were made with the Surface Water Section, U.S. Geological Survey, to conduct site tests for pH and conductivity at locations chosen by the Geotechnical Unit.

Ten sites were selected and testing was performed on each of two dates, chosen to reflect conditions at low stream flow and at moderately high stream flow. All results were within acceptable limits indicating no significant sulfide contamination is present in the streams under normal circumstances. In view of the proven sulfide presence along Long Creek and the probable presence of pyrite in the rocks flooring Little Horse Creek between sites B and C, results indicate the sulfide-bearing rocks being traversed are not releasing their sulfide at significant rates under natural conditions. Test results are summarized in the following table. The formations traversed by the test stream at the test sites are listed below the table.

Site No.	Site Description	Low Flow 8-12-80 pH	Low Flow 8-12-80 Cond.	High Flow 9-29-80 pH	High Flow 9-29-80 Cond.
A	Tributary to Little Horse Creek at Dean Farm; 250' rt., sta. 1336+50 -L-	7.1	39	7.1	60
A-1	Tributary to Little Horse Creek at Dean Farm; 270' rt., sta. 1336+00 -L-	7.0	24	7.1	61
B	Little Horse Creek at Dean Farm; 280' rt., sta. 1336+00 -L-	6.8	45	6.7	24
C	Little Horse Creek at Frame and Brick Dwelling; 100' rt., sta. 1301+50 -L-	6.7	45	6.8	40
C-1	Little Horse Creek 200 Feet Upstream From Site C; 200' rt., sta. 1303+50 -L-	6.7	45	7.0	46
D	Little Horse Creek at Mouth to Fontana Lake; 50 lt., sta. 1268+00 -L-	7.1	39	6.8	50
E	Panther Creek at Mouth to Fontana Lake; Near Centerline, sta. 1267+50 -L-	7.1	25	6.6	26
F	Long Creek Near Tatham Gap on Tatham	6.9	12	6.9	12
G	Long Creek Above Old Robbinsville Reservoir Pool; 250' rt., sta. 388+50 -L-	6.7	14	6.7	14
G-1	Long Creek Below Old Robbinsville Reservoir Pool; 250' rt., sta. 388+50 -L-	6.7	14	6.7	14

The formations associated with each test site are as follows:

Site No(s).	Formations Traversed
A, A-1, B	Dean Fm.
C, C-1	Dean Fm.; Horse Branch Member, Ammons Fm.
D	Dean Fm.; Horse Branch Member, Ammons Fm.; Grassy Branch Member, Ammons Fm.
E	Ammons Fm.; Grassy Branch Member, Ammons Fm.
F	Brasstown Fm.

Site No(s).	Formations Traversed
G	Brasstown Fm.; Nantahala Fm.
G-1	Nantahala Fm.; Dean Fm. (?)

SUMMARY

There is a potential sulfide leachate problem through that section of this project which traverses the Nantahala Formation. The observed trend of the laboratory analyses toward higher sulfide content near surface, coupled with field observation of shallow to outcropping rock in the interval of highest sulfide concentration makes virtually any cutting likely to produce some problems. While sulfides are present throughout the Great Smoky Group Rocks, they do not generally constitute a significant hazard for this proposed construction.

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