PROCEEDINGS OF THE 43RD ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

AUGUST 19-21, 1992
Fayetteville, Arkansas

CO-SPONSORED BY

Department of Civil Engineering, University of Arkansas

Arkansas State Highway and Transportation Department

and

Arkansas Geological Commission

CHAIRMAN OF LOCAL ARRANGEMENTS

Sam I. Thornton
BELL 4190, University of Arkansas
Fayetteville, AR 72701

43RD HIGHWAY GEOLOGY SYMPOSIUM
AUGUST 19-21, 1992

FAYETTEVILLE, ARKANSAS

Program: TUESDAY, 18 AUGUST
5:00 to 6:30 PM Registration (Hilton)

WEDNESDAY, 19 AUGUST
8:00 to Noon Registration (Continuing Education 4th floor)

Highway Geology Symposium

HISTORY ORGANIZATION AND FUNCTION *

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on February 16, 1950, in Richmond, Virginia. Since then 39 consecutive annual meetings have been held in 26 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 Annual Symposium moved to different locations as follows:

Year	HGS Location	Year	HGS Location
1965	Lexington, KY	1966	Ames, IA
1967	Lafayette, IN	1968	Morgantown, WV
1969	Urbana, IL	1970	Lawrence, KS
1971	Norman, OK	1972	Old Point Comfort, VA
1973	Sheridan, WY	1974	Raleigh, NC
1975	Coeur d'Alene, ID	1976	Orlando, FL
1977	Rapid City, SD	1978	Annapolis, MD
1979	Portland, OR	1980	Austin, TX
1981	Gatlinburg, TN	1982	Vail, CO
1983	Stone Mountain, GA	1984	San Jose, CA
1985	Clarksville, IN	1986	Helena, MT
1987	Pittsburgh, PA	1988	Park City, UT
1989	Montgomery, AL	1990	Albuquerque, NM
1991	Albany, NY	1992	Fayetteville, AR

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 composed of approximately 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. j 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 contribution to the symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. Officers except for the treasurer may only succeed themselves for one additional term.

A number of three-member standing committees conduct the affairs of the organization. The lack of rigid requirements, routing, and the relativity relaxed overall functioning of the organization is what attracts many of the participants.

Meeting sites are chosen two or 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 protem of the Steering Committee.

The symposia are generally for two and one-half days, with a day-and-a-half for technical papers and a full-day for the field trip. The symposium usually begins on Wednesday morning. The field trip is usually Thursday, followed by the annual banquet that evening. The final technical session generally ends by noon on Friday.

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite a few examples, in Wyoming, 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 with 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 the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area which illustrated the planning, construction and maintenance of transportation systems. In 1985, the one day trip illustrated new highway construction procedures int he greater Louisville area. The 1986 field trip was through the Rockies of recent interstate construction in the Boulder Batholith. The trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

At the technical sessions, case histories and state-of-the-art papers are most common with highly theoretical papers the exception. The papers presented at the technical sessions are published in the annual proceedings. Some of the more recent proceedings may be obtained from the Treasurer of the Symposium.

* Revised from the 41st Highway Geology Symposium Proceedings.

Highway Geology Symposium

STEERING COMMITTEE OFFICERS

Mr. Harry Moore - Chairman Geological Engineering Supervisor I Tennessee Department of Transportation Geotechnical Section, PO Box 58 Knoxville, TN 37901 (615) 594-6219 or (615) 933-6776	1994
Mr. Charles T. Janick - Vice Chairman Soils Engineer Pennsylvania Department of Transportation 1118 State Street Harrisburg, PA 17120 (707) 787-5404	1994
Mr. Earl Wright - Secretary Geotechnical Branch Kentucky Department of Highways Frankfort, KY 40622 (502) 564-2374	1993
Mr. Russell Glass, Treasurer N.C. DOT, Geotechnical Section Box 3279 Ashville, NC 27702 (704) 298-8599	1993

Highway Geology Symposium

H.G.S.
STEERING COMMITTEE MEMBERSHIP LIST

Name	Term Expires
Mr. David Bingham 3713 Lancelot Court Raleigh, N.C. 27604 Ph. (919)876-0416	1993
Mr. Vernon Bump Division of Engineering Dept.of Transportation Pierre, South Dakota 57501 Ph. (605)773-3401	1993
Mr. Richard Cross New York State Thruway Authority 200 Southern Blvd. P.O. Box 189 Albany, N.Y. 12201-0189 Ph. (518)471-4277	1994
Mr. John B. Gilmore Colorado Hwy. Dept. 4340 East Louisiana Denver, Colorado 80222 Ph. (303)757-9275	1992
Mr. Russell Glass N.C. D.O.T. P.O. Box 3279 Geotechnical Section Asheville, N.C. 27702 Ph. (704)298-7599	1993
Mr. Joseph A. Gutierrez Vulcan Materials Company P.O. Box 4195 Winston-Salem, N.C. 27105 Ph. (919)767-4600	1994
Mr. Richard Humphries Golder & Associates 3730 Chamblee Tucker Rd. Atlanta, Georgia 30341 Ph. (404)496-1893	1994

Mr. Jeffery Hynes 1991
Colorado Geological Survey
1313 Sherman St., Rm. 715
Denver, Colorado 80203
Ph. (303)866-3520

Mr. Charles T. Janick 1994
PA. Dept. of Transportation
1118 State Street
Harrisburg, Pennsyvvania 17120
Ph. (707)787-5404

Mr. Harry Ludowise 1992
6308 NE 12th Avenue
Vancouver, Washington 98665
Ph. (206)693-1617

Mr. Henry Mathis 1992
Manager, Geotechnical Branch
Kentucky Dept. of Highways
Frankfort, Kentucky 40622
Ph. (502)564-2374

Mr. Willard McCasland 1994
Oklahoma D.O.T.
200 N.E. 21st Street
Oklahoma City, Oklahoma 73105
Ph. (405)521-2677

Mr. Marvin McCauley 1994
CA. Dept. of Transportation
5900 Folsom Boulevard
Sacramento, California 95819
Ph. (916)739-2480

Mr. Verne McGuffey 1992
New York D.O.T.
Bldg. 7 State Campus
1220 Washington Avenue
Albany, New York 12232
Ph. (518)457-4710

Mr. Harry Moore 1994
Tennessee D.O.T.
P.O. Box 58
Knoxville, Tennessee 37901
Ph. (615)594-6219

Mr. Gary Riedl 1992
Wyoming, Hwy. Dept.
P.O. Box 1708
Cheyenne, Wyoming 82002-9019
Ph. (307)777-7450

Mr. Sam I. Thorton 1993
University of Arkansas
Dept. of Civil Engineering
Fayetteville, Arkansas 72701
Ph. (501)575-6024

Dr. Terry West 1994
Associate Professor
Earth & Atmos. Sci. Dept.
Purdue University
West Lafayette, Indiana 47907
Ph. (317)494-3296

Mr. W.A. Wisner 1993
Florida Dept. of Transportation
P.O. Box 1029
Gainesville, Florida 32602
Ph. (904)372-5304

Mr. Earl Wright 1993
Geotechnical Branch
Kentucky Dept. of Highways
Frankfort, Kentucky 40622
Ph. (502)564-2374

Mr. Terry Yarger 1994
Montana Dept. of Highways
2701 Prospect Avenue
Helena, Montana 59620
Ph. (406)444-6280

ARKANSAS

Arkansas Geological Commission, Vardelle Parham Geology Center, 3815 West Roosevelt Road, Little Rock, AR 72204. Phone 501-371-1488 or 663-9714.

HISTORICAL SEQUENCE OF ORGANIZATIONAL NAME:

First Survey (Owen's Survey), 1857-60
Second Survey (Reconstruction Survey), 1871-75
Branner Survey, 1887-93
Geological Survey of Arkansas, 1923-45
Division of Geology, Arkansas Resources and Development Commission, 1945-55
Arkansas Geological and Conservation Commission, 1955-63
Arkansas Geological Commission, 1963-present

NAMES AND TITLES OF ORGANIZATIONAL DIRECTORS AND DATES SERVED:

David Dale Owen, State Geologist, 1857-60
W. F. Roberts, Sr., State Geologist, 1871-73
George Haddock, State Geologist, 1873-74
William Hazeldine, State Geologist, January-June 1874
Arnold Syberg, State Geologist, June 1874-January 1875
John C. Branner, State Geologist, 1887-93
George C. Branner, State Geologist, 1923-42
Richard J. Anderson, Acting State Geologist, 1942-43
Joe W. Kimzey, State Geologist, 1943-45
Harold B. Foxhall, Director and State Geologist, 1945-51
Norman F. Williams, Director and State Geologist, 1951-55; Director and State Geologist, 1955-63; Director and State Geologist, 1963-present

Note: From 1907-23 The Professor of Geology, University of Arkansas, acted *ex officio* as part-time State Geologist. Office holders were A. H. Purdue, N. H. Drake, and G. H. Cady.

ARKANSAS GEOLOGICAL SURVEY

HISTORY

The beginning of a state geological survey in Arkansas was in 1857 and was known as the "First Survey" or "Owen's Survey." In 1857-58, the first geological survey of Arkansas was conducted by Dr. David Dale Owen. Two reports were published as a result of his work, the *First Geological Reconnaissance of Arkansas* and the *Second Geological Reconnaissance of Arkansas*. The first report was published in 1858 and then in 1859 the Legislature made an appropriation to continue the work. Dr. Owen died in 1860 and his brother, Richard Owen, and his assistant, Edward T. Cox, edited the second report and had it published in 1861. These two volumes remain as the foundation to the present day State Geological Survey of Arkansas. It should also be noted that this work was done at a cost to the state of $16,800.

For the next 10 years no geological surveys were conducted because of the Civil War and the period of reconstruction following the war. Activity resumed in 1871 when the State legislature enacted legislation to form a geological survey. W. F. Roberts was appointed as State Geologist, and Dr. George Haddock was hired as his assistant. At this time they started a geological reconnaissance survey in western Arkansas. From 1873 through 1874 Dr. George Haddock, William C. Hazeldine, and Arnold Syberg held the office of State Geologist. During the 4 years (1871-74), no official reports were published, only a 63-page pamphlet by George Haddock in 1873. This period is known as the "Second Survey" or "Reconstruction Survey." Dr. John C. Branner was quoted in an Arkansas Gazette article in 1919 about the surveys of Owen's time and the period in the early 1870's.

> It is to be noted regarding these surveys that they cover a period of seven years, that they cost the state $51,428, but that, with the exception of Dr. Owen's reports, the work was of no value.

It was not until 1887 that the State had another state geological survey. At that time Dr. John Casper Branner was appointed State Geologist and remained in that position until 1893. He was an outstanding geologist of his time and was highly regarded by his colleagues. It was the excitement of the possibility of gold and silver in Garland and Montgomery Counties that sparked interest in establishing a new geological survey. The fight was led by Colonel Elias W. Rector of Hot Springs, a member of the Lower House. Over this 7-year period, Dr. Branner's survey produced some monumental economic geology reports which led to a much better understanding of the overall geologic relationships within the state. As a result of his work, nineteen volumes of his reports were published. During this period, one of Dr. Branner's assistants was Herbert Hoover, who later became President of the United States. Since the main reason for the reestablishment of the survey was to ascertain the potential for gold and silver in western Arkansas, it was essential that this be one of the investigations conducted. Dr. Branner's staff made an evaluation of the gold and silver prospects open at that time and showed that at least in the case of the mines then open, there was no validity in claims being made about the gold possibilities. This so irritated some people in the State that funding for the geological survey was withdrawn, not to be reestablished until 1923.

From 1907 until 1923, the Legislature appropriated a small amount of money for the Department of Geology at the University of Arkansas to conduct geologic surveys. The geological work was to be conducted under the direction of a commission composed of the Governor, the President of the University, and the Commissioner of Mines, Manufactures, and Agriculture. The professor of geology at the University was to designate a small portion of his time to survey work. The work at this time was done by Professor A. H. Purdue, Professor A. A. Steel, and Dr. N. F. Drake. The most notable contributions published during this period were on the slates of Arkansas and the coal fields in the Arkansas Valley.

In 1923 the Geological Survey of Arkansas was once again established and has continued to the present. George C. Branner, son of John Branner, was appointed State Geologist and held that position until 1942. George Branner was not a geologist by training, and the amount of new work performed in the 18 years he headed the survey was only a fraction of what his father had accomplished in less than a third of the time. George Branner served as State Geologist until the beginning of World War II, when he served as a Colonel in the Army. During this period very little was added to the knowledge of the geology of the State. Richard J. Anderson served as Acting State Geologist in 1942-43 and Joe W. Kimzey was State Geologist from 1943 to 1945.

The end of the war brought about a reorganization of State government and placed the Geological Survey in the Arkansas Resources and Development Commission as its Division of Geology. This reorganization took place in 1945 and Harold B. Foxhall served as the Director and State Geologist until 1951. During this period, the Division of Geology experienced some growth. In 1947, a young geologist from Oklahoma came to work for the survey, his name was Norman F. (Bill) Williams.

In 1951, N. F. Williams was appointed Director and State Geologist of Arkansas and holds that position today. Since the reorganization after World War II, the survey has experienced several name changes and reorganizations. In 1955, under a reorganization, the geological survey was called the Arkansas Geological and Conservation Commission. In 1963, this Commission was designated as the Arkansas Geological Commission. In 1977, State government was reorganized into twelve major departments. The Arkansas Geological Commission was placed in the Department of Commerce. In 1983, the Legislature abolished the Department of Commerce and again the Arkansas Geological Commission became a separate entity in state government.

In the early years, the Survey was housed in the State Capitol Building. It was not until 1966 that it moved out of the Capitol to temporary quarters west of the Capitol while plans were being made for a dedicated building. In 1970, the Survey moved into its current building and named it the Vardelle Parham Geology Center in honor of Vardelle Parham, a long-time Chairman of the Arkansas Geological Commission.

In 1977, the State Land Survey Division was added to the Arkansas Geological Commission. Thus today, the Arkansas Geological Commission is made up of the Geology Division and the Land Survey Division. The number of employees has grown to 32, including 11 full-time geologists.

ORGANIZATION

The Arkansas Geological Commission has a staff of 32 and consists of the Geology and Land Survey Divisions. The agency has a Board of Commissioners consisting of seven members with contiguous terms of 7 years. They are appointed by the Governor with the advice and consent of the Senate, and each staggered district must be represented by membership on the Commission.

The Land Survey Division, created in 1973 as part of the Office of the State Land Commissioner, was transferred to the Arkansas Geological Commission in 1977. The State Surveyor serves under the authority, direction, and approval of the State Geologist. The Land Survey Division has an Advisory Board composed of seven members who assist the State Surveyor in developing policies and regulations to establish uniform standards for professional surveying and mapping methods in the state.

Geology Division

The primary purpose of the Arkansas Geological Commission is to increase the knowledge of the geology of the State and to stimulate the orderly development and utilization of the State's mineral resources. The Geology Division is organized into three sections: Administrative Services, Information Services, and Technical Services, all of which are under the direct supervision of the State Geologist.

The Administrative Services section consists of administrative, accounting, and secretarial services, and provides all administrative support of the agency. Specific activities include preparing budgets and operation plans, monitoring and processing expenditures, typing and reproducing letters and reports, maintaining an agency filing system, maintaining buildings, and purchasing and inventorying supplies and equipment.

The Information Services Section's primary function is the distribution of information prepared and maintained by the Geology Division. This is accomplished through four offices: Maps and Publications Sales, Geological Library, Print Shop, and Cartographic Information Center. Maps and Publications Sales office has available for sale all available U.S. Geological Survey topographic and planimetric maps of Arkansas and publications relating to the geology and hydrology of the State prepared by the Geology Division. This office also provides general information to the public and operates the agency's reception and telephone services. The Geological Library maintains more than 35,000 references relating to the geology of Arkansas, other states, and various parts of the world. This library is used extensively by Arkansas Geological Commission geologists, by other government agencies, universities, and the general public. It is also a repository for government documents prepared by the U.S. Geological Survey and the U.S. Bureau of Mines. The agency's print shop maintains an inventory of publications prepared by the Arkansas Geological Commission. The Arkansas Affiliate of the National Cartographic Information Center operates on a cooperative agreement between the U.S. Geological Survey and the Arkansas Geological Commission. The USGS provides a listing on microfiche of all cartographic data available for the United States, with particular emphasis on Arkansas.

The Technical Services Section is composed of the geologic staff and technical support personnel. Its primary responsibilities are (1) to encourage the orderly development of the State's mineral, oil and gas, and water resources; (2) to maintain current geologic and topographic map coverage of the State; (3) to study and report on the geologic factors affecting the State's environment; and (4) provide a public source of geologic information. Technical Services is divided into five major activities: Economic Geology, Environmental and Areal Geology, Hydrology and Subsurface Geology, Mineral Exploration and Lignite Investigation, and the Technical Support Group.

The Economic Geology and the Mineral Exploration and Lignite Investigation activities are responsible for the development of information on the mineral resources of the State. Included in this program is the agency's drilling operation, which is used extensively in evaluating mineral resources.

The Hydrology and Subsurface Geology activity is responsible for studies of the State's oil and gas potential, ground-water investigations, and geologic investigations of the subsurface derived primarily from examination of rock cuttings, core materials, and geophysical logs throughout the State.

The Environmental and Areal Geology activity is responsible for the development of surface geologic maps in the State and the study of geologic factors affecting the environment. A good geologic map is a prerequisite for most geologic programs; the search for oil and gas and other minerals, the protection of the environment, and the conservation of mineral and water resources. Therefore, the geologic mapping program is a continuing one. The Commission's environmental program is geared to deal with specific urban areas, the resolution of individual environmental problems, or geohazards. These studies can provide general information on the geology, help solve construction problems caused by geologic factors, evaluate flood hazards, ascertain problems on quantity and quality of water, and provide natural resource information.

Because many areas of traditional geologic interest have come to the forefront of the public eye, geologic educational programs have become a significant activity in recent years.

All of the above Technical Services activities are supported by a Technical Support Group, which consists of the Chemical Laboratory, Sample Library, and the Cartographic Section. The Chemical Laboratory is an important adjunct of the mineral resources program. Its primary purpose is to provide the staff with chemical analyses on the wide variety of mineral and rock samples collected for various projects. The results of these chemical tests are critical to the evaluation of mineral deposits. The Sample Library maintains cuttings and cores of selected wells in Arkansas. These samples are critical to the expansion of our knowledge of the subsurface geology, development of the State's oil and gas resources, development of mineral resources, and ground-water data. Currently, the repository contains cuttings for over 2,800 wells and more than 300,000 feet of drill hole core from selected areas in Arkansas. The samples are used extensively by the Arkansas Geological Commission staff and are frequently used by company geologists and graduate students in geology. The Cartography Section supports the Technical staff by displaying information from geologic projects on maps, charts, and figures for publication. All of the programs of the technical staff interact and are flexible to meet current trends.

In addition to the major activities in the Technical Services Section, the Agency has several ongoing cooperative projects with the Geology Division of the U.S. Geological Survey. In addition, the Arkansas Geological Commission has three cooperative programs with the Water Resources Division of the U.S. Geological Survey: the Ground-water Survey, Stream Gauging Program (surface water), and Water Quality Program. Each of these is a 50-50 cooperative program in which the costs are split but with the majority of the work done by USGS personnel using their equipment and facilities.

The Arkansas Geological Commission also has a 50-50 cooperative topographic mapping program with the Mid-Continent Mapping Center of the U.S. Geological Survey. These funds provide for the preparation and publication of topographic maps in Arkansas and for the revision of existing maps.

In addition to the ongoing cooperative programs, the Geological Commission has been involved in many short term cooperative projects with the USGS. Examples of these are the State Geologic Map of 1976, COGEOMAP project in the Ouachita Mountains of Arkansas and Oklahoma, the CUSMAP project in northern Arkansas and southern Missouri, and the Strategic/Critical Minerals Program. The Commission has enjoyed an excellent relationship with the USGS over the years and anticipates many more years of cooperation.

Land Survey Division

The Land Survey Division is divided into three sections: Administrative Section, Records Repository Section, and the Corner Restoration Section. The Administrative Section provides for the administration of the Division and management of the physical plant; it also plans, develops, and implements overall programs. And finally, the Administrative Section develops and prescribes the necessary land surveying standards in order to promote uniformity and quality in land surveying practices throughout the State. The Records Repository Section establishes, maintains, and provides safe storage facilities for survey data concerning all monuments established by the United States Public Land

Survey and other monuments placed by surveyors. It also furnishes, upon request, copies of records created and maintained by the division.

The Corner Restoration Section's objectives are to restore, maintain, and preserve the Land Survey monuments established by the United States Public Land Survey and to provide a sufficient number of geologic control stations to permit statewide use of the State Plane Coordinate System.

SIGNIFICANT LANDMARKS AND ACCOMPLISHMENTS

One of the principal activities of the Arkansas Geological Commission (AGC) is to provide information on the State's mineral resources to both prospective and existing industries. Many of these mineral resources have been found as a result of investigations by the staff of the AGC. Some of these have had a major impact on the economy of the State and Nation.

(1) Bauxite was first identified by Dr. John Branner, State Geologist, from a sample brought in by a local contractor, Ed Weigel, who was using it to pave roads near Little Rock. Eventually this led to the discovery of the largest bauxite deposit in the United States and has resulted in over 80 million tons of bauxite being mined in Arkansas since 1898.

(2) The barite deposits in Hot Spring County were described in a published report of the AGC about 10 years prior to the mining operations. Within another 10 years, the Chamberlain Creek barite deposit was furnishing over half the Nation's supply.

(3) The first successful gas well in the state was sited by Dr. John Branner.

(4) The mining of vanadium, chalk, and clay and the production of bromine has led to the development of several major industries in the State. Geological data compiled over the years by research and investigations of the AGC staff have been the primary source of information used in the development of these resources.

Another primary function of the Arkansas Geological Commission is the preparation and updating of geologic maps and reports on Arkansas. Numerous maps and publications would qualify as significant contributions to the advancement of geologic information of the State, but only a few can be considered as landmarks.

- Geologic Map of Arkansas, 1929.
- Geologic Map of Arkansas, 1976.
- First Geological Reconnaissance of Arkansas.
- Second Geological Reconnaissance of Arkansas.
- The 19 reports published under the direction of Dr. John Branner (1887-93).

MAJOR PERSONALITY FEATURES

The classic work of David Dale Owen was a fine beginning for geologic studies in the State. John Casper Branner's great ability as a geologist and organizer was a giant step forward. Branner had on his staff R. A. F. Penrose and J. Francis Williams as well as Herbert Hoover and others who made substantial contributions to our understanding of the geology of the state and an appreciation of its mineral potential.

In the 1960's and 1970's Hugh Dinsmore Miser had a great influence on the work of the AGC staff, especially in the Ouachita Mountains. It was also through the influence of Mr. Miser that Charles Milton started his classic studies of the mineralogy of igneous rocks within the State, a study which is continuing.

FUTURE PROJECTIONS

The Arkansas Geological Commission plans to continue making available the best geologic information of the State of Arkansas with the funds appropriated. As always the geologic map is one of the best products to display and disseminate this information. Currently, the AGC is involved in COGEOMAP and CUSMAP projects, each of which have an emphasis on geologic mapping. Another area that will be expanded is the conversion of information in paper files to a computerized data base system.

Over the years the AGC has collected over 300,000 feet of core and cuttings from approximately 2,800 wells in the state. It is our goal to provide a facility to house and conduct research for these materials.

THE CRITICAL STATE OF DEBRIS FLOW

P. S. LIN, Professor, Department of Civil Engineering, National Chung-Hsing University, Taichung, Taiwan, R.O.C.
and
C. W. LOVELL, Professor of Civil Engineering, Purdue University, West Lafayette, Indiana, U.S.A.

ABSTRACT

In view of variation of the composition of lateritic gravelly deposits in western Taiwan, several factors, such as volume ratio of coarsely-grained material to finely-grained material, maximum aggregate size, and degree of saturation were analyzed by an experimental program to determine the parameters greatly influencing the strength of lateritic gravels. This study has established a model for in situ shear strength of lateritic gravels predicted by laboratory results. Further, the critical state of debris flow has been defined in terms of the angle of the slope and certain soil parameters and geometric factors.

INTRODUCTION

Pleistocene stratigraphy is present in large areas of Taiwan with an abundance of gravelly deposits which occur frequently throughout the tropics and subtropics. Most of these areas are located to the west of the Foothill geologic province of Taiwan. Some of the political, economical, and cultural centers of Taiwan are located in these areas, which are then vital for various construction activities. These areas are covered with lateritic gravelly deposits of depths of approximately 10 to 200 meters. A study was conducted by Hung et al. in 1987 on the engineering properties of different proportions of gravels to soils. In 1983, Cheng (Ref. 2) and Wu (Ref. 3) performed experiments on the effects of fine-grained material proportions and water content on the strength and other properties of compacted lateritic gravels. These studies, however, emphasized the properties of laboratory compacted materials rather than in situ behavior. The structure of this composite soil is generally unstable due to loosely-formed gravel aggregates with a sand-mud mixture between them. The erosion by rainfall on this unstable structure, especially on a steep slope, brings about landslides, which have caused great floods and damage. Therefore, study of the engineering properties of the gravel deposits is both necessary and important.

SIMULATION MODELS

The first part of this paper applies modified and parallel gradation models to constitute laboratory test specimens for the study of the engineering properties of lateritic gravels. These two gradation models, for which the maximum grain sizes are reduced to 5.08 cm, 2.54 cm and 1.27 cm for the purpose of laboratory tests (the maximum grain sizes are 30 cm), are related to the physical properties of field lateritic gravel. The first model shown in Fig. 1 is called a modified model. Its gradation is obtained when the sizes retained on the #4 sieve are adjusted to the maximum sizes of 5.08, 2.54, and 1.27 cm, while those portions passing through the sieve are kept at the same sizes as in the field. The second model, shown in Fig. 2, is called the parallel model. This model shifts the gradation curves parallel to the original one in

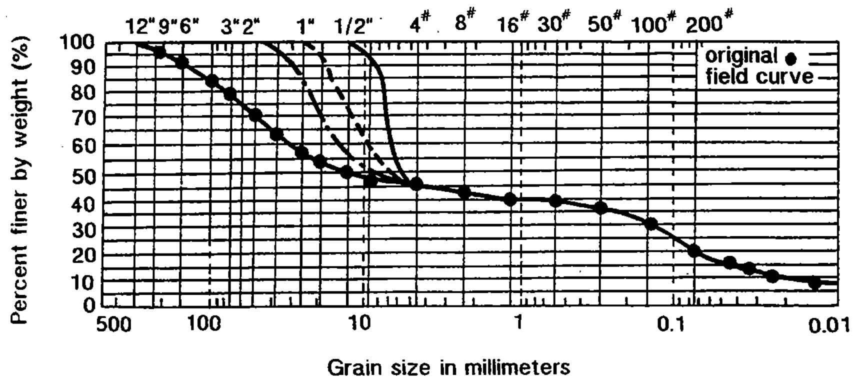


Fig. 1. Grain size distribution of the modified model

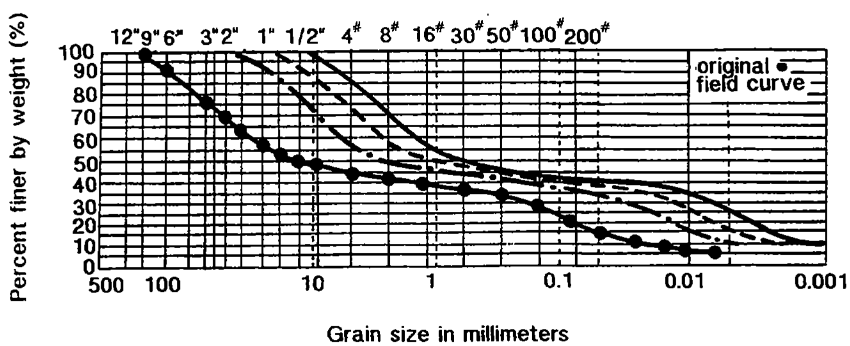


Fig. 2. Grain size distribution of the parallel model

the field, while keeping the maximum sizes as 5.08, 2.54 and 1.27 cm.

In addition, the elastic moduli are also estimated by the differential scheme of composite materials in different proportions of laterite to gravels. The experimental results (Ref. 4) show that the modified (M) model has a better prediction for both one-dimensional compressibility and shear strength properties; whereas the parallel (P) one is an effective model for the permeability characteristics. The compression index and rebound index decrease when the maximum grain size increases; whereas the coefficient of permeability and the friction angle, ϕ, ϕ' (for total and effective stress respectively) tend to increase with the increase in maximum grain size.

From Table 1, it is observed that the static moduli of elasticity by the differential scheme analysis are larger than those observed from the experiments. As for dynamic moduli of elasticity, both values are very close to each other, indicating that the dynamic properties can be well predicated by the differential scheme. Generally speaking, the elastic moduli predicated by the differential scheme are in good agreement with the experimental observations.

ENGINEERING PROPERTIES OF LATERITIC GRAVELS

In order to study the engineering properties of lateritic gravels, field measurements, such as the direct-shear test and the plate-load test were performed at Linkou Tableland. The results of the above tests are shown in Fig. 3 and Fig. 4, respectively.

Table 1. The results of experimental observations and of theoretical prediction (composite materials)

Sample No.	2-phase composites			3-phase composites
	V20-1	V30-1	V40-1	V20-2
Gradation (dry weight) (g) 2.54 cm-1.27 cm	432	500	916	150
Gradation (dry weight) (g) 1.27 cm-#4	377	717	706	250
Gradation (dry weight) (g) Graywacke	--	--	--	325
Gradation (dry weight) (g) passing #4	2,247	1,968	1,687	2,247
Wet density, ρ_m (g/cm^3)	2.009	2.074	2.113	1.968
Dry density, ρ_d (g/cm^3)	1.831	1.908	1.983	1.781
Volume of remained on #4 (%)	19.99	29.93	40.07	19.88
Passing #4 wet density (g/cm^3)	1.895	1.899	1.903	1.892
Passing #4 dry density (g/cm^3)	1.683	1.683	1.687	1.681
Passing #4 water content (%)	12.58	12.81	12.81	12.58
Static elastic modulus, E_s (Kg/cm^2) σ_3 = 1(Kg/cm^2) experimental result	429	397	547	364
Static elastic modulus, E_s (Kg/cm^2) σ_3 = 1(Kg/cm^2) theoretical result	502	657	895	462
Static elastic modulus, E_s (Kg/cm^2) σ_3 = 2(Kg/cm^2) experimental result	821	815	859.5	752
Static elastic modulus, E_s (Kg/cm^2) σ_3 = 2(Kg/cm^2) theoretical result	857	1,120	1,524	767
Static elastic modulus, E_s (Kg/cm^2) σ_3 = 4(Kg/cm^2) experimental result	962	925	--	847
Static elastic modulus, E_s (Kg/cm^2) σ_3 = 4(Kg/cm^2) theoretical result	976	1,277	1,737	867
Dynamic elastic modulus, E_d (Kg/cm^2) experimental result	18,657	23,459	30,028	13,267
Dynamic elastic modulus, E_d (Kg/cm^2) theoretical result	17,585	22,339	29,149	14,929

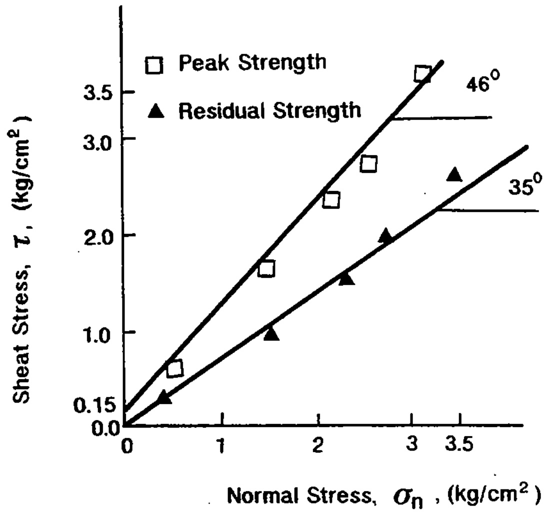


Fig. 3. Shear stress versus normal stress for the insitu direct-shear test at Linkou site

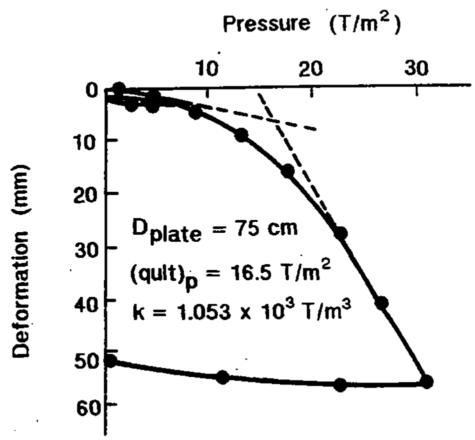


Fig. 4. Pressure versus deformation for the plate-load test at Linkou site

The relationships of the engineering properties between the laboratory and the in situ gravels were also studied. The results show that the properties of in situ materials can be predicted from the laboratory tests.

The relatively flat surfaces of the lateritic deposits has become a part of a newly developing area at Linkou. However, heavy rainfall often causes great damage as a result of landslides and soil erosion. The previous results have been applied to the gravelly deposit excavation and bracing, based on the laboratory and field properties of lateritic gravels at Linkou Tableland.

In view of gradually diminishing resources of river aggregates, this study also included an examination of gravelly deposits to determine their possibilities as an alternative for various engineering works (Ref. 5). Therefore, twenty open pits were excavated and investigated in the area to find the in situ unit weights, specific gravities and grain size distribution curves. Other general engineering properties, such as compressive strength, moduli of elasticity, Poisson's ratios...etc., are also obtained to provide information for aggregate resources evaluation. From the test results, as shown in Table 2, it is found that the gravelly deposits in the Linkou Tableland are not good for concrete aggregates. However, they are suitable for subbase or base course aggregates.

A MODEL FOR SHEAR STRENGTH OF LATERITIC GRAVELS

In view of variation of the composition of lateritic gravelly deposits in Western Taiwan, Lin (Ref. 6) selected several factors,

Table 2. The engineering properties of lateritic gravels

Proper-ties	Wet unit weight (T/m)	Compressive strength (kgf/cm)	Modulus of elasticity x10 (kgf/cm)	Poisson's ratio
1	2.098	1200	4.11	0.185
2	2.211	680	8.06	0.366
3	2.159	480	3.46	0.279
•	2.009	160	3.01	0.08
•	2.114	320	4.02	0.188
•	2.320	232	7.09	0.104
•	•	•	•	•
18	1.939	1334	3.06	0.15
19	2.084	2000	5.97	0.56
20	2.011	480	3.64	0.02

such as maximum aggregate size, void ratio and degree of saturation for an experimental program to determine the parameters greatly influencing the shear strength of lateritic gravels. This study has established a model for in situ shear strength of lateritic gravels predicted from laboratory results. The in situ peak strength angle, ϕ_p, can be predicted by Eq. 1.

$$\phi_p = 77.05 - 0.17\ dmax - 77.51e - 8.75\ S \qquad (1)$$

where dmax, e and S are in situ maximum aggregate size, void ratio and degree of saturation, respectively. Field tests and laboratory tests on compacted samples were also carried out to check the model prediction. Testing results and model prediction (of ϕ_p) are shown in Table 3. It is found that the in situ shear strength parameter, ϕ_p, predicted by the model is close to that obtained by in situ direct-shear tests.

The in situ shear strength of lateritic gravels can also be predicted by a model (Eq. 2) developed for rockfill by Barton (Ref. 7).

$$\phi' = R \log (Sc/\sigma'_n) + \phi r \qquad (2)$$

where ϕ' is the angle of internal friction, Sc is the intrinsic compressive strength, σ'_n is the normal stress, ϕr is the residual strength angle, and R is the regression constant. Cheng (1988, Ref. 8) modified Eq. 2 to predict the strength angle for lateritic gravels by Eq. (3)

$$\phi' - \phi r = R \log K + R \log (d_m^{\frac{1}{2}}/\epsilon) \qquad (3)$$

where dm is the mean aggregate size, ϵ is the compressive strain, and R and K are regression constants. Results of in situ direct-

Table 3. The results of in situ direct-shear test and of model prediction

	τ_p (kg/cm^2)	σ_p (kg/cm^2)	$\frac{\tau_p}{\sigma_p}$	d_{max} (cm)	e	S	ϕ_p, degree	
							in situ direct shear test	model prediction
Sample no. 1	1.651	1.41	1.171	30.5	0.273	0.69	46	45
Sample no. 2	2.343	2.13	1.100	30.5	0.270	0.67		

shear tests prove that the in situ shear strength can be well predicted by the above models.

THE CRITICAL STATE OF DEBRIS FLOW

On August 9th-10th, 1982, high intensity rainfall (365 mm of rain in two days) brought devastation in the form of landslides, debris flows, and floods at Linkou and its vicinity. Many houses and farms were destroyed and several people were killed. After that heavy rainfall, surface erosion of natural slopes was widely observed. There were also several slumps in the Linkou industrial area which was founded on the compacted lateritic soils.
Generally speaking, the infiltration of rainwater into the slope increases the degree of saturation and reduces the shear strength of the soil. Water may also reduce the cementation of gravels and remove the capillary forces within the tension zone near the surface of the slope. A study on the critical state of debris flow in lateritic gravelly deposits is thus important and has been performed.

Takahashi (Ref. 9) summarized the processes of debris flow initiation as shown in Fig. 5. In case (a), a comparably dense and only slightly cohesive sediment layer has been saturated and a surface water flow appears when a part of the layer becomes unstable enough to begin massive motion. There is sufficient water to fill the increased void spaces, and therefore, one can consider that the beginning of massive motion is, at the same time, the initiation of debris flow. In case (b), a seepage flow occurs in

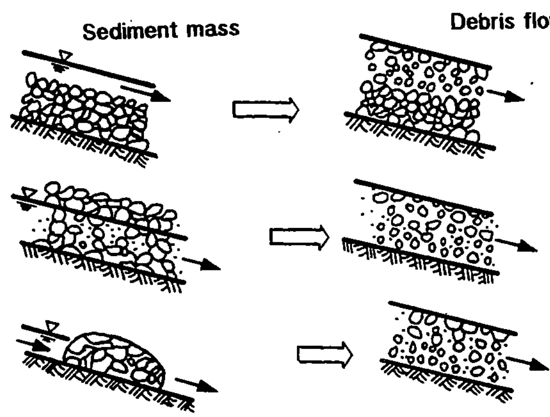


Fig. 5. Processes of debris flow initiation (after Takahashi, 1985)

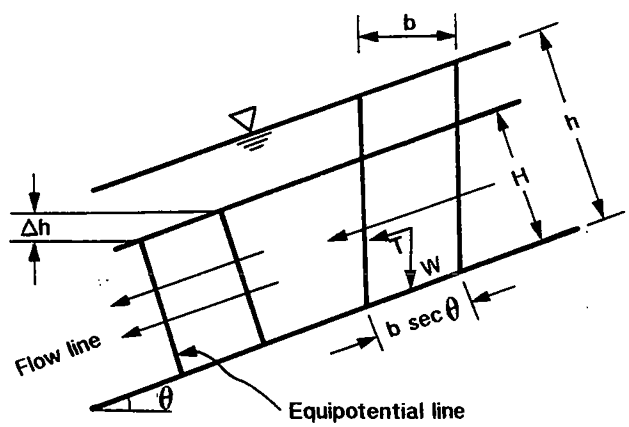


Fig. 6. Schematic diagram for the influence of seepage force

a very porous sediment layer when instability prevails in the whole layer, and its structure is collapsed. In case (c), water is continuously supplied to a moving block produced by a landslide. Meanwhile, the block will gradually deform and break down into a debris flow. If the length of the slope is inadequate, the motion will cease before the block transforms into a debris flow. Cases (a) and (c) would be the most common processes in reality. A theoretical analysis developed for the criteria of occurrence of various features of sediment motion was also given in Ref. 9.

Fong (Ref. 10) considered the influence of seepage force for case (a). In Fig. 6, H is the thickness of the sediment layer, h is the height of water table, θ is the slope angle, ϕ is the strength angle of the debris bed, γ_{sat} and γ_{sub} are respectively the saturated and submerged unit weights of the debris bed, γ_w is the unit weight of water, and n is the porosity of the debris bed. The hydraulic gradient along the oblique plane b sec θ is given by Eq. 4:

$$i = \Delta h/b \cdot \sec\theta = \sin\theta \tag{4}$$

The seepage force per unit volume is given by Eq. 5:

$$D = i\ \gamma_w = \gamma_w \sin\theta \tag{5}$$

The volume of debris bed per unit area is:

$$V = (b \cdot H \cdot \sec\theta \cdot 1)/(b \cdot \sec\theta \cdot 1) = H \tag{6}$$

The seepage force per unit area is:

$$F = D \times V = \gamma_w\ H \cdot \sin\theta \tag{7}$$

The total weight per unit area is:

$$W = [(h-H)b\cdot\sec\theta\cdot 1\cdot\gamma_w + \gamma_{sub}\cdot b\cdot H\cdot\sec\theta\cdot 1]/(b\cdot\sec\theta\cdot 1)$$

$$= \gamma_{sat}\ H + \gamma_w(h-2H) \tag{8}$$

Let T' be the component of W, parallel to the oblique plane. Then,

$$T' = W\sin\theta = [\gamma_{sat}\cdot H + \gamma_w(h-2H)]\sin\theta \tag{9}$$

Therefore, the driving force,

$$T = F + T'$$

$$= [\gamma_{sat}\cdot H + \gamma_w(h-H)]\sin\theta \tag{10}$$

The effective shear resistance of the debris bed, R is:

$$R = (\gamma_{sat} - \gamma_w)\ H\cdot\cos\theta\cdot\tan\phi \tag{11}$$

The critical state of debris flow is given by Eq. 12:

$$\tan\theta \geq \frac{(\gamma_{sat}-\gamma_w)H}{\gamma_{sat}\cdot H+\gamma_w(h-H)}\tan\phi \tag{12}$$

Similarly, if the height of water table is below the ground surface of the sediment layer, the critical state of debris flow is given by Eq. 13:

$$\tan\theta \geq \frac{1}{1+[h\gamma_w/(\gamma_{sat}-\gamma_w)H]}\tan\phi \tag{13}$$

From the model test, Fong found that when the critical state of debris flow is reached, the sediment layer becomes unstable enough to begin massive motion.

Hung (Ref. 11) and Chen (Ref. 12) studied the prevention and control of landslides and flood disasters, and suggested that the most important thing in control for lateritic tablelands is to limit the development of erosion gullies. Slope surfaces of the gravel formation should also be protected from erosion. Runoff must be controlled. Debris flow from the erosion gullies should be checked by detention basins. Drainage systems both at the top and beneath the tableland should be properly designed and maintained.

CONCLUSIONS

Based on the fundamental information and the results from laboratory and in situ tests obtained in the past four years, conclusive results of this research program have been produced in the following five categories:

1) Gradation models for lateritic gravels. The modified gradation model has a better predictive ability for both one-dimensional compressibility and shear strength properties; whereas the parallel gradation one is a more effective model for the permeability characteristics.

2) Aggregate potential of lateritic gravels. The gravelly deposit is not good for concrete aggregate; however, it is suitable for subbase or base course aggregate.

3) Model for shear strength of lateritic gravels. The in situ strength angle, ϕ_p, can be predicted by Eq. 1 or Eq. 3, repeated as follows:

$$\phi_p = 77.05 - 0.17\ \mathrm{dmax} - 77.51e - 8.75\ S \tag{1}$$

$$\text{or } \phi' - \phi_r = R \log K + R \log (\mathrm{dm}^{1/3}/\epsilon) \tag{3}$$

4) Critical state of debris flow. The initiation of debris flow can be predicted by Eq. 12, where θ is the slope angle.

$$\tan\theta \geq \frac{(\gamma_{sat}-\gamma_w)H}{\gamma_{sat}H+\gamma_w(h-H)} \tan\phi \tag{12}$$

5) The most important thing in disaster prevention and control for lateritic tablelands is to limit the development of erosion gullies. Drainage systems both on top and beneath the tableland should be properly designed and maintained.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the helpful discussion with Professor J. J. Hung. The research was sponsored by National Science Council, Republic of China.

REFERENCES

1. Hung, J. J., et al. (1978), "Preliminary Study on Engineering Properties of Composite Soil", Bull. Coll. Eng., National Taiwan University, Taipei, Taiwan.

2. Cheng, W. L. and Chen, J. K. (1983), "The Study of Large Scale Lab Tests on Gravelly Laterite", Proc. Annu. Meet. Chin. Inst. Civ. Hydraul. Eng., Taipei, Taiwan.

3. Wu, W. K. (1983), The Influence of Water Content on the Strength Characteristics of Gravelly Laterite, Master Thesis, Dept. of Construction Engineering and Technology, National Taiwan Institute of Technology, Taipei, Taiwan.

4. Lin, P. S. (1986), "A Study on Engineering Properties of Compacted Lateritic Gravels", Journal of the Chinese Institute of Engineers, Vol. 9, No. 6.

5. Lin, P. S. and Chu, B. L. (1986), "The Engineering Properties of Lateritic of Gravels in Linkou Terrace", Annual Report, National Science Council, R.O.C.

6. Lin, K. R. (1988), "A Model for Shear Strength of Lateritic Gravels at Linkou Terrace", Master Thesis, Dept. of Civil Engineering, National Chung-Hsing University, Taichung, Taiwan.

7. Barton, N. and Kjaernsli, B. (1981), "Shear Strength of Rockfill", Journal of Geotechnical Engineering Division, ASCE, Vol. 107, No. GT7.

8. Cheng, S. Y. (1988), The Influence of Gradation on the Shear Strength Characteristics of Lateritic Gravels, Master Thesis,

Dept. of Civil Eng., National Chung-Hsing Univ., Taichung, Taiwan.

9. Takahashi, T. (1985), "Debris Flow: Its Mechanics and Hazard Mitigation", Proc. of the ROC-JAPAN Joint Seminar on Multiple Hazards Mitigation, National Taiwan Univ., Taipei, Taiwan, R.O.C.

10. Fong, S. Y. (1988), Preliminary Study on the Critical State of Debris Flow of Lateritic Gravels at Linkou Terrace, Master Thesis, Dept. of Civil Eng., National Chung-Hsing Univ., Taichung, Taiwan.

11. Hung, J. J., Kou, J. T. and Chen, R. H. (1985), "A Study on Floods and Landslides of Linkou Tableland and the Vicinity", Annual Report, National Science Council, R.O.C.

12. Chen, R. H. and Wang, C. L. (1985), "The Failure Mechanism of Lateritic Soil Slopes", Proc. of the ROC-JAPAN Joint Seminar on Multiple Hazard Mitigation, National Taiwan Univ., Taipei, Taiwan, R.O.C.

THE INFLUENCE OF POST-DEPOSITIONAL EFFECTS ON THE ENGINEERING PROPERTIES OF A MARINE CLAY

Kenneth L. McManis & Mysore Nataraj
Department of Civil Engineering
University of New Orleans

Abstract

The characteristic features of many soils reflect the post-depositional environment of the site. These effects can be very significant and even dominate the material properties of interest. A Pleistocene deposit located along Louisiana's gulf coast provides a classic representation of the post-depositional features found in an overconsolidated marine deposited clay. The development of the macro- and microstructure of the clays and the geologic conditions during the post-depositional periods are discussed.

Engineering tests on specimens from different depths revealed material that was sensitive to the size and methods of sampling employed. The test results conducted on different core sample sizes are compared with hand-cut samples and the in-situ vane shear test. The manor in which the macro or micro structure of the clays from different depths controlled the test results is reviewed. While the easily recognized features that govern the performance of the upper fissured clays can be identified, a microanalysis is required to explain the brittle-like response and sensitivity of the lower gray clay. The material properties in combination with different exploration techniques used can provided a significantly different description of these clays. Identification of the features developed during post-depositional conditions contribute to an understanding of the soil response. The importance of these geological conditions and their influence on the site evaluation is discussed.

Introduction

The characteristic features of overconsolidated and stiff soils reflect the post depositional environment of the site. These formations are known for their heterogeneous characteristics and complex soil structure. The engineering behavior of these deposits is often influenced by the structure produced through post-depositional environments. These effects can be very significant and even dominate the material properties of interest. A knowledge of the nature of these changes is of great value in understanding properties, interpreting the soil profile data, and in constructing geologic history (1,2).

The properties and sensitivity of the intact soil are produced by its microstructure. However, the macrostructure quite often controls the engineering behavior of the mass. The presence of joints, fissures, silt and sand seams, root holes, and other anomalies are common characteristics of stiff or overconsolidated soils and can greatly affect the strength or drainage characteristics of the site. In problems concerning stability, settlements, or drainage, the geotechnical investigation must carefully document the clay macrostructure.

There have been numerous papers written on experiences with sites where stiff or overconsolidated clays were encountered. However, local experience often dictates engineering practice but is not necessarily reviewed in conventional geotechnical literature. Most offer little in the way of a comprehensive discussion. Overconsolidated or stiff soils are commonly encountered, yet the lack of a single resource that addresses their specific problems has been cited.

Sampling and Testing Program

The difficulties of sampling and testing stiff or overconsolidated soils require an appreciation for their formation. The TRB Committee on Soil and Rock Properties made a survey of engineers whose practice routinely involves stiff and/or overconsolidated soils (3). The survey requested specific information on current practice for these soils. Table 1 shows the response of state departments of transportation (DOT). The two most common features identified as being characteristic of these soils are fissures and the potential to swell. The site investigation and testing programs in most of the states reporting are similar in scope. There doesn't appear to be any systematic or extensive effort to include other investigative techniques beyond those commonly employed in other soils.

Other tests have been found to be useful in geotechnical investigations and can provide valuable assistance in establishing profile details for site studies. X-ray radiography, X-ray diffraction and electron microscopy (SEM) have been used successfully in many studies. X-ray radiography (ASTM D 4452) is particularly valuable for determining sample quality, heterogeneity and the distribution of anomalous features, failure mechanisms, etc. These techniques provide information on the composition and fabric of the micro- and macrostructure.

An extensive investigation was conducted in a Pleistocene formation at a site in southwest Louisiana. The profile included many end products from physical, chemical, and biological processes occurring in depositional and post-depositional environments. These geological factors were very important in evaluating the engineering properties of the soil.

Conventional sampling and testing methods were used in making the site investigation. This normally would include open-drive, 76 mm (3 inch) thin-wall tube samples. Larger tube samples (125 mm / 5 inch) and hand-cut samples were also taken in this study. The field tests - the standard penetration test (SPT) and the vane shear - that were conducted exceeded the norm for most conventional studies. X-ray radiography and SEM studies of the soil fabric and geologic features were used to complement and to further evaluate the engineering tests.

Table 1 - Survey of DOT Geotechnical Practice for Overconsolidated Stiff Soils
(Paul Mayne, 1992)

LOCATION	SOIL TYPE/GEOLOGY	LABORATORY TESTING PROGRAMS: (OCR) COMPRESS CONSOL.	LABORATORY STRENGTH	LAB SATURATE	OTHER LAB	FIELD AND IN SITU TESTING: SAMPLING METHODS	IN SITU TESTING	COMMENTS, REMARKS, GEOTECHNICAL PROBLEMS, FIELD INSTRUMENTATION
Alabama	Fissured OC	C	UU, UC	BP	No	Shelby	SPI	Problems with cut
Arkansas	Marine Clays	C	UC, TX	NA	NA	S,D, 94mm W	SPT, CPT MPMT	Near Surface Soils Critical - Change on exposure
Colorado	Weathered Claystone	E	UU, CU CD	BP	No	S,W, Contin	SPT, VST CPT	Swelling potential of stiff clays; Su for Caisson Design
Florida	Stiff Marls	NA	UC	NA	No	NA	SPT, CPT, DMT	NA
Idaho	Tertiary Lake Deposits; Residuum	C,E	DS CIUC UU, UC	BP, S	No	P, Ring, S	CPT, SPT (PMT,VST BS)	Cuts/Fills; Foundation Support; Stiff Clay properties variable Slope Stability; Volume Swell; Pile Driving; Residual Strength
Illinois	Tills and Desic. Alluvium	C (OCR>4)	UC, TX	No	No	S	NA	Stability for slickensided/fractured clays
Louisiana	Quaternary Pleistocene Terraces	NA	UC, TX	No	No	S Rotary	CPT	Slope Stability; Slickensided Clays
Minnesota	Glacial Tills; Decorah Shale	NA	CU, DS, UC	high BP	No	S, SS	SPT (PMT)	Slopes - weak layers/seams; Piles - Low adhesion; Bearing Cap.
Montana	Reworked Cretaceous Shales	C, E	UC, DS, TX	No	No	S	SPT, (insitu DS)	Swelling; Strength Loss with time in cut/fills of clay
New Mexico	Quaternary Deposits	C, (CRS)	DS, UU	No	No	S, Dennison	SPT, (CPT)	Use geology to aid in detecting CC materials
New York	Desic. Glacial Lake Varved Clays	C	UU, CU, CD	No	xrays Perm.	S, Driven, D	SPT, Large SPT	Long-term slope stability
Ohio	Glacial Tills	C	CU, DS, UC	BP	Multiax.	P, D	CPT, PMT	Landslide and Emb. Failures; Chain saw for block samples
Oklahoma	Residual Clay/Silt; Crustal Alluvium		UU, CU, DS	high BP	Ring Shear	3" & 5" S, P, D	SPT, CPT, DMT, PMT, VST	Slope Stability; Swelling; macrofabric important
Oregon	OC Silts & Clays; Resid.	C	UC	BP	CIUC	S, (Block)	SPT, (PMT)	Landslides with fissured/slickensided soils
Texas	OC Alluvial Clays - Gulf of Mexico Desiccated Clays	NA	TX	No	(SHANSEP)	S, Barrel	Texas Cone	Slickensided Clays with high PI Landslide Failures (Use inclinometers)

CRS = constant rate of strain
C = Consolidation
E = Expansion

S = Soaking
BP = Backpressure Saturation

D = Drive
S = Shelby
D = Denison
W = Wireline
P = Pitcher
OPS = Osterberg Piston

SPT = Standard Penetration Test
CPT = Cone Penetration Test
PMT = Pressuremeter Test
DMT = Flat Dilatometer Test
VST = Vane Shear Test
IBS = Iowa Borehore Shear
SW = Seismic Wave Measurement
PLT = Plate Load Test

General Site Description

The soils at the study site are typical of large areas within this and neighboring states. The area is poorly drained and has a flat to slightly rolling topography. Surface elevation is approximately 4.6 m (15 ft) NVGD. The soils originated from alluvial and coastal plains sediment of the Pleistocene Prairie Terrace. As a result of low sea periods, the material was subjected to thousands of years of consolidation, desiccation, oxidation and erosion. The surface soils of this formation have been severely weathered and are easily recognizable, even when they are overlain by more recent material.

The study area was being prepared for future residential development and will include waterfront lots. A pit was being excavated adjacent to the sampling site. Excavation extended to a depth of 8.25 m (27 ft). The excavated material was being used in embankment construction.

The soil profile consisted of four distinguishable strata, Figure 1. Samples from the top strata were composed of oxidized clays and were characteristic of material subjected to periods of extreme desiccation. They were very stiff, weathered and closely fissured. The different strata were easily distinguishable by their coloring and geologic features. Underneath the weathered clays, a uniform, medium-to-stiff, dark gray clay with fine silt lenses or pockets was found. The transition occurred at an approximate depth of 9 m (30 ft). An X-ray diffraction analysis of specimens from 4 m and 11 m (13 ft and 36 ft) depths were similar and identified the major clays present as smectite (probably montmorillonite) and kaolinite. Shells were encountered in the two lowest strata. The type of shells found were common to a marine bay environment.

Post Depositional Features

From the surface and to a depth of 8 m (27 ft), the soil was highly oxidized and characteristic of materials subjected to periods of extreme desiccation. These soils were composed of stiff and fissured clays. An examination of tested specimens from this zone also revealed many slickensides. The four zones or strata identified had unique features and could be readily distinguished from one another by their colors.

Within the top 2.75 m (9 ft), the soil was composed of a mottled, yellow and gray, silty clay. Iron oxide pellets 3.2 mm (1/8 inch) and larger were abundant in the upper portion of this zone, Figure 2. They were found less frequently at lower depths, but radiographs revealed an accumulation of iron oxide in small fissures or cracks and in patches within the sample as represented by darker areas. Vertical and irregular fissures appeared at the bottom of this zone.

From a depth of 2.7 m to 4.6 m (9 ft to 15 ft), the soil consisted of fissured, very stiff, red clay. In some areas burrowing organisms had formed a network of channels which had

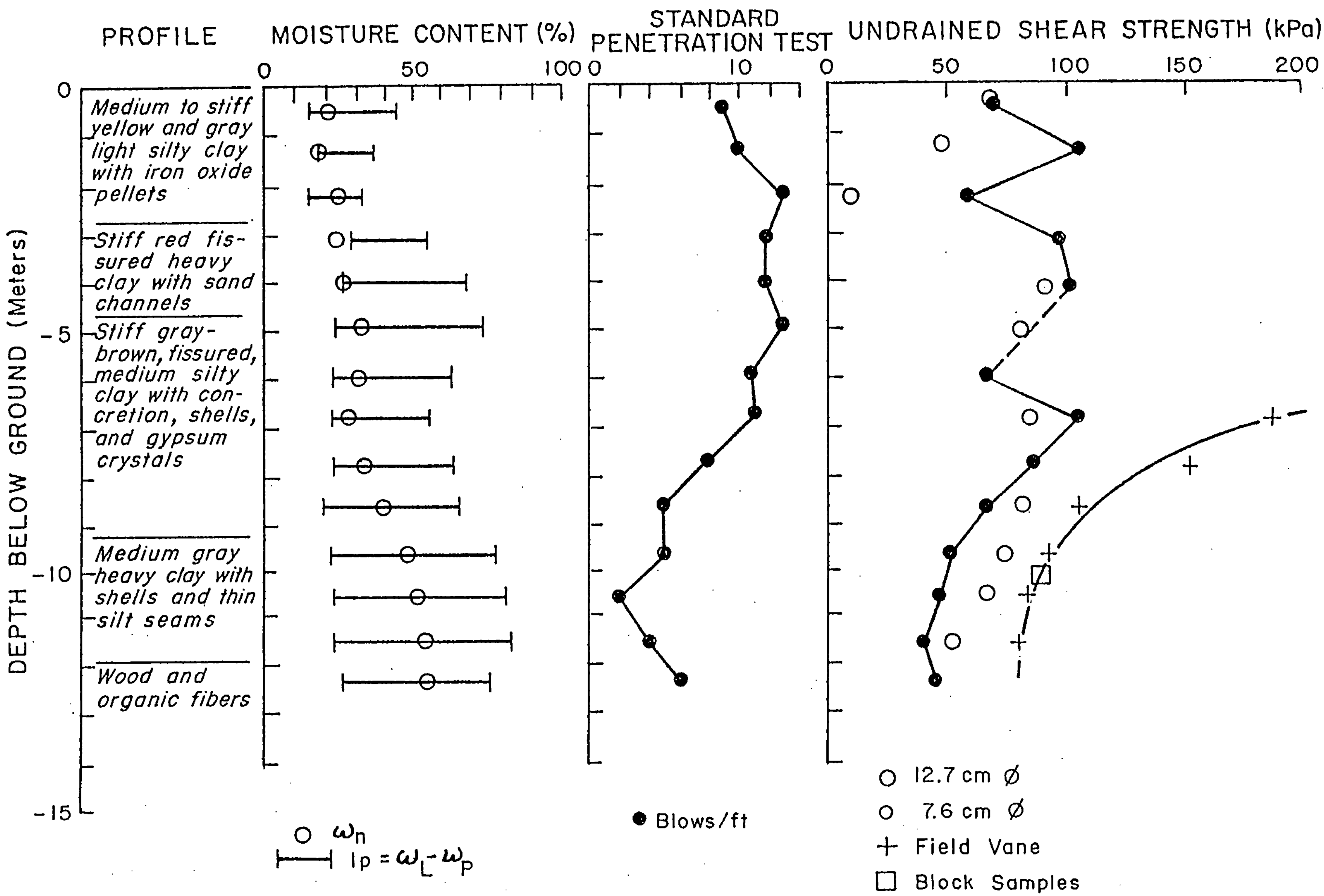


FIGURE 1. SUMMARY OF A TYPICAL SOIL PROFILE – PLEISTOCENE

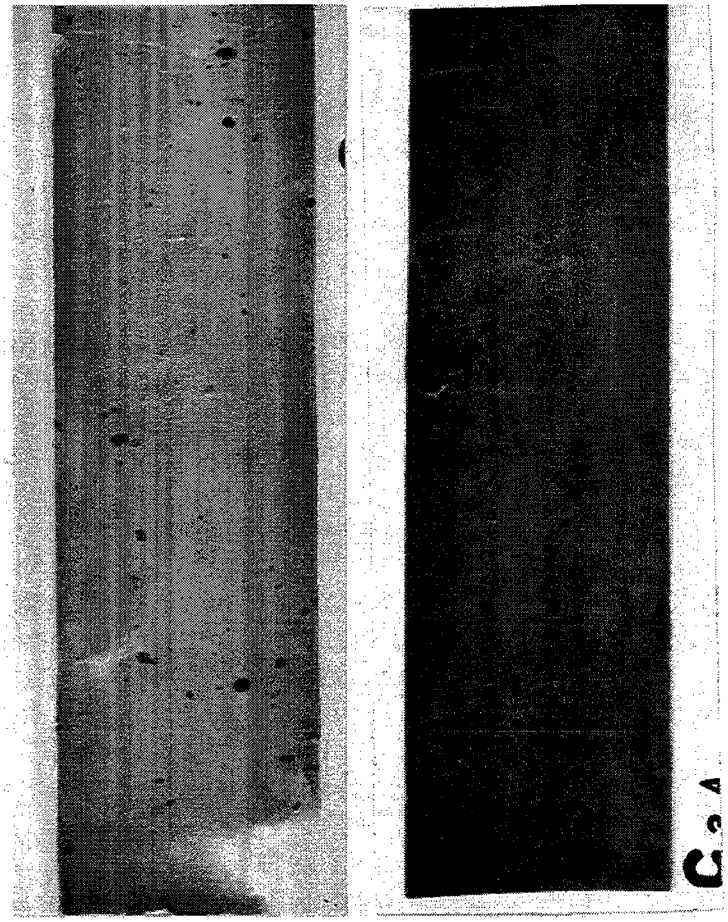


Figure 2. X-ray radiograph of core slices from Zone 1. The left radiograph is from a core near the surface (1 m) and contains many iron oxide pellets. The radiograph on the right represents the soil near the bottom of Zone 1 (2+ m). The dark areas represent locations where iron oxide has accumulated. Fissures and possible root holes in the lighter areas can be seen.

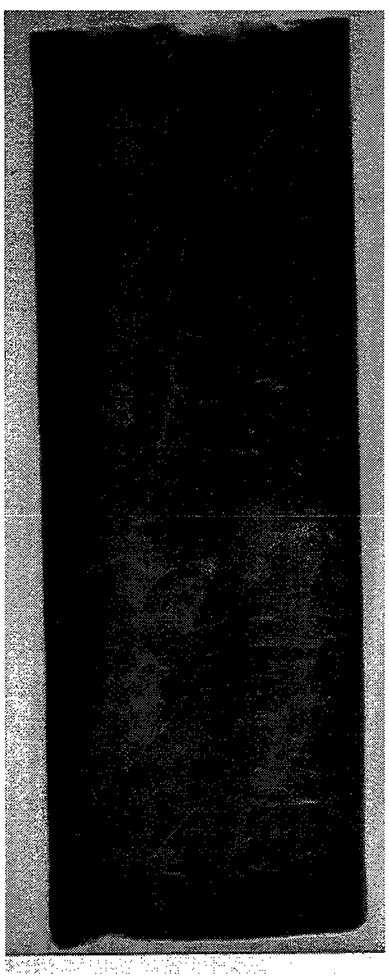


Figure 3. X-ray radiograph of the severely-fissured red clay from Zone 2 (2.7 m to 4.6 m).

been filled with a clean, white, medium sand. In other areas, red silt pockets were common. Many slickensides were observed in specimens tested from this zone. Radiographs of these samples present a complex network of intersecting, closely spaced fissures, many with a very shiny surface, Figure 3. The light areas in the centers of fissures indicate open cracks and channels that could transmit fluids.

Between 4.6 m and 9.1 m (15 ft and 30 ft), a fissured, stiff, gray-streaked, brown clay was found. Large gypsum crystals, approximately 76 mm (3 inch) in length, were found to be common at the 4.5 m (15 ft) depth in the adjacent excavated pit. Smaller crystals were found in some of the samples taken at this approximate depth. Gypsum forms from dissolved calcium and sulfate when sea water evaporates. During this period of receding sea, the smectite-rich clays were oxidized and developed shrinkage cracks. The gray streaks in the clay were confined to and followed the geometry of the fissures. The gray appeared to be a silt intrusion. Failure along a gray silt seam or plane was common in laboratory compression tests, Figure 4. During sampling a small natural slide in this zone occurred in the excavated pit near the sampling site. The slide consisted of fragmented blocks of clay, with failure occurring along gray fissures. Removal of the support provided by the surrounding soil had caused the cracks and fissures to open up, and led to the slope failure. The fissures in this zone, though abundant, appeared to be less frequent and were spaced further apart than those of the red clay above.

Dark vertical lines observed in several radiographs are an indication of iron-lined fissures. The occurrence of this feature has been identified as providing a drainage mechanism in other investigations (4). Failures in this zone were almost always along a fissure or included portions of a fissure. In examining the surface of the failure plane/fissure, it generally had a wet appearance.

Marine shells were encountered in this third zone and in the formation below. These included several different species; crossostrea (oyster), mercenaria, mulinia and most commonly the rangia cuneata (clam). Oysters are generally found in a more saline environment and are the reef-forming mollusk in Louisiana. Only a few oyster shells, showing signs of extensive reworking and containing borings from other organisms, were found. The final resting point of a shell does not necessarily reflect the original depositional environment. The rangia cuneata shells, however, were found to be abundant in the fourth zone and the lower areas of the third zone. This is a shallow-brackish-water clam. They do not form reefs but are found scattered as individual units within the soils of a lake or bay environment. These shells are common to a marine bay environment, the conditions under which the soils at this level were formed.

Under the three desiccated zones, a dark, gray clay of medium plasticity was found. It contained thin silt seams and isolated silt pockets. Clam shells were found in some areas floating within the clay matrix, Figure 5. Fragments of wood and organic fibers were found below 13 m (42 ft). The soil in this zone was more uniform in appearance and did not seem to have been as severely altered by post-depositional conditions. Engineering tests did

Figure 4. X-ray radiograph of a sliced core specimen that was tested in compression. Failure occurred along a fissure, but intersected a shell producing an increase in the compressive strength. The finer dark lines represent fissures lined with iron oxide.

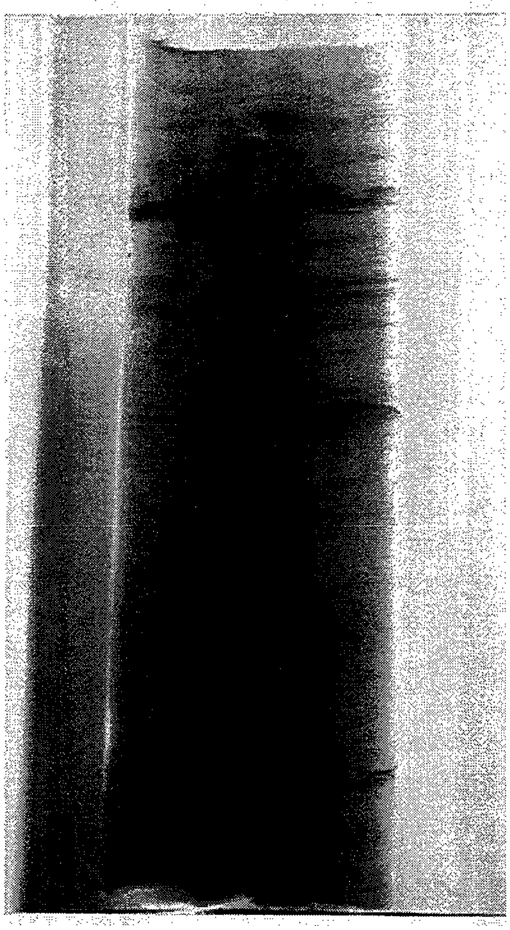


Figure 5. X-ray radiograph of a core sample from Zone 4 (below 13 m). Note the clam shells suspended in the clay matrix and horizontal layering.

indicate a brittle strength and a sensitivity with respect to sample size and quality. A microanalysis of the soil's fabric with the SEM showed concentrations of pyrite crystals interrupting the parallel bedding of the clays, Figure 6. Judging by the amount of disruption in the parallel bedding observed in the different electron micrographs, these pyrite crystals probably formed after the clay was deposited.

Engineering Analysis

Engineering tests on specimens from the different zones and depths revealed material that was sensitive to the size and methods of sampling employed, Figure 1. Test results were also influenced by the unique geologic features of each zone and the testing technique. The objectives of the testing program and the characteristics of the soil formation may not be compatible with methods employed in a conventional testing program. The character of the desiccated soils demonstrated a greater incompatibility between the different tests conducted than that seen in the plastic clays beneath the zone of weathering.

Zones 1, 2, and 3

The stiffness and low plastic consistency of the soils in this zone can be seen in the Atterberg tests, Figure 1. The natural moisture content is approximately that of its plastic limit. The undrained strength determined from laboratory tests conducted on specimens from these fissured soils gave a wide range of values. The major cause for the test scatter is attributable to the frequency and presence, or lack thereof, of fissures in a particular sample. The failure surface in many samples followed the irregular shape of the fissure geometry. Other features affecting the outcome of some tests included the presence of shells, concretions, silt pockets, etc. The intact soil between the fissures was very stiff. The undrained strength measured by the pocket penetrometer was as high as 150 kPa (3000 psf). However, when the samples were removed from the formation, the release in support allowed the fissures to separate and weaken the soil. The stiff, fissured samples deteriorated rapidly with extended storage time. Water seems to migrate along the surfaces of the fissures causing them to soften and swell.

There was some indication that the smaller, 75 mm (3 inch) samples had a tendency to increase the shear strength over that obtained from the 125 mm (5 inch) tube samples (5). This was attributed to the remolding of the fissures in sampling. This was supported further in that a number of the larger samples (125 mm tube and hand-cut) failed along well-defined natural fissure planes during attempts to trim test specimens. Also, tests conducted on remolded samples and specimens retrieved in the standard penetration test (SPT) generally, yielded strengths greater than that from the thin-wall "undisturbed" samples.

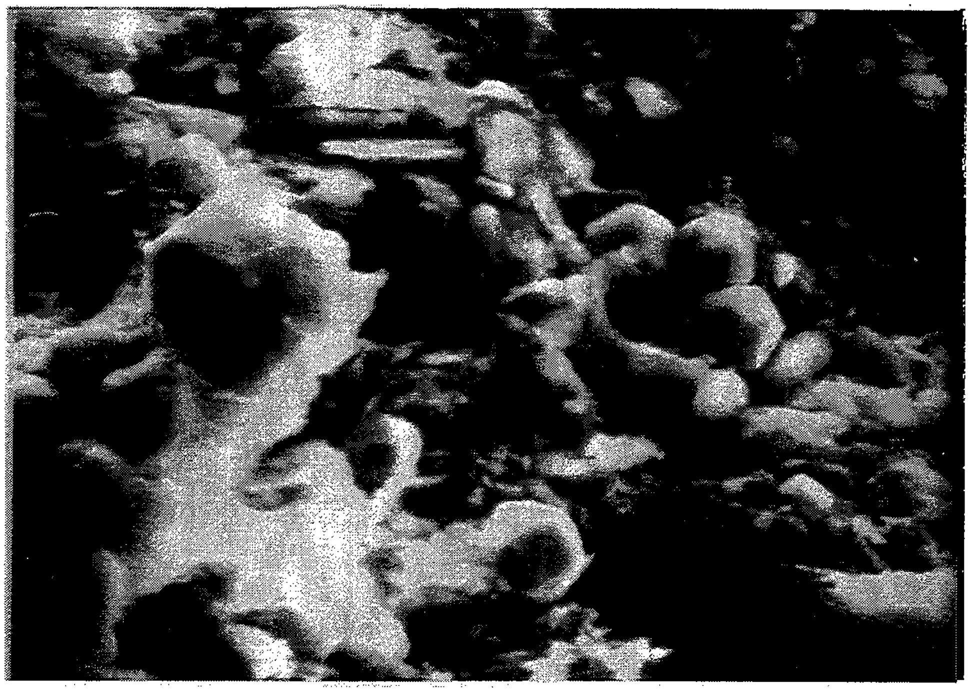


Figure 6. Electron micrograph showing pyrite crystals.

Zone 4

The results of the different tests - in-situ and laboratory - demonstrate better agreement in the gray plastic clay of this zone. The differences in undrained strength tests measured in the laboratory were consistent with the quality (size and sampling method) of the sampled specimens. The variation of strength with depth established by all sample types was similar. The hand-cut and larger samples provided higher strengths. Tests conducted on hand-cut block samples were equal to the in-place vane shear tests. The laboratory tests of the soil from this zone produced an abrupt or brittle failure. The strain corresponding to failure of the hand-cut was much smaller than that of the tube samples. The failure strain of the 125 mm (5 inch) tube samples was less than that of the 75 mm (3 inch) tube samples. A comparison of the remolded to the undisturbed, hand-cut sample strength is 5, i.e., a sensitive soil.

Clusters of tiny pyrite crystals were found to be abundant on the plane surfaces of clay flakes in the SEM investigation, Figure 6. They were also found lining the pore spaces and poorly developed cracks in the soil fabric. The pyrite crystals were identified as secondary mineral products. In effect, their crystal growth appeared to disrupt the laminated fabric produced by natural deposition and produces a more open fabric like that found in flocculated soils. This accounted for the soil's sensitivity.

Summary and Conclusions

The macrostructure, i.e., fissures and other anomalies, in combination with sample disturbance and stress release greatly influenced the results of the laboratory tests of the stiff clays in zones 1, 2, and 3. The range of test values obtained in this study for these desiccated soils is attributed to two factors: the remolding of fissures in the smaller, more proportionately disturbed samples; and the geometric orientation, spacing, and frequency of the fissures, i.e., the probability of fewer fissures occurring in smaller samples.

The growth of the pyrite crystals detected in the SEM microanalysis was found to modify and produce an irregular, open metastable fabric. This gave the soil its sensitivity and influenced the failure mechanism of the soil. The large tube samples (125 mm / 5 inch) and hand-cut samples were less disturbed and more representative of the in-place soil. The larger samples in all four soil zones provided the least sample disturbance and the most detailed information of each zone.

X-ray radiography provided an analysis of the heterogeneity and distribution of the anomalous features produced during deposition and post-deposition of the soils, thus furnishing information and clues on the micro- and macrostructure of the soil that might not otherwise be detected. It provided insight into the mechanisms that govern the permeable and strength of the soil and was also useful in evaluating the quality of the "undisturbed" samples. Radiography should be considered for routine use in site investigations involving soils with similar post-depositional features.

The post-depositional features that have developed at this site are common throughout the Louisiana Pleistocene Prairie Terrace and other soil formations of the Gulf Coastal Plains. They have a profound effect on the results of conventional site investigations. Interpreting the test results of a desiccated and weathered soil can be very confusing. Local practice has in many cases evolved from the experience of individuals. The performance of the project often depends on the compatibility of the engineering properties selected and the unique features of the site. Thus, it is important that the post-depositional and structural features be identified, and that their characteristics be fully understood. In order to accomplish these goals, the conventional site investigation should include tests and activities beyond those shown in Table 1.

REFERENCES

1. Mitchell, James K., **Fundamentals of Soil Behavior**, John Wiley and Sons, Inc., 1976.

2. Bennett, R.H.; Bryant, W.R.; and Hulbert, M.H.; Editors, **Microstructure of Fine-Grained Sediments: From Mud to Shale**, Springer-Verlag New York, Inc., New York, N.Y., 1990.

3. Mayne, Paul, Summary of survey on state-of-practice for stiff and/or overconsolidated soils, TRB Committee A2L02, January 1992.

4. Ferrel, R. Jr. and Carpenter, P., "Microtexture and Microchemistry of Clay-Rich Sediment," **Microstructure of Fine-Grained Sediment: From Mud to Shale**, Springer-Verlag New York, 1990.

5. McManis, K., and Arman, A., "Sampling and Testing In Stiff Crustal Clays," **Geotechnical Aspects of Stiff and Hard Clays**, Geotechnical Special Publication No. 2, ASCE, April 1986.

ABSTRACT

OVERVIEW OF THE COMPLEX HIGHWAY GEOLOGY IN WEST-CENTRAL ARKANSAS

By

Charles G. Stone

Arkansas Geological Commission
3815 West Roosevelt Road
Little Rock, Arkansas 72204

This is a brief summary of the complex highway geology involving Paleozoic rocks largely in west-central Arkansas. The geological overview begins in the orogenically distorted, thrust-faulted sequences of the Ouachita Mountains, proceeds northward to the mildly deformed transitional foreland belt of the Arkoma basin, to the relatively stable platform of the Boston Mountain plateau of the southern Ozarks.

About 50,000 feet of intricately folded, thrust-faulted, and mildly to severely sheared deep-marine rocks of Cambrian to Pennsylvanian age are exposed in the Ouachita Mountains. The rock types in decreasing order of abundance are: shale, sandstone, siltstone, chert-novaculite, limestone, conglomerate, and tuff. The interbedded character of the rocks, rapid changes in structure, variation in surficial weathering, quartz veins, and igneous dikes may cause some highway construction problems. Three sites selected for a discussion in the Ouachita Mountains are: (1) Interstate Highway #430 in western Little Rock (Pulaski County); (2) new east-west U.S. Highway (arterial bypass) at Hot Springs (Garland County); and (3) scenic Talimena Drive (Arkansas Highway #88 and Oklahoma Highway #1) on Rich Mountain (Polk County, AR and Le Flore County, OK).

In the southern Arkoma basin, there are thick, interbedded sequences of deep-to-shallow-marine Pennsylvanian sandstone and shale which, in places, are cut by thrust faults. The strata are usually inclined rather steeply along the crests of the anticlines and more gently tilted in the synclines. In the central and northern Arkoma basin, the progressively thinner Pennsylvanian formations are primarily shallow marine and deltaic deposits. They are composed mostly of sandstone, shale and contain some thin coal beds. The rocks typically occur in gently dipping, broad structural features that are displaced by an occasional normal fault. Interstate Highway #40 from Conway (Faulkner County) westward to near Fort Smith (Sebastian County) and adjoining highways have numerous exposures showing the bedrock types and deformational styles in the Arkoma basin. Locally, thick river alluvial and terrace deposits overlie the bedrock and further complicate the evaluation of highway construction problems.

The Mulberry fault, a normal fault downthrown to the south, with displacement locally of about 2,500 feet, separates the Arkoma basin from the Boston Mountains. Interbedded, very gently, often southward-dipping sequences of shallow marine and deltaic deposits composed of sandstone, shale, limestone, and minor chert and coal beds of Mississippian and Pennsylvanian age are displaced by a few normal faults in the Boston Mountains. The massive sandstones usually cap the rugged, mountainous plateau surfaces, and limestones and shales interbedded with sandstone typically are present in the deep, narrow valleys. Alternating rock types, locally deep weathering, and minor cavernous solution features can create many construction problems. Excavations along the scenic, newly relocated U.S. Highway #71 (Crawford and Washington Counties) provide exposures of these rocks. Several stops along this route will be examined on the Symposium field trip.

EVALUATION OF COAL REFUSE FOR ACCESS ROAD CONSTRUCTION AT AN ABANDONED MINE LANDS SITE, SOUTHWEST INDIANA

O'Hara, Kevin C. and T.R. West, Department of Earth and Atmospheric Sciences, Purdue University, West Lafayette, IN 47907-1397

ABSTRACT

An initial step in reclamation of an Abandoned Mined Lands site is the establishment of a reliable access road. Design and construction must consider axle loads of tractor-trailer shipments of material, and of heavy construction equipment. Commonly a thick base course and surface course are constructed from crushed stone to prevent subgrade failure. However, this may be too conservative and costly for low use, access roads.

Reclamation is currently underway at the Friar Tuck abandoned coal strip mine, Sullivan and Greene Counties, southwest Indiana. Initial cost estimates for the access road led to a search for less expensive, on-site construction materials. Extensive laboratory testing was performed to evaluate properties of coarse refuse when used as road construction material.

Two types of coal refuse were evaluated; Type I coal refuse, a highly weathered material found at shallow depths (0-2 feet) in the refuse deposits and Type II, unweathered material occurring at depths below three feet. Basic soil characteristics were determined. California Bearing Ratio (CBR) tests were preformed to evaluate bearing capacities and to determine which materials, if any, was a suitable structural component for the roadway. CBR testing included the typical soaked and non-soaked conditions. Additional CBR tests were performed after the materials were subjected to freezing and thawing cycles.

Results showed significant differences for the materials relative to CBR testing. Type II coal refuse experienced a 57.4 percent strength decrease from the dry to the saturated case, compared to a decrease of 23.4 percent for Type I material. Differences appear to be directly related to weathering history.

Comparison of CBR values and other parameters indicated that Type I refuse is likely to perform better as a road construction material. The extreme decrease in strength by Type II material suggests that excessive particle degradation occurs during saturation. Also, the presence of unweathered pyrite in Type II coal refuse indicates the possibility of acid leachate production during service.

Road construction using crushed stone was compared to that for a coal refuse/crushed stone combination. Design details for the proposed access road were used to calculate appropriate volumes and tonnages. Using a one mile length for the road, the partial replacement of stone by coal refuse resulted in an estimated $46,000 savings.

Results suggest that utilization of Type I coal refuse is a viable and economic alternative to the standard use of crushed stone for access road construction at coal reclamation sites.

INTRODUCTION

Background

The Friar Tuck Abandoned Mined Land (AML) site is located in adjacent portions of Sullivan and Greene Counties in southwest Indiana, approximately one mile north of the town of Dugger. It was the location of both surface and underground coal mining operations from 1929 to 1965 (Dombrouski, 1985). Because the site was mined before many of the more stringent reclamation laws were in effect, there was minimal regulation of coal refuse disposal practices and land reclamation. As a result, the site has developed into a serious environmental concern due to contamination of streams with acid mine drainage and sediment, and also due to the unattractive appearance of its many acres of barren coal refuse. In fact, the Friar Tuck site was ranked ninth on a list of thirtynine sites classified as major areas causing offsite environmental effects (Allen, 1978).

Previous and Ongoing Research

The first geologic study of the Friar Tuck Mine site was performed by R. P. Dombrouski, Purdue University, M.S. thesis, 1985. The purpose was to conduct an engineering geology investigation of the site and to assess the "quality and quantity of materials on the mine site, and to locate potential soil borrow areas for reclamation purposes" (Dombrouski, 1985). Much of the subsequent work at the site has utilized the valuable baseline data provided by Dombrouski's research.

Beginning in 1987 a joint study was undertaken by several staff members of the Indiana Geological Survey, Indiana University, and by T.R. West and graduate students at Purdue University, for the Indiana Department of Natural Resources, Division of Reclamation. The purpose was to study various erosional and hydrogeologic processes at the site and to develop the most feasible reclamation methods and designs. Three studies have been conducted on the engineering geology aspects of the Friar Tuck site by researchers at Purdue University. Project details by Peterson, Kuo, and Choi have been reported at several conferences on mine reclamation (Kuo and West, 1990; West et al., 1990; Choi and West, 1992) with the complete studies provided in the individual theses (Peterson, 1989; Kuo, 1990; and Choi, 1992).

The current study involved reclamation aspects of the project from summer 1989 through May 1991 involving plans and specifications for the third phase of reclamation. Construction work on this phase began October 1991 and is nearing completion in July 1992. A portion of this study became the subject of an M.S. thesis by the first author (O'Hara, 1992).

One task performed by Purdue personnel involved the selection and design of the route for an improved access road at the Friar Tuck site. An original mining roadway was still in use, but many sections had deteriorated recently owing to increased field activity. Two reasons exist for improving the onsite roadway. First, the anticipated increase in traffic from reclamation construction operations will require a high-quality, low maintenance route. Second, an agreement by the Division of Reclamation may allow a nearby municipality to dispose of treated sewage sludge at the site. This would benefit both parties, providing an economical method for sludge disposal and the addition of important nutrients to aid in the reclamation process. Based on these anticipated uses, it was concluded that the access road would require design and construction sufficient to accommodate axle loads typical of semi-trailer traffic.

The initial goal of the design process was to evaluate load-bearing capacities of materials present in the existing roadway, and also those which might be used as raw materials during construction of the new roadway. These materials included cinders, spoil, loess, glacial till, and coarse coal refuse. During this initial phase of evaluation an interest developed in the use of the coal refuse as a road construction material which resulted in the M.S. thesis study (O'Hara, 1992).

Statement of Problem

Various techniques are used to reclaim abandoned mined land sites and as each site is unique these techniques must be evaluated for each situation. An underlying goal for all sites is to achieve the best possible results with the available funding (Allen, 1978). A way to accomplish this goal is to evaluate the constituents at a site with an intention to optimize the contribution of each.

A first step in the reclamation process is to establish a reliable access route. Layout and design of this route involves not only consideration of present conditions but also the anticipated post-reclamation conditions and uses. For the Friar Tuck site the anticipated use is as a wildlife refuge (Harper, et al., 1989), with a minimal need for continued post-reclamation access. Some reclaimed sites, however, have been used for housing developments, fishing and boating areas, and parks (Allen, 1978), all of which require well-designed, high-quality access roads.

A major task of the roadway designer is to evaluate the bearing capacity of the materials along the proposed alignment. It must be decided what materials (preferably onsite) can and should be used as fill when grade modification is required (Oglesby, 1975). Many of the parameters used in this process are commonly based on empirical values related to soil classification systems. Unfortunately, many materials encountered on an abandoned mine site are atypical and the normal classifications are not directly applicable. Without a characterization of these materials, designers may choose to avoid their use during roadway construction. This leads to increased construction costs when more expensive, off-site materials are used.

Purpose of Study

The purpose of this study was to determine the suitability of coal refuse as a material for access road construction. Because of the large quantities of coal refuse present at abandoned mined land sites, utilization of this material could prove beneficial from both economical and reclamation aspects. In this work the objective was to characterize and quantify the parameters most important for roadway design. It is hoped that these results can be used in other studies to maximize quality and minimize costs of road construction of other reclamation projects.

ORIGIN, OCCURRENCE AND UTILIZATION OF COARSE COAL REFUSE

The Origin and Disposal of Coarse Coal Refuse

Coal seams, by nature, contain thin layers, or partings, of nonmarketable material such as shale, pyrite, clay, or siltstone. These materials may also occur in thicker stratigraphic units lying either above or below the coal. During mining operations these impurities are commonly mixed with the coal, requiring some form of post-mining separation. Before mining became the mechanized process of today, separation of the coal refuse, also called gob, was performed by hand, both underground and at the surface. Later technologies made use of the differences in specific gravity between the coal and the impurities to increase efficiency of the separation process (Collins and Miller, 1976).

Characteristics of coal refuse vary because of differences in mining methods, separation methods, and the composition of the non-coal constituents (NAS, 1975). Coarse refuse in general is typically a dark gray to black material composed of shale with fragments of coal, sandstone and clay. It is well graded and consists primarily of particles less than 4 inches in size (Collins and Miller, 1976). Several properties of coal refuse are affected by aging and weathering. Most notable is the change which occurs in the surface zone of a refuse pile exposed to oxygen and the leaching effects of water. This "zone of reaction" extends downward as much as 2 feet from the surface; it is in this zone where the production of acid from the oxidation and dissolution of pyritic material takes place (Allen, 1978). Table 1 provides a comparison of physical properties of coal, fresh refuse, and weathered refuse.

Table 1. Physical Characteristics of Friar Tuck Coal Refuse

Property	Type I Coal Refuse	Type II Coal Refuse
Components	85% carbonaceous shale and rock 15% coal	44% coal 25% carbonaceous shale and rock 17% pyritic shale and rock 14% fissile, hard shale
In Situ Density	41.0 lb/ft^3	61.5 lb/ft^3
Natural Moisture Content	35%	19%
Bulk Specific Gravity	1.64	1.92
Apparent Specific Gravity	1.83	2.32
Bulk Specific Gravity (Saturated, Surface-Dry)	1.77	2.17

Changes in laws regulating coal refuse disposal make it necessary to examine both past and present disposal practices. In the past, coal refuse was disposed of on the surface by conveyor belt, aerial tram, small rail-car systems, and trucks. Material was generally end dumped to form large, steep piles. Pile location was commonly based on availability of space rather than any rational or consistent design. Such haphazard methods resulted in piles of various shapes and sizes becoming scattered throughout virtually every portion of a mining site. The Friar Tuck Mine site is an excellent example of this practice. The lack of proper compaction produced piles which were more permeable and conducive to air and water transport. As a result oxidation, leaching, and erosion rates were high which contributed to both air and water pollution. For refuse with high carbon content the risk of spontaneous combustion was also increased.

The enactment of stringent state and federal regulations resulted in more carefully planned and implemented disposal of coarse coal refuse. Modern methods consist of controlled spreading, layering and compaction of the refuse into relatively homogeneous piles that are less susceptible to infiltration of air and water (Collins and Miller, 1976). Spreading and compacting are primarily performed with bulldozers, but hauling equipment also contributes to the compaction process. If necessary, terraces may be formed on the sideslopes of the refuse piles to prevent excessive runoff and erosion. The final and most costly step in present day reclamation operations is the coverage of the refuse pile with soil material and the subsequent establishment of vegetation (Doyle, 1976). The soil is usually placed in layers 8 to 12 inches thick that are effectively compacted by the traffic action of the scrapers, trucks, and bulldozers used. Thickness of soil cover averages 18 to 24 inches. Following soil placement, quantities of fertilizer and agricultural lime are applied and incorporated. Finally, a permanent seed mixture typically consisting of grasses and legumes is planted and covered with mulch to aid in germination. It is also common that different tree varieties are planted on the reclaimed refuse piles.

The Occurrence of Coal Refuse

Studies performed in 1968 showed that over 1.5 billion tons of coal refuse had accumulated in the mining regions of the United States (Vogely, 1968). Other studies concluded that almost 25 percent of all mined coal is rejected as coarse refuse (U.S. Department of the Interior, 1973). Given that over 400 million tons of coal were mined in 1969 and conservatively assuming constant production rates since that time, it is very possible that more than 3.5 billion tons of refuse exist at present.

The states of Illinois and Indiana were among the first to attempt to inventory the locations and quantities of refuse piles within their borders. A report prepared in 1975 by the Cooperative Wildlife Research Laboratory of Southern Illinois University identified 290 refuse piles present throughout that state. In Indiana, the Earth Satellite Corporation performed an aerial survey in 1971 and 1972 under the auspices of a grant from NASA and identified 149 refuse piles of two acres or larger located in 15 counties of the southwestern part of the state. An additional survey performed in 1978 by the Laboratory for Applications of Remote Sensing (LARS) at Purdue University revealed 1,520 acres of coarse refuse at 211 different sites (Allen, 1978).

Coal Refuse at the Friar Tuck Site

During its period of operation from 1929 to 1965, more than 5 million tons of coal were mined at the Friar Tuck site by both surface and underground operations. The coal was processed at an on-site cleaning and preparation plant which also served as a central facility for other mines in the area (Dombrouski, 1985). As a result, an extremely large volume of coal refuse was produced and disposed of at the site.

The majority of the coal refuse at Friar Tuck is located in four large piles, with smaller deposits found in at least three additional areas scattered throughout the site. The larger piles are referred to as the Southwest, Southeast, Northwest, and Northeast gob piles, which collectively cover more than 122 acres (Allen, 1978). Peterson (1989) estimated the total volume of these piles to be 2.4 million cubic yards. Each refuse pile possesses its own unique problems and challenges to effective reclamation, some of which include extensive gully erosion, acidic seeps, and excessively steep slopes.

Coal Refuse Utilization

The utilization of coal refuse as an engineering material is not a new concept. The mining industry in particular has traditionally used this material for the construction of haul roads, embankments and dikes for operating coal mines. Although there is no indication that the haul roads were designed or built according to any particular specifications, the prevalence of this practice indicates that the coal refuse must have performed rather well. Unfortunately, the extension of coal refuse utilization to other areas of construction has developed slowly in the United States. Great Britain, on the other hand, has made significant progress in this respect and has been a pioneer in the field (Maneval, 1974). Perhaps the relatively small size of that country has led to a more keen perception of the finite supply of natural construction materials. The result has been the creation of a situation more favorable for coal refuse utilization.

British Use of Coal Refuse

For many years burnt coal refuse, usually called "red dog", was used for the construction of railway and road embankments in Great Britain. The material was also permitted for use as subbase in road construction projects that were under the supervision of the Ministry of Transport (Maneval, 1974). Use of unburnt refuse, however, was not permitted because a common belief that the material was excessively susceptible to spontaneous combustion. A disastrous refuse bank slide at the village of Aberfan, South Wales in 1966, in which many people died, initiated a steady change of policy concerning unburnt coal refuse. Following the accident, a major increase in funding allowed the British National Coal Board (NCB) to devote efforts to the task of developing improved procedures for the construction of stable refuse piles, as well as finding new uses for unburnt coal refuse. Several test embankments were constructed which provided evidence that, with proper compaction procedures, coal refuse can perform satisfactorily as an engineering soil material.

Several basic guidelines concerning the use of coal refuse have been established during the course of the British research. When used in large fills the residual air voids should be less than 10 percent of the total volume, and less than 5 percent if the refuse has a high potential for spontaneous combustion (Maneval, 1974). It was also determined that weathered refuse has several advantages over fresh refuse. For example, because weathered refuse typically has a larger proportion of fines, it is possible to achieve lower permeability and thus reduce water infiltration. Another cited advantage is that weathered, stockpiled refuse typically has moisture contents closer to the optimum amount for compaction.

Research was also performed to evaluate the feasibility of preparing "artificial red dog" through controlled incineration of the coal refuse. The burnt product is sometimes desirable owing to its lack of combustion potential and a much lower rate of acidic leachate production. Although the incineration technique was found to be technically sound, it was economically impractical.

As a result of extensive testing and research performed in Great Britain, the original technical reservations concerning coal refuse have been eliminated in that country. In fact, the government has officially encouraged the use of coal refuse whenever possible in typical construction projects. But even with such a positive political climate, less than 10 percent of the yearly production of coal refuse is used for construction purposes (National Academy of Sciences, 1975).

U.S. Utilization of Coal Refuse

The thorough evaluation of coal refuse as a construction material has not been given as high a level of priority and government sponsorship in the United States as in Great Britain. Although nationwide acceptance and support are lacking, there are many documented cases of coal refuse utilization by individual states. "Red dog" has been used for open grading and road shoulders in Alabama, and has been approved by the Pennsylvania Department of Transportation for use as anti-skid material in asphalt surfacing (Collins and Miller, 1976). Unburnt coal refuse has been successfully used as embankment material in Illinois, Ohio, and Pennsylvania. Most notable was the use of 1.5 million cubic yards of anthracite refuse near Wilkes-Barre, Pennsylvania in 1973 for the construction of an embankment which forms the approach to a bridge crossing the Susquehanna River.

There have also been studies by various entities to determine the engineering characteristics of coal refuse. Pennsylvania State University, in particular, has contributed much information in this respect. Operation Anthracite Refuse, 1970, was one of the earliest studies to evaluate new ways of utilizing the material. Conclusions of the study show that anthracite refuse could be used as anti-skid material, aggregate for bituminous paving mixtures, and as a medium for plant growth (Collins and Miller, 1976).

In 1975, the Environmental Protection Agency conducted a study to determine the leachate characteristics of three different base course mixtures containing coal refuse. Base courses were used in the construction of a parking lot at the EPA Mine Drainage Control Field Site at Crown, West Virginia. The first mixture consisted of a 12 inch layer of 75 percent coal refuse and 25 percent fly ash. The second consisted of a 6 inch layer of fly ash-treated coal refuse overlain by a 6 inch layer of fly ash-treated coal refuse mixed with 5 percent hydrated lime. The third mixture was simply 15 inches of untreated coal refuse. All three base courses were capped by 3 inches of asphalt base and 1 inch of asphalt wearing surface.

A drainage collection system was constructed to evaluate the chemical characteristics of the leachate from each of the three base courses. The leachate from both mixtures using fly ash and/or hydrated lime had nearly neutral pH and contained no excessive amounts of harmful chemical constituents. The untreated base, however, produced a leachate with high acidity and excessive concentrations of some heavy metals. It was thus concluded that the addition of fly ash or hydrated lime to coal refuse is beneficial, if not necessary, when leachate quality is a major concern.

In 1976 the Federal Highway Administration published a report entitled "Availability of Mining Wastes and Their Potential for Use as Highway Material". Although the publication was not the product of an actual testing program, it was the first major government-sponsored report to provide a comprehensive survey of previous research. It contains specific sections pertaining to coal refuse as a highway construction material. The first section describes the attributes of coal refuse as an engineering material plus applications which are best suited. The second portion discusses problems associated with the use of coal refuse and suggests remedial measures for dealing with the problems.

Advantages of Coal Refuse

The large quantities and concentrations of coal refuse typically present in mining regions provide sources capable of supplying even the largest construction projects. In addition, because the refuse is usually found in piles, it can easily be excavated without forming unwanted surface depressions or pits. One of the useful properties of coal refuse is its well graded nature, providing optimum compaction into a dense, stable mass. It is also nonplastic, a feature that precludes many of the problems associated with expansion and contraction of fill material.

Disadvantages of Coal Refuse

One principal objection to the use of coal refuse in highway construction is the tendency for spontaneous combustion. Another problem has been the pyritic nature of the material, and the resulting production of acidic leachate. A third objectionable characteristic is the tendency for significant weathering and breakdown of fresh coal refuse. Because the constituents are typically softer materials such as siltstones and shales with a high clay content, exposure to various climatic extremes can create excessive degradation.

The problem of spontaneous combustion most easily can be solved through proper and efficient compaction of the coal refuse. By reducing the void ratio and the internal circulation of air, a corresponding reduction in the heat-producing oxidation of pyrite can be achieved. In addition, a surface cap composed of natural soil can act as an effective oxygen barrier. Acid production can also be reduced by this procedure, and the addition of lime and/or fly ash can serve to neutralize the leachate that is formed. The problem of weathering is more difficult to solve; in fact, rather than trying to prevent weathering of fresh refuse, it may be beneficial to permit an initial period prior to use during which a large percentage of the associated volume changes and alterations can occur (Collins and Miller, 1976). This may prevent many of the more serious post-construction effects of the weathering process.

Summary

Considerable research has focused on the possibility of widespread use of coal refuse in highway construction. Unfortunately, the negative characteristics associated with this material will likely prevent any significant increase in its utilization, at least in the near future. The continued availability of high-quality, natural aggregates at reasonable prices will also contribute to the reluctance to experiment with new materials.

As a result of these factors, it appears that coal refuse will be utilized under unique conditions and applied on an individual basis. Access road design for the Friar Tuck abandoned mine site is one such situation. Because the access road project is not governed by more stringent requirements of public roadways, there is a greater possibility that experimental materials can be utilized. However, the use of such material must be warranted both from a practical and economical point of view. Studies of the coal refuse at the Friar Tuck site were conducted with these considerations in mind. The objective was to determine the pertinent physical characteristics of the coal refuse and develop design recommendations for its use. Based on those recommendations it is possible to determine the overall feasibiity of coal refuse utilization to construct the Friar Tuck access road.

TESTING: PROCEDURES AND RESULTS

Introduction

The testing program to evaluate the coal refuse at Friar Tuck had two basic goals. The first was to determine the standard physical properties used to classify a soil material. These properties include grain size distribution, Atterberg limits, and material composition. All are useful because they provide an indication of the likely behavior of the coal refuse under various climatic and performance conditions. The second goal was to evaluate and quantify properties which are particularly important in the utilization of material for road construction. In situ bulk density, specific gravity, compacted density, and California Bearing Ratio values comprise these important parameters.

Collection of Sample Material

As was mentioned previously, there is typically, a difference between weathered and unweathered coal refuse; this difference is very apparent at the Friar Tuck site. In areas where the material was originally disposed of in large piles, a one to two foot thick weathered surface zone has developed. Below this depth, however, the coal refuse has remained in an unaltered condition. In other areas where the refuse was spread out as a thin (1 to 3 ft. thick) veneer over original soil, virtually the entire thickness has been transformed into the weathered variety. Both unweathered and weathered coal refuse occur in sufficient quantities to provide a convenient source of road construction material. In order to determine which of the two types would provide the best overall performance, each was evaluated separately in the testing program. Weathered material was described as Type I and the unweathered material as Type II. This designation is used to refer to these materials throughout this paper.

The Southeast Gob Pile was chosen as the source area for both Type I and Type II coal refuse. This particular pile has been exposed to weathering for over 40 years with minimal surface disturbance. The contrast between the weathered surface zone and the deeper unweathered material is likely the most distinct at this location. All samples were collected from a relatively flat area 20 feet square on the upper region of the pile. Selecting a flat area assured that the sample material was not affected by physical segregation that occurs on slopes during erosion.

For ease of transport and handling, samples of coal refuse were collected in 50-60 pound increments placed in heavy duty, trash compactor bags. Four bag samples were obtained for each of the two types of material. Ideally, it would have been best to obtain a single composite sample from each zone, from which all subsequent subsamples would be taken for testing. Lateral differences in material characteristics would thus be eliminated. Such a large sample, however, was not possible to collect because of handling difficulties and laboratory preparation restrictions.

Type I refuse was collected from the surface down to a depth of one foot. Type II refuse was collected from the interval between the depths of 4 feet and 5 feet.

The testing program was designed to include four individual trials of each material for each specific test. Four separate results should provide a meaningful range and representative average value. Since there were four bags of each type of material, one sample was taken from each bag for a particular test. This method made it possible to account for the lateral differences in material characteristics.

Physical Properties

Physical properties were measured to describe the nature of the coal refuse. These properties characterize the nature of the undisturbed, in situ material.

Description

Type I coal refuse is generally a dark gray material consisting predominantly of shale and coal fragments. It is moderately loose, easy to excavate by hand, and tends to be very dusty under most climatic conditions. The portion retained on the No. 4 U.S. Standard Sieve consisted of a mixture of approximately 85 percent by weight shale and 15 percent by weight coal. The shale fragments are subangular and range up to 2.5 inches in size. Coal fragments are typically smaller with a blocky shape.

Type II coal refuse is dark gray to black and also consists primarily of shale and coal. Because of the lack of weathering, four separate components of this material were identified by sorting. The most prominent constituent is coal, comprising 44 percent by weight. Carbonaceous shale and rock, subangular in form, are 25 percent. Pyritic shale and rock account for 17 percent, with the remaining 14 percent consisting of a fissile, hard shale. Maximum size of any of the various components is approximately 5 inches. Type II coal refuse is very compact and more difficult to excavate by hand than Type I.

In Situ Density

The in situ density of a material is important for construction purposes. By knowing both the original and compacted densities, volumes of the required borrow or fill can be calculated.

In situ densities were determined according to ASTM Method D-1556 (ASTM, 1990). Known as the sand cone method, this procedure involves excavating a small hole (0.1 to 0.2 cubic feet) in the material. The mass of dry, uniform sand required to fill the hole is measured and converted to volume through a calibration procedure. Using the dry weight of material removed, a dry density can then be calculated. The results of the in situ density tests are presented in Table 1.

Natural Moisture Content

Natural moisture content is another important characteristic affecting details of construction. Regarding compaction, it is essential to know how the existing moisture content compares to the optimum moisture. From this it is determined whether the material will need to be wetted or dried prior to the compaction. Moisture contents were measured in accordance with ASTM Method D2216. Results of the natural moisture content tests are shown in Table 1.

It is interesting to note that Type I refuse had a 35% moisture content yet generally displayed a very dry appearance. Type II refuse had a 19% moisture content with an obvious damp appearance. It is concluded that the contrast is due to the material's composition and its degree of weathering. Type I refuse contains a high percentage of weathered shale which is able to hold a much larger quantity of water while maintaining a dry surface appearance. Type II refuse, by contrast, has a much smaller percentage of shale which is unweathered and has a lower porosity. As a result, the lower moisture content material is able to provide a noticeably damp surface condition.

Specific Gravity

Specific gravity of a material is a measure of its solid density as compared to that of water. Specific gravity is typically not determined for conventional soil materials. For most soils, the mineral constituents commonly are those with specific gravities between 2.5 and 2.8 (Holtz and Kovacs, 1981). Therefore it is common to assume a value of 2.65 unless other factors make it necessary to be more exact. Coal refuse is obviously not a typical soil material, and its very low in situ density is indicative of a specific gravity much less than normal soils. Specific gravity tests were performed for both Type I and Type II refuse in accordance with ASTM Methods D854 and C127. Method D854 is a procedure which applies to material particles passing the No. 4 U.S. Standard seive; C127 is for those particles retained on the No. 4 seive.

Method D854 involves the use of a pycnometer, this being a volumetric flask with a minimum capacity of 100 ml. A soil sample weighing at least 25 grams or more (oven dried) is used for the test. The soil sample is placed in the pycnometer, and distilled water added to fill the flask about three-fourths full. Entrapped air is removed by either subjecting the pycnometer to a partial vacuum or by gently boiling the contents for at least 10 minutes. The boiling method was used as it was easier to accomplish with the available equipment. After removing the entrapped air, the pycnometer and its contents are cooled to room temperature. Distilled water is added to completely fill the pycnometer, which is then weighed. Specific gravity of the material is calculated using the following equation which relates the dry weight of the sample to the weight of water displaces in the pycnometer:

$$\text{Specific Gravity} = \frac{W_o}{[W_o + (W_a - W_b)]}$$

where W_o = weight of sample of oven dried soil, g.

W_a = weight of pycnometer filled with water, g.

W_b = weight of pycnometer filled with water and soil, g.

Method C127 is generally used to determine the bulk density and apparent density of coarse aggregates for construction, but is used in conjunction with D854 for soils containing a large range of particle sizes. The procedure involves soaking a sample of oven-dry material in water for a period of 24 ± 4 hours. The amount of sample needed depends upon the nominal maximum particle size, in inches. Type I refuse required a sample weight of 18 kilograms, whereas Type II refuse required 25 kilograms. Following the soaking period, the sample is rolled in an absorbent cloth to remove surface water films. The sample is then weighed both in air and while submersed in water, using a wire mesh container to support the material.

As previously indicated, specific gravity is determined by comparing the weight of the sample to the weight of water displaced. From this method, absorption is obtained by comparing the oven-dry weight and the saturated weight. Three basic equations are involved yielding apparent specific gravity, bulk specific gravity, and bulk specific gravity (saturated-surface-dried basis). One value may be more applicable than the others, depending on the conditions controlling the material in a given use. The equations are as follows:

$$\text{Apparent Specific Gravity} = \frac{A}{A-C}$$

$$\text{Bulk Specific Gravity} = \frac{A}{B-C}$$

$$\text{Bulk Specific Gravity (Saturated-Surface-Dry)} = \frac{B}{B-C}$$

where A = weight of oven-dry specimen, g.

B = weight of saturated-surface-dry specimen in air, g.

C = weight of saturated specimen in water, g.

The results of specific gravity tests are shown in Table 1. Type II coal refuse, because of its unweathered condition, shows higher values for all measures of specific gravity. Specific gravity values for particles smaller than the No. 4 sieve were especially high for Type II refuse. The presence of small, unweathered pyrite fragments probably contributed to these high values. For the anticipated use of the coal refuse at the Friar Tuck site, the bulk specific gravity value is the most applicable. This is because the material is not to become saturated; the important parameter is the amount of space occupied by a given weight of the material.

Properties for Material Classification

Several specific properties and index tests are used to classify soils. They are independent of the appearance, origin or nature of the material. The two fundamental characteristics are grain size distribution and Atterberg limits of the material. Using results from these two parameters, materials can be classified based on several classification systems.

Grain Size Analysis

The grain size distribution of a soil material is important because it provides a general indication of the engineering characteristics. This parameter supplies information on anticipated compacted densities of materials, and is commonly used to determine the most efficient compaction methods. The grain size analysis can also provide a good indication of the hydraulic conductivity to be expected.

The typical grain size analysis is obtained from a series of stacked sieves that separate the larger grain sizes, plus a hydrometer analysis to characterize the smaller silt and clay size components. In the current study, it was not considered essential to obtain the size distribution of this finer portion of the coal refuse and no hydrometer analysis was performed. Of more importance was the behavior of this finer portion, which can be determined by the Atterberg limits test.

When performing grain size analyses it is important to select the proper preparation procedure suited to the type of material being tested. As the coal refuse had a fairly high percentage of shale, it was anticipated that there would also be a corresponding high percentage of clay in the fine portion. ASTM Method D2217 was selected because it specifies a wet preparation procedure that provides optimum removal of clay particles adhering to the larger grains.

Four individual grain size analyses were performed on both Type I and Type II coal refuse. Based on results of these tests, there appeared to be no significant variation between lateral locations in the sample area. Composite curves derived by averaging individual tests are shown in Figures 1 and 2. It is apparent from the distributions that Types I and II differ basically in the overall sizes of their constituent particles. The curve for Type I refuse has shifted upward on the graph, reflecting the decrease in particle sizes caused by weathering. The curves indicate that both types of refuse consist of well-graded material.

Atterberg Limits

Water is one of the most important components influencing the engineering behavior of a soil. It is therefore important to know at what specific water contents the behavior will change. Atterberg Limits are essentially the specific water contents of these critical stages in a soil's mode of behavior (Holtz and Kovacs, 1981).

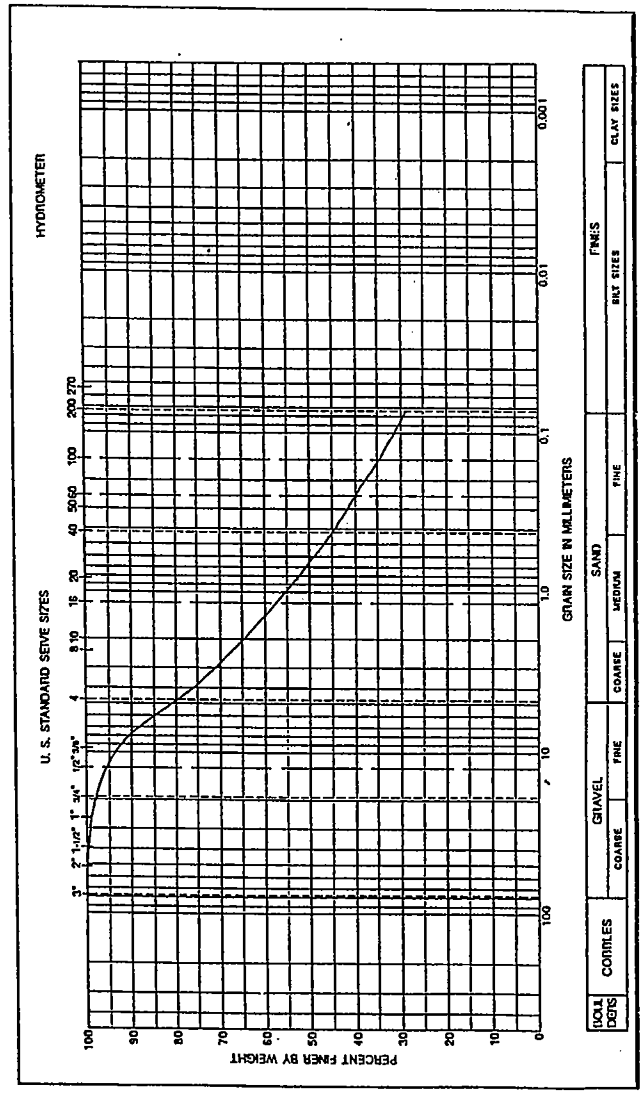


Figure 1. Average Grain Size Distribution, Type I Coal Refuse.

The determination of liquid and plastic limits is performed using material passing a No. 40 sieve. ASTM Method D4318 is the applicable procedure that was followed for tests on the samples of coal refuse.

The liquid and plastic limit tests are performed manually by laboratory personnel. This introduces a greater likelihood of unwanted variation in sample preparation and test performance. Because of these factors, several trials are typically performed and averaged to determine these indices.

Using the liquid and plastic limits values for a particular soil, two additional parameters can be determined. The plasticity index, I_p, is the difference between the liquid limit and plastic limit. This value indicates the range of moisture content over which the material will behave in a plastic manner. The liquidity index, LI, indicates the nature of the soil in its natural state relative to both the liquid and plastic limits. This value is calculated using the following equation:

$$LI = \frac{(W_n - W_p)}{I_p}$$

where LI = Liquidity Index

W_n = Natural moisture content

W_p = Plastic Limit

I_p = Plasticity Index

If the value of the liquidity index is negative then the soil is drier than the plastic limit. If the value lies between 0 and 1 the soil will exhibit plastic behavior. A value greater than 1 indicates that the soil will exhibit liquid behavior. Both the plasticity index and the liquidity index are important as they indicate how the fines in a soil will behave at various moisture contents. These values influence how a soil should be prepared and used in construction operations. They can also determine the suitability of a particular soil relative to the anticipated service conditions.

Because shale is present in both types of coal refuse, it was anticipated that the fine portions of each would exhibit a significant percentage of clay. It was surprising to learn from the Atterberg tests that the fines for both materials show nonplastic behavior. During preparation and

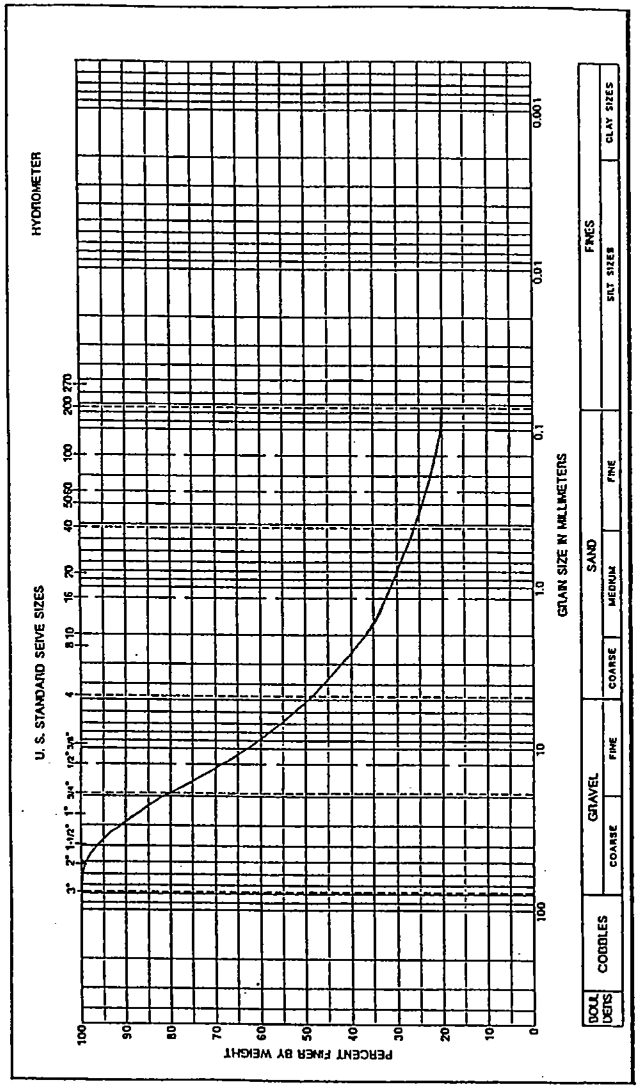


Figure 2. Average Grain Size Distribution, Type II Coal Refuse.

performance of the Atterberg tests, the samples exhibited high degrees of dilatancy at higher moisture contents; at lower moisture contents the samples were nonpliable to the extent that they could not be prepared and grooved correctly in the liquid limit device. Based on these results, it probably would have been valuable to perform hydrometer tests to determine the distribution of the fine portion. However, a lack of time at that stage in the testing program prevented the inclusion of these tests. Results most likely would have indicated that much of the fine fraction was silt-sized material, rather than the more cohesive clay, which is typically .002 mm or less in size. This would account for the lack of plasticity shown by the material.

Soil Classification Systems

The two primary classification systems in use today, each with a different origin and purpose, are the Unified Soil Classification System and the AASHTO Classification. The Unified Soil Classification System, or USCS, was originally developed for use in airfield construction during World War II. It was modified in 1952 by the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers to make it applicable to dams, foundations, and other types of construction. The basic principle of the USCS is that coarse grained soils can be characterized by the size distribution of their constituent particles, whereas fine-grained soils can be classified according to their plasticity, as measured using Atterberg limits. The USCS is commonly used by U.S. Government agencies, geotechnical consulting firms and soil testing laboratories (Holtz and Kovacs, 1981). It was also adopted by the American Society for Testing and Materials as the basis for soil classification.

The other prominent classification system is the American Association of State Highway and Transportation Officials, or AASHTO. It originated from an extensive research program of the U.S. Bureau of Public Roads in the 1920s. That agency was concerned with the characteristics of soils used in secondary road construction and thus developed a system for their classification. The original classification system was based on the performance of soils used as surfacing material or with a thin asphalt wearing layer. Since that time the system has been modified to allow determination of the suitability of soils for embankments, subgrades, subbases, and bases.

In the AASHTO system, soils are classified into eight different groups, designated A-1 through A-8. Each group has various subgroups that are specified with an additional number or lower case letter, such as A-2-4 or A-1-b. In addition to the group and subgroup classifications, there is another parameter, the group index, that is calculated using a formula relating grain size percentages and Atterberg limit values. The index value can also be determined using a nomograph. The purpose of the group index is to estimate the service performance of soils, most typically when used as pavement subgrade material. Many pavement design methods make use of the group index as an indicator of soil behavior, especially when extensive laboratory testing is either not required or too costly. Such might be the case for many secondary road projects at the county or township level.

Classification of Friar Tuck Coal Refuse

Because both the ASTM and AASHTO designations are useful in characterizing a soil material, each method was used to classify Type I and Type II coal refuse. The classification process was based on the average grain size distribution curves of Figures 1 and 2, and on the results of the Atterberg limits tests. Table 2 shows the information used to classify each of the two types of coal refuse, along with the resultant ASTM and AASHTO designations.

Table 2. Classification Data and Designations, Type I and Type II Coal Refuse.

	Type I Coal Refuse	Type II Coal Refuse
% Gravel (4.75 mm-75 mm)	21	52
% Sand (.075 mm-4.75 mm)	51	28
% Fines (<.075 mm)	29	20
Plasticity	Non-plastic	Non-plastic
ASTM Classification	SM Silty sand with gravel	GM Silty gravel with sand
AASHTO Classification	A-2-4	A-1-b
AASHTO Group Index	0	0

Compaction Characteristics

Soil materials in place typically do not have sufficient strength and other desired qualities required for road construction. Through mechanical compaction, however, many of these properties may be improved. By increasing the density using compaction, the following changes are commonly achieved:

1) increase in shear strength.

2) decrease in shrinkage potential.

3) decrease in compressibility.

4) decrease in permeability.

Typically, all of these factors are important when placing road construction materials.

Compaction Tests

The standard laboratory compaction test, the Proctor test, was developed by R.R. Proctor in 1933. Proctor proposed that compaction is a function of four variables: 1) dry density, 2) water content, 3) compactive effort, and 4) soil characteristics (Holtz and Kovacs, 1981). The test procedure developed by Proctor is an impact type, in which a special hammer is dropped a specified number of times onto a soil sample contained within a steel mold. In the Standard Proctor Test, ASTM D698, the hammer weighs 5 pounds and is dropped from a height of 1 foot; the soil is compacted in 3 layers with 25 blows of the hammer for each layer. In the Modified Proctor Test, ASTM D1557, the hammer weighs 10 pounds and is dropped 18 inches; the soil is compacted in 5 layers with 25 blows per layer. The procedure followed for a compaction test is relatively simple. Several samples of soil are prepared at different moisture contents, usually varying by 2-4 percent each. Each sample is compacted as described above, depending upon which test method is chosen. The dry densities achieved by compaction are plotted as a function of water content on a typical graph. A curve is then drawn through the data points. The peak of the curve indicates both the maximum dry density and the optimum water content of compaction for the given soil at the particular compactive effort.

One factor that greatly influences the compacted density of a soil is the grain size distribution. If the soil is well graded, having a fairly even distribution of all grain sizes, then a high compacted density can be achieved. This occurs because smaller particles serve to fill in the pore spaces of successively larger particles, resulting in a lower percentage of air remaining in the voids. For poorly graded soils with grain sizes within a very narrow range, a very open structure is formed. With a large amount of pore space that can only be occupied by air, lower compacted densities are the result.

Compaction tests were performed on both Type I and Type II coal refuse for two reasons. First, it was important to know what densities could be expected for the materials if utilized for road construction purposes. By comparing the compacted densities to the in situ density values, a conversion could be made relating the required borrow volume to the final compacted volume of material. The second reason was related to the scheduled performance of California Bearing Ratio tests. Because the strength characteristics of a material vary greatly with density, CBR tests should be performed at densities to be expected in service. The most likely use of coal refuse for the Friar Tuck access road would be as borrow material. In this utilization the material would be excavated, transported and compacted to form a particular component of the roadway structure. Any CBR tests should thus be performed under conditions approaching maximum density and optimum moisture content.

Compaction tests for Type I and Type II coal refuse were performed according to ASTM D698, which is the standard Proctor Test. There was, however, a slight difference in preparation methods for the samples of each type. D698 states that if more than 10 percent of the material is retained on the 3/4 inch sieve, then Method D of the procedure shall be followed. This method requires the replacement of material retained on the 3/4 inch sieve with an equal amount of material that passes the 3/4 inch sieve and is retained on the Number 4 sieve. This procedure also calls for the use of a 6 inch diameter mold rather than the typical 4 inch diameter size, and for each of the three compacted layers to receive 56 blows with the hammer instead of the typical 25 blows. Type II coal refuse, because of its coarser gradation, qualified for the use of Method D.

Type I refuse, on the other hand, had less than 10 percent but more than 7 percent retained on the 3/4 inch sieve. This fell under the jurisdiction of Method C of D698, wherein the material retained on the 3/4 inch sieve is discarded, without replacement. Like Method D, Method C requires the use of the 6 inch mold, with 56 blows per each of the 3 layers.

The compaction curves for Type I and Type II coal refuse are shown in Figures 3 and 4. The maximum densities for the two materials are significantly different. This may be the result of two factors: 1) differences in specific gravity of the constituent particles, and 2) differences in compaction efficiencies resulting from grain size distribution. In order to determine which material is actually in a better state of compaction, the void ratios of each can be compared. A lower void ratio signifies that a higher percentage of the soil mass is composed of solids rather than air. The void ratios of each material at the maximum compacted densities are shown in Table 3. It is apparent that Type II coal refuse, with a void ratio of 0.16, can be compacted to a much denser configuration than Type I refuse with a void ratio of 0.44. Although permeability tests were not performed, the lower void ratio of Type II coal refuse suggests that the material would also have a lower permeability.

Table 3. Compaction Characteristics of Coal Refuse

Property	Type I Coal Refuse	Type II Coal Refuse
Maximum Dry Density	71 lb/ft^3	102 lb/ft^3
Optimum Water Content	31%	14%
Void Ratio at Maximum Density	0.44	0.16

Another significant difference in the compaction test results are the optimum water contents for each material. Type I refuse has an optimum of 31 percent, whereas Type II has an optimum of 14 percent. This difference is very noteworthy, considering that the materials are similar in nature and in origin. The value of 31 percent for Type I refuse is also very high in terms of typical values of optimum water content. As with the natural moisture content values, it is

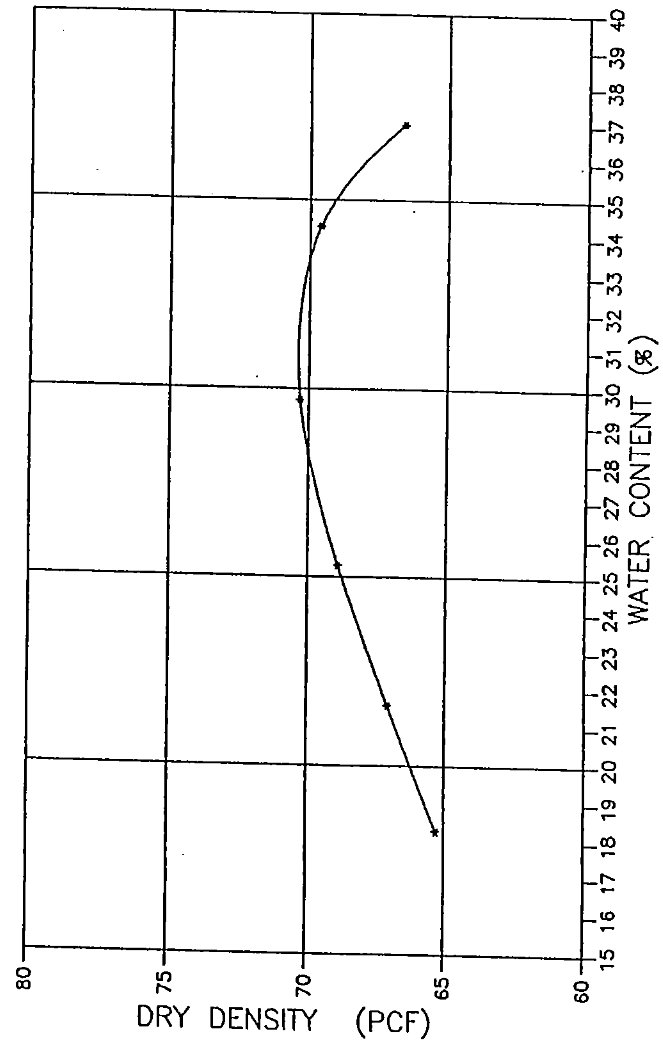


Figure 3. Compaction Curve for Type I Coal Refuse.

concluded that the differences in the optimum water contents are due to the high percentage of weathered shale in Type I coal refuse, and the higher value of absorption for this material. Such a large degree of absorption requires a higher overall percentage of water to develop adequate films on the surfaces of the particles to provide the needed lubrication for compaction. Because the absorption of water by Type II refuse is much lower, the films can begin to form at much lower values of water content.

California Bearing Ratio Tests

Many of the index properties and classification data for a given soil are important in determining suitability as a road construction material. This information provides information on the following characteristics: behavior under certain climatic conditions; optimum handling and placement procedures, including most efficient methods of compaction; magnitude of degradation expected during service; and the types of stabilization methods best suited for a particular soil material. The design of a roadway structure, however, requires a quantitative determination of the load-bearing capacities of the materials being used. The California Bearing Ratio test provides this information. The results of the CBR test are used in conjunction with data on the expected traffic volume and axle loads for the roadway; respective thicknesses for the various components of the pavement structure are then determined.

CBR Testing Procedures

The applicable method for CBR testing of soils is ASTM D1883. A variety of service conditions can be simulated for the CBR test. One of the most common is the "soaked" test, in which the compacted sample is soaked in water for a period of four days before performing the penetration part of the test. The total amount of swell during soaking is also measured. The soaking process helps to simulate a worst-case condition of total saturation. Although the use of proper drainage for road construction projects should prevent full saturation, the results of the soaked test provide an added factor of safety in the design process. In the case of the Friar Tuck access road, however, the use of soaked test data may not be overly conservative. The lack of a

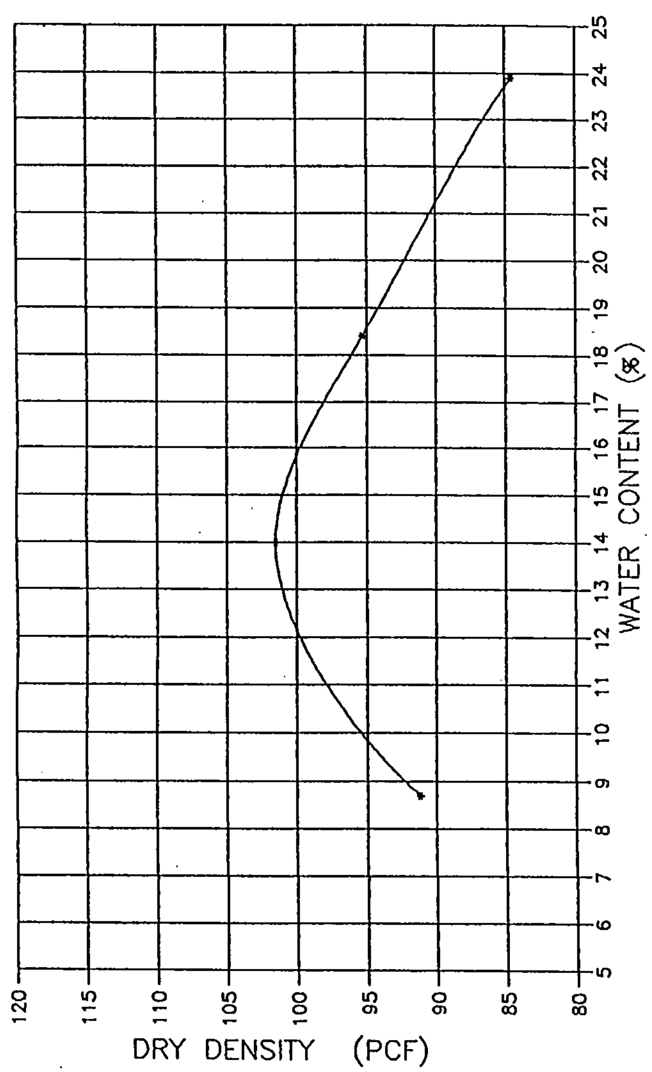


Figure 4. Compaction Curve for Type II Coal Refuse.

water resistant surface course (such as an asphalt seal) and the difficulties in providing good drainage to all segments of the roadway alignment, make saturation a real possibility at the Friar Tuck site.

Although the CBR values determined from the soaked test would be used in the actual design, it was decided to perform nonsoaked tests on both Type I and Type II coal refuse. Based on the extreme differences in weathering history of the two types of refuse, one would expect a difference in the reaction to, and performance following, soaking. The results of soaked and non-soaked tests could be compared to determine if soaking does, in fact, affect one type significantly more than the other.

An additional concern was the reaction of the coal refuse materials to freezing conditions, especially repeated cycles of freezing and thawing. It had been observed that the existing roadway at the site had experienced a rather extreme period of deterioration in the spring of 1990. Because many portions of the roadway were originally constructed of coal refuse, it seemed possible that this material had been directly affected by the climatic conditions of the preceding winter. In fact, the month of December, 1989, was one of the coldest in recent years.

In order to determine the possible susceptibilities to freeze/thaw action and any resultant strength losses, an original test was devised for this purpose. The main steps of the procedure are as follows:

1) Compact material in CBR mold.

2) Soak for 24 hours.

3) Subject specimen to 9 freeze/thaw cycles, 1 per day;

1 cycle = air temperature reduced from +5.0 C to -15.0 C at constant rate over 4-hour period; temperature remains at -15.0 C for 8 hours; air temperature increased from -15.0 C to +15.0 C over 4-hour period; temperature remains at + 15.0 C for 8 hours.

4) 300 ml water in the form of a 1" thick ice cylinder placed on top of sample, allowed to melt and infiltrate during thawing portion of 10th cycle. Addition of water is intended to simulate possible infiltration during mid-winter thaw. 1" of water (ice) is equivalent to approximately 6" of average snow.

5) Subject specimen to additional 9 freeze/thaw cycles.

6) Soak for 24 hours.

7) Remove from water, drain 15 minutes, perform CBR penetration test.

Temperature control was accomplished using a walk-in size environmental chamber housed in the Civil Engineering Department, Purdue University.

Freeze/thaw testing was performed on both Type I and Type II coal refuse. Four samples of each were tested in this manner. Half of the samples were used as controls, for which all steps of the process except freeze/thaw cycling were performed. As in the standard soaked and non-soaked CBR tests, it was expected that differences would occur in response to freeze/thaw cycles for both types of coal refuse.

CBR Results, Soaked and Non-Soaked Tests

The load/penetration data for the soaked and non-soaked CBR tests were plotted for comparison. The actual CBR values from these tests, along with compacted densities, water contents, and amounts of swell (where applicable) are shown in Table 4. Individual samples are identified using the following format: "CBR", (I) or (II) - for type of coal refuse, trial number, S for soaked, and N for Non-soaked. For example "CBRII2-S" indicates trial number 2, under soaked conditions, for Type II coal refuse.

By examining the test results shown in Table 4, one feature becomes clear, the CBR values for non-soaked samples of Type II refuse are very high, averaging, 79.2 percent. The values for Type I non-soaked samples are lower, averaging 42.7 percent. This large difference was anticipated due to the weathering processes that Type I refuse has undergone. Weathering has resulted in the significant weakening of individual particles and, in general, created a lightweight friable material. Type II coal refuse, by contrast, has been protected from surficial weathering and remained in a much more massive, and stronger, condition.

Table 4. CBR Data, Soaked and Non-Soaked Tests.

Type I Sample	Water Content (%)	Dry Density (pcf)	Swell (in)	CBR (%)
CBRI1-N	30.5	70.4	NA	45.0
CBRI2-N	33.1	72.6	NA	40.0
CBRI3-N	29.6	70.6	NA	38.5
CBRI4-N	30.7	70.8	NA	47.1
Average	31.0	71.1	NA	42.7
CBRI1-S	32.3	69.9	.016	34.7
CBRI2-S	29.0	70.0	.019	37.3
CBRI3-S	30.1	70.9	.023	29.3
CBRI4-S	30.7	69.5	.020	29.3
Average	30.5	70.1	.020	32.7
Type II Sample				
CBRII1-N	20.8	92.6	NA	78.0
CBRII2-N	16.9	98.6	NA	75.3
CBRII3-N	14.9	102.9	NA	82.3
CBRII4-N	17.4	98.0	NA	81.0
Average	17.5	98.0	NA	79.2
CBRII1-S	20.6	91.3	.033	19.7
CBRII2-S	17.2	95.9	.037	37.7
CBRII3-S	15.4	97.8	.015	43.8
CBRII4-S	17.0	96.6	.036	33.7
Average	17.6	95.4	.030	33.7

More interesting is the comparison of the respective losses in strength caused by soaking. Type I coal refuse falls from a CBR value of 42.7% to 32.7%, a decrease in strength of 23.4 percent. Type II coal refuse drops from a CBR value of 79.2% to 33.7%, a decrease of 57.4 percent. Obviously the effects of soaking are much more damaging to Type II refuse than to Type I refuse. Again, this is be explained by the differences in weathering history. Type I refuse was exposed at the surface for over 40 years and has undergone many cycles of wetting, perhaps even saturation. Alternatively, Type II refuse has remained four feet below the surface in a protected state, and probably has not been exposed to water to the same extent. Also, being confined at four feet below the surface, the amount of volume change and degradation following wetting was likely much less. Thus the effect of the 4-day soaking period is more damaging, probably due to degradation or slaking of the shale that comprises a large portion of Type II coal refuse. The intensified effect of soaking is also indicated by values of swell that are approximately 50% greater for Type II material.

CBR Results, Freeze/Thaw Cycle Tests

Table 5 summarizes the results of the freeze/thaw CBR tests, as well as compacted densities, water contents, and amount of swell during freeze/thaw cycling. Samples are identified in the following manner: "FT" for Freeze/Thaw, followed by (I) or (II) to indicate the refuse type, (C) or (V) to indicate a control sample or variable sample, and finally the sample number. For instance, FTI-V3 represents sample number three of Type I refuse, for the freeze/thaw cycling case.

It is apparent from Table 5 that both types of coal refuse were significantly affected by the cycles of freezing and thawing. The control samples for Type I refuse averaged 39.1% CBR, whereas the variable samples averaged 12.6% CBR. This represents a 68 percent loss in strength due to the effects of freezing and thawing. The control samples for Type II refuse had a much lower initial strength, with a CBR value of 10%. However, the Type II variable samples had a CBR value of 7.4%, which is only a 26 percent loss in strength.

Table 5. CBR Data, Freeze/Thaw Tests.

Type I Sample	Water Content (%)	Dry Density (pcf)	Swell (in)	CBR (%)
FTI-C1	30.6	72.6	.013	40.0
FTI-C2	30.6	72.4	.007	40.7
FTI-C3	29.7	72.2	-.003	36.5
FTI-C4	30.8	72.2	-.001	39.3
Average	30.4	72.4	.004	39.1
FTI-V1	31.5	71.7	.214	10.7
FTI-V2	31.1	72.0	.216	12.0
FTI-V3	30.5	72.0	.257	15.0
FTI-V4	29.3	72.4	.264	ND
Average	30.6	72.0	.238	12.6
Type II Sample				
FTII-C1	17.3	94.4	.023	11.7
FTII-C2	17.0	95.5	.038	16.7
FTII-C3	16.7	90.8	.025	7.1
FTII-C4	16.1	88.9	.004	4.3
Average	16.8	92.4	.023	10.0
FTII-V1	17.3	95.8	.145	6.7
FTII-V2	16.5	97.3	.175	7.1
FTII-V3	17.7	95.4	.112	5.6
FTII-V4	16.8	93.7	.117	10.2
Average	17.1	95.6	.137	7.4

Type I refuse also averaged larger amounts of swell than did Type II refuse. Originally it was anticipated that the unweathered state of Type II refuse would contribute to greater amounts of expansion upon freezing. What may have played a larger role were the grain size distributions of the two types of coal refuse. Recalling that Type I refuse was classified as a silty sand and Type II refuse as a silty gravel, one realizes that the silty sand is more susceptible.

The Army Corps of Engineers uses a system which classifies soils into four groups of frost heave susceptibility based on grain size distribution (Yoder, 1959). The groups are designated as F1 through F4, ranging from least to most susceptible, respectively. Type II coal refuse is classified as F1, whereas Type I refuse is classified as F4. This may explain the larger degrees of swelling experienced by the samples of Type I refuse. The specific amounts of frost heave for subgrades, subbases, and bases are apparently important when dealing with asphalt or rigid pavements, where expansion can lead to buckling or cracking of the wearing surface. This parameter is not nearly as important for unsurfaced roads. What is important, however, is the manner in which the bearing capacity is affected by the expansion. The test results may help to provide an indication of the degree to which the bearing capacities are affected by the destructive action of freeze/thaw cycles.

Summary

The various types of testing performed on Type I and Type II coal refuse achieved several goals. The first was to determine values for physical properties, the specific composition, and the classification of the materials. This information is useful not only in terms of engineering applications but also for other scientific considerations.

The second goal was to demonstrate the degree to which differences in weathering history have affected these physical properties. The characteristics of coal refuse are highly variable and depend on the properties of the coal seam and the mining disposal methods. However, several generalizations can be made regarding the effects of weathering on the behavior of Friar Tuck coal refuse.

The primary goal, and the focus of the testing program, was to determine the compaction characteristics and bearing capacities (as measured by CBR) of the two types of coal refuse. These parameters provide the basis for design and construction of an access road at the Friar Tuck site. By performing the CBR tests under three different conditions, the effects of various climatic and service conditions were evaluated. Although the non-soaked CBR values do not have a major practical significance, they do demonstrate the relative amounts of strength during soaking and in freezing and thawing. Data shown in Table 6 below summarize the most significant findings of the CBR testing program, and provide the basis for the design considerations presented next.

Table 6. Overall Results, CBR Testing.

Condition	Type I Coal Refuse	Type II Coal Refuse
Non-Soaked	42.7% CBR	79.2% CBR
Soaked	32.3% CBR	35.3% CBR
Freeze/Thaw	12.6% CBR	7.4% CBR

DESIGN RECOMMENDATIONS AND CONCLUSIONS

Introduction

Many of the factors affecting roadway design have been discussed in the preceding text. These include selection of materials for various components of the roadway structure, testing of these materials to determine bearing capacities and other parameters, and various design methods that apply. Two issues are addressed below. First is the determination of the components of a roadway structure for which coal refuse might be utilized. Second is the determination of which of the two types of refuse is best suited for each use. Associated with this issue is the determination of the specific use which has the greatest potential to provide a significant cost reduction.

Choice of Material

Two types of coal refuse were examined with regard to potential use in the Friar Tuck access road. Each has advantages and disadvantages which must be considered in the decision process. Environmental impacts as well as effects on the overall reclamation plans for the site are involved. To select the most suitable type of coal refuse, one first must narrow the range of possible uses for the material. Each use is examined and conclusions made regarding the two types of coal refuse.

Wearing Surface

The wearing surface should be hard enough to withstand the abrasive action of traffic. It should also possess a particle gradation which provides a proper seal against excessive infiltration of water. One characteristic which contributes to such a seal is the presence of a clay which serves to bind the particles together at the surface.

Analyses of Type I and Type II refuse indicated that each contain a large percentage of shale. Although Los Angeles abrasion tests were not performed on these materials, shale in general does not possess a high abrasion resistance. It is anticipated that much of the traffic on the access road would be tractor-trailer vehicles and heavy construction equipment. Therefore, it is likely that both types of coal refuse would undergo considerable degradation. It was also determined from Atterberg tests that both materials are non-plastic, indicating either a very small amount or no clay fraction. Without clay to provide a seal against water infiltration, the bearing capacity of the underlying materials could be severely affected. From this, it is concluded that coal refuse would not perform well as a wearing surface.

Subgrade

Typically, the subgrade consists of the original, in-place soil material. Because it is located at the bottom of the layered roadway system, the subgrade does not require particularly high values of bearing capacity. Some of the required features are good drainage, ease of compaction and resistance to swell during saturation. Both Type I and Type II coal refuse rate well with regard to these characteristics, with the possible exception of drainage. Based on the well-graded nature of coal refuse and the percentage of fines, a low permeability is likely, perhaps in the range of 10^{-5} to 10^{-6} cm/sec.

The proposed alignment of the access road will not have a significant distance across in situ coal refuse. Thus its contribution as natural subgrade is minimal. There are some areas, however, where low lying segments will require extra fill to bring the road up to grade. Based on the observed and measured characteristics of coal refuse, the material should be well-suited for such fills. To determine which of the two types would provide better overall performance, the conditions most likely to occur within the subgrade must be considered. As the subgrade occupies the lowest position in the pavement structure, and because these locations would be topographically in low-lying areas, the likelihood of complete saturation is great. Although both Type I and Type II coal refuse have nearly identical values of bearing capacity following soaking, it is concluded that Type I would be the better choice for subgrade fill. The fact that Type II refuse loses more than 50 percent of its strength during soaking suggests that other physical changes such as particle breakdown might be occurring. For areas where the thickness of fill approaches 5 feet, such breakdown could increase the likelihood for excessive settlement. In addition to the negative mechanical aspects, the unweathered nature of Type II refuse might also present some problems in terms of environmental effects. The likelihood of frequent saturation would contribute to the production of excessive amounts of acidic leachate.

The utilization of Type I coal refuse as fill for low-lying areas seems feasible. There would be some qualifications and restrictions controlling its placement. Details of this use are discussed in a subsequent section on design and construction specifications.

Subbases and Bases

The primary difference between a base and subbase is that bases are typically materials of higher quality and strength. There are some situations where both are needed in a road cross section. When the subgrade is either highly frost susceptible or simply has poor bearing capacity, a large thickness of overlying layers is required to protect it from frost penetration or excessive stress. The use of a subbase helps reduce the cost of using the more expensive, high-quality base course material throughout. In the case of the Friar Tuck access road project, limited funding prevents the placement of two separate components below the surface course, which most likely would be crushed stone. Thus, the evaluation of coal refuse as a base course is the primary consideration.

Two different types of material can function effectively as base courses. The first is an open graded material such as crushed quarry stone which contains a small amount of fines. Because of the reliance on grain-to-grain contact, however, this type of material must be resistant to abrasion. The second type of material is dense-graded, where the interstices are filled with sufficient amounts of fines to provide stability through the interlocking structure of the material. This gradation typically possesses high density and low permeability, while being susceptible to frost action. Based on the grain size analyses, both Type I and Type II refuse are of the dense-graded variety.

Unlike a subgrade material, the utilization of a base course nearer the surface presents the possibility of freeze/thaw action. In terms of the respective responses of Type I and Type II coal refuse to freeze thaw cycles, Type II experienced a smaller percentage loss of strength compared to the control samples. However, the actual value of residual bearing capacity is still higher for Type I refuse. Because bearing capacity is of prime importance for a base course, Type I would be the best suited in this respect.

Based upon the preceding discussion, there are two basic uses for which coal refuse could be employed: as a subgrade soil for filling low-lying areas, and as a base course material. In both cases, Type I refuse appears to be better suited for these applications than does Type II refuse. Because Type I refuse has already experienced a significant weathering process, the likelihood of major physical changes during service is reduced. Also reduced is the chance for production of acidic leachate which could further contribute to the existing water quality problems at the Friar Tuck site. Having considered these and other factors it is concluded that Type I coal refuse is the preferred material for use in road construction.

Utilization of Material

The two possible uses for Type I coal refuse discussed above are not mutually exclusive. In other words, the material could be used both for subgrade fill and for base course road construction. It is apparent that base course use would require a greater amount of material, as this application would be required for the entire length of the proposed roadway, some 1.1 miles. Because of the greater amount required for the base course, only this application will be discussed in terms of design and construction. This approach also seems most appropriate for discussion as requirements are more stringent for base course use.

The Design Process

Some of the most widely used methods for design of roadways are those based on the CBR test. Because this study focused on the respective CBR values of Type I and Type II coal refuse, this method will be used to formulate the design.

Choice of CBR Design Method

There are several design charts and methods which use the CBR value as the basis for roadway design. Among these are the Kentucky Method and Wyoming Method but others exist. One method, devised by the Asphalt Institute, takes several important factors into account and is relatively simple to use (Foster, 1965). The Asphalt Institute chart considers traffic volume, axle loads, and bearing capacity in its determination of the overlying pavement required to prevent bearing capacity failure of the material below. In this discussion pavement does not refer to only the asphalt surface, but to all the layered components of the roadway.

Traffic Evaluation

One of the components of the Asphalt Institute chart is the traffic factor. Evaluation of this factor begins with a determination of the magnitude of expected axle loads. Although there are likely to be many different types of vehicles using the Friar Tuck access road, the most common will be either dump trucks or tractor-trailer vehicles. This type of traffic is expected for delivery of reclamation-related materials such as riprap, crushed stone, fertilizer, and mulch. Possible shipments of treated sewage sludge would also be delivered in this manner. As this type of traffic would account for the highest axle loads, this load condition will be used in the roadway design.

In Indiana, the legal single axle load limit is 18,000 lb. For tandem axle arrangements, such as those used by tractor-trailers, the allowable combined load is 32,000 lb. It is not known what percentage of the maximum load is most generally carried; for the sake of this design, it is assumed that 75 percent of the maximum, or 24,000 lbs, is a typical axle load. It also seems likely that the trucks in question would not be loaded to the limit, given the "off-road" nature of the Friar Tuck site.

The Asphalt Institute chart considers not only the magnitude of axle loads, but also the volume of traffic. The traffic classifications used for the chart include light, medium, heavy, and very heavy. The ranges for each of these categories are as follows:

Light - maximum of five trucks per lane per day.

Medium - maximum of 25 trucks per lane per day.

Heavy - maximum of 250 trucks per lane per day.

Very Heavy - Unlimited trucks per lane per day.

It is unknown what volume of traffic will be using the Friar Tuck access road on a regular basis. Although the majority of traffic will probably qualify as "light", there may be days when a specific activity in the reclamation process will require a large number of shipments within a short period of time. It is for these situations that the roadway must be designed. A "medium" designation will most certainly account for the expected traffic, and will probably introduce a slightly conservative factor in the design process.

Bearing Capacity

When determining the thicknesses of roadway components, the soaked CBR values are typically used for design purposes. Using these values ensures that a worst case situation with a high degree of saturation is considered. Thus, the appropriate CBR value for Type I coal refuse is 32.3 percent. It should be noted that although CBR values resulting from freeze/thaw action were measured, these values are not normally used as a basis for design. Rather, the damaging effects of climate are controlled through proper design and construction specifications.

Determination of Design Thickness

The purpose of all design methods is to determine the thickness of overlying material required to prevent bearing capacity failure of a particular underlying material. An added requirement is that the overlying material must be of higher quality than the one below. In the proposed design for the access road, coal refuse is to be used as a base course beneath a crushed stone wearing surface. Below the base course will be the existing soil which will function as the subgrade.

In this study no attempt was made to accurately determine the types of soil materials which will be traversed by the proposed access road. Most materials found at the site, whether original or a product of mining, have estimated CBR values of at least 3 percent. These estimates are based on the correlation of CBR values to group index values, which can be determined from grain size distribution and Atterberg indices. The only material that is likely to perform very poorly is the natural loess that is present in many parts of the site. A fine silt material such as loess is subject to excessive expansion when present within the frost zone. If this material occurs as the subgrade within the proposed alignment of the access road, the construction specifications require its removal to at least 2 feet below present grade. Excavation of the soil to this depth ensures removal of the portion which has been most affected by frost penetration. The loess would then be replaced with a higher quality material, perhaps coal refuse.

Subgrade Protection

For the design problem under consideration, it is assumed that the subgrade will have a CBR value of no less than 3 percent. The roadway will be designed for an axle load of 24,000 lb. with a "medium" traffic volume designation. Using these values, the Asphalt Institute chart can be applied. The resultant thickness will be that of the combined coal refuse base course and crushed stone wearing surface required to prevent subgrade failure. Figure 5 shows the design

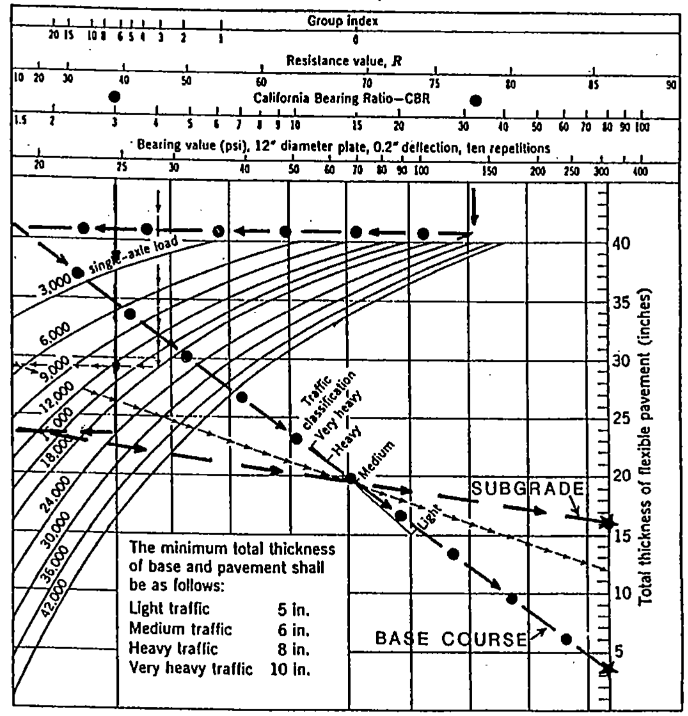


Figure 5. Thickness Determination Using the Asphalt Institute Design Chart.

chart, along with the specific points and lines constructed to determine the appropriate thickness. As indicated on the figure, the total thickness of coal refuse and crushed stone must be at least 16 inches.

Base Course Protection

Having established the criteria for the protection of the subgrade, the coal refuse base course must now be considered. Again using Figure 5, and given a CBR value of 32.3 percent for Type I coal refuse, the required thickness of crushed stone surface can be determined. The thickness is determined to be a minimum of 3 1/2 inches. For simplicity, the roadway could consist of a 12-inch thick coal refuse base course and a 4-inch thick crushed stone wearing surface.

The Construction Process

The preceding sections have provided details on the proper design thicknesses for the coal refuse base course and crushed stone surface course. Equally important is the manner in which construction of the roadway proceeds. The following sections provide a sequential process which should result in the effective construction of a competent roadway.

Selection and Acquisition of Borrow Material

Assuming that the proposed roadway alignment prevails, the first step is to determine approximately how much coal refuse borrow material will be needed. For utilization as a base course, the amount will obviously depend upon the length and width of the access road. The amount needed as artificial subgrade fill may be slightly more difficult to calculate. However, a comparison of the proposed final grade with existing topographic contours should readily lead to volume calculations for this amount. It is important that the required compacted volume be related to an equivalent uncompacted volume. There is a significant difference in these two volumes for Type I coal refuse, which has an in-place density of 41 pounds per cubic foot and a standard Proctor density of 70 pounds per cubic foot. As a result, 1.7 cubic yards of natural material are required for every cubic yard of compacted material.

The utilization of coal refuse should also conform to the overall reclamation plans for the site. Because Type I material normally exists only in the top 2 to 3 feet of a coal refuse deposit, the selection of a borrow area must be carefully considered. In some areas of the Friar Tuck site, coal refuse was spread thinly over large areas. In most cases the thickness is such that all of the material would be classified as Type I. By removing this material and exposing the underlying natural soil, many of these areas could be made more suitable for subsequent vegetation. The unsightly appearance of these areas could also be eliminated. A disadvantage of this method is that many areas of thin gob deposits contain extennsive gullies, creating problems for the removal of the coal refuse.

The other possible source of Type I coal refuse would be the weathered surface zones of larger piles or deposits of the material. Unfortunately, the removal of the surficial layer would expose unweathered material to water and oxygen, thus restarting the cycle of acidic leachate production. This method would be feasible if several smaller piles were removed and consolidated into larger ones, with the exposed unweathered refuse capped with suitable soil material. A similar process was scheduled for the reclamation of the Southwest Gob Pile in late 1991. In this area, the steep slopes of the refuse pile were graded back yielding a much gentler gradient. The excess material was placed in a large fill area and covered with soil to prevent high rates of infiltration and acid production.

Regardless of the specific borrow areas that are selected, the coal refuse found in those areas should be evaluated to verify that the properties are similar to those used in the design. It is possible that differences between various coal seams could affect the properties of the refuse produced during extraction. Even differences in mining methods or equipment could conceivably change the nature of the material.

Preparation of the Subgrade

Before the coal refuse base course can be placed, the subgrade must be adequately compacted to reduce the possibility for subsequent settlement or bearing capacity failure. Most of the materials over which the access road is to be constructed are cohesive in nature, and are typically clayey silts or silty clays. These materials would be compacted most efficiently with a sheepsfoot roller.

It is likely that many different types of subgrade materials will be encountered. Because of this, requirement of a specific percentage of Proctor density would be difficult to establish as the value would depend on the type of material at a given location. A more appropriate alternative to a density requirement would be to specify a minimum number of passes of the compaction equipment.

After thorough compaction of the subgrade, it is recommended that the compacted surface not be rolled or smoothed before placement of the base course. By placing the coal refuse over the existing "tracks" of the sheepsfoot roller, a better interlock can be achieved between the base course and the subgrade. Smoothing the subgrade to a planar surface could increase the likelihood of a failure along that surface. Such a zone would also be more susceptible to the formation of ice lenses and to subsequent frost heave.

Base Course Construction

After the proper preparation of the subgrade, the coal refuse base course can be constructed. It was determined previously that 12 inches of base course would be required. The construction specifications would most likely require the placement and compaction of two 6-inch lifts. Because Type I coal refuse is a low plasticity material and is classified as a silty sand with gravel, a sheepsfoot roller would not be specified. Instead, a vibrating smooth drum roller would be more appropriate. The well graded nature of the coal refuse, along with the vibratory action of the roller, would ensure a high degree of compaction.

The CBR results presented in this research were performed at or near maximum standard Proctor density, and this degree of compaction should also be achieved during construction. Because no modified Proctor tests were performed, it is unknown what percentage of modified density would achieve the same degree of compaction. It is possible that a significantly higher compacted density could be achieved in a modified Proctor test. If CBR values at this density were found to be higher, then a percentage of the modified density could be specified. A reduction of design thickness would not be recommended, however, even with higher CBR values. This would serve to provide an additional safety factor in the design.

It is concluded that the best performance would be attained by leaving the uppermost lift of coal refuse in the smooth condition formed by the vibratory roller. It is anticipated that the angular nature of the stone surface course would create sufficient penetration to provide an interlocking zone between the two materials.

Surface Course Construction

Placement of the 4-inch layer of crushed stone as a wearing surface would also be accomplished best through the use of vibratory compaction. The most likely method of ensuring optimum compaction would be to specify a minimum number of passes by the compaction equipment. Construction of an extremely smooth surface is not necessary for the crushed stone, as it will likely experience an initial period of shifting and minor rutting in response to traffic. A short-term maintenance program might be required for grading and repair of any excessive degradation.

An essential step in the completion of the roadway surface is the construction of a crowned center to encourage runoff. It is also recommended that a thin layer of fine stone chips and dust be applied to the crushed stone surface and incorporated thoroughly in the upper portion. This type material, on wetting, forms an effective seal against water infiltration. Because the base course is dense graded and relatively impermeable, it is essential to prevent infiltration and accumulation of water at the surface course/base course transition.

Cost-Effectivenes of Coal Refuse Utilization

One of the primary goals of this work was to determine the cost-effectiveness of using coal refuse in place of typical construction materials (i.e., crushed stone). The most effective way to quantify results is to consider an actual case where specific values for length and width of the roadway are known. It is then possible to compare costs of using only imported material versus that of using coal refuse as a significant component of the roadway structure. The dimensions used for this calculation are approximately those specified for the proposed access road.

The Friar Tuck access road will consist of two segments. The New Hope loop is a section 2920 feet long which will mainly traverse deposits of coal refuse in the eastern part of the site. This section is intended primarily for access to refuse piles there and will probably not experience large volumes of traffic. The Main Road section begins at the site entrance and extends the entire distance through Friar Tuck Valley to the base of the Southwest Gob Pile. When completed, it will be approximately 5400 feet in length. This section will be the most heavily traveled and require sufficient bearing capacity to withstand higher traffic volumes and loads as discussed previously. This is the portion to be built according to the higher standard, and is the section which will be considered in terms of cost.

In order to compare the cost of using crushed stone alone with the cost of using a coal refuse/crushed stone combination, one must compare the cost of 16 inches of crushed stone to 4 inches of crushed stone. The width of the roadway is to be 18 feet, with a total length of 5400 feet. At 16 inches thick, approximately 7700 tons of crushed stone would be required for construction. At a typical cost of ten dollars per ton, this results in a total cost of $77,000. Approximately 1925 tons would be required to place 4 inches of crushed stone, at a cost of $19,250. Through the use of a coal refuse/crushed stone combination, it is apparent that a large portion of the cost ($57,750) is eliminated.

It should be noted that there would be some excavation and hauling costs associated with the use of coal refuse that would not be applicable to the crushed stone. However, even at a conservative estimate of $3/cubic yard for excavation and placement, the cost of using coal refuse would be only approximately $11,000. Thus, over $46,000 of savings would still be achieved.

Oglesby, C.H., 1975, Highway Engineering, Third Edition, John Wiley & Sons, Inc., New York, 783 p.

O'Hara, K.C., 1992, Evaluation of Coarse Coal Refuse as a Material for Access Road Construction, M.S. thesis, Purdue University, West Lafayette, Indiana, 173 p.

Peterson, W.K., 1989, "Geotechnical Assessment for the Reclamation of the Friar Tuck Abandoned Strip Mine Site, Southwestern Indiana," M.S. thesis, Purdue University, West Lafayette, Indiana, 229 p.

U.S. Department of the Interior, 1973, Methods and Costs of Coal Refuse Disposal and Reclamation, Bureau of Mines Information Circular 8576, 77 p.

Vogely, W.A., 1968, Tle Economic Factors of Mineral Waste Utilization, Proceedings, First Mineral Waste Utilization Symposium, Chicago, Illinois, pp. 7-19.

West, T.R., K.C. Kuo, and W.K. Peterson, 1990, Site Investigation of a Coal Strip Mine Area, Southwest Indiana, USA, To Provide Geotechnical and Hydrological Data for Reclamation Design, Proceedings, Sixth International Congress, International Association of Engineering Geology, Amsterdam, Netherlands, pp. 1491-1500.

Yoder, E.J., [illegible], Principles of Pavement Design, John Wiley & Sons, New York, 569 p.

References Cited

Allen, J., 1978, Derelict Lands of Indiana, Indiana Dept. of Natural Resources, Division of Reclamation, 46 p.

Annual Book of ASTM Standards, Volume 4.08: Soil and Rock; Dimension Stone; Geosynthetics; American Society for Testing and Materials, 1990.

Choi, J.C., 1992, Evaluation of Treatment Methods for Acid Mine Drainage at the Friar Tuck Abandoned Strip Mine Site, Southwestern Indiana, M.S. thesis, Purdue University, West Lafayette, Indiana, 170 p.

Choi, J.C., and T.R. West, 1992, Laboratory Testing to Evaluate the Effectiveness of Limestone and Apatite for Acid Mine Drainage Control, Proceedings, 35th Annual Meeting, Assoc. of Engr. Geologists, Los Angeles, CA.

Collins, R.J., and R.H. Miller, 1976, Availability of Mining Wastes and Their Potential for Use as Highway Material Volume I-Classification and Technical and Enviromental Analysis, Federal Highway Administration Report, No. FHWA-RD-76-106.

Dombrouski, R.P., 1985, Engineering Geology of the Friar Tuck Strip Mine Area, Greene-Sullivan Counties, Indiana - To provide basic information for surface reclamation, M.S. thesis, Purdue University, West Lafayette, Indiana, 139 p.

Doyle, W.S., 1976, Deep Coal Mining, - Waste Disposal Technolooy, Noyes Data Corporation, Park Ridge, New Jersey, 392 p.

Foster, C.R., 1965, Pavement Design Procedures, National Asphalt Paving, Association, QIP Publication No. 79.

Harper, D., Nawrot, J., and D. Fishbaugh, 1989, Reclamation Alternatives in Research and Reclamation Feasibility Studies at the Friar Tuck Site, Sullivan and Greene Counties, Indiana, First Annual Report of the Indiana Geological Survey to the Division of Reclamation, IDNR.

Holtz, R.D., and W.D. Kovacs, 1981, An Introduction to Geotechnical Engineering, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 733 p.

Kuo, K.C., 1990, Engineering Geology, Hydrogeology and Reclamation Design for the Friar Tuck, Abandoned Mine Lands Site in Southwest Indiana, Ph.D. thesis, Purdue University, West Lafayette, Indiana, 354 p.

Kuo, K.C., and T.R. West, 1990, Reclamation of A Gob Pile in Southwest Indiana, An Innovative Approach, Proceedings, 1990 National Symposium on Mining, Univ. of Kentucky Bulletin 153, pp. 181-190.

Maneval, D.R., 1974, Utilization of Coal Refuse for Highwav Base or Subbase Material, in Proceedings of the Fourth Mineral Waste Utilization Symposium, Chicago, Illinois, pp. 222-228.

National Academy of Sciences, 1975, Underground Disposal of Coal Mine Wastes, Washington, D.C., 172 p.

LANDSLIDES ON CROWLEY'S RIDGE

MCFARLAND, John David, Arkansas Geological Commission, 3815 West Roosevelt Road, Little Rock, AR 72204.

ABSTRACT
Recent study has provided a first order approximation of the landslide features, processes, and potential of Crowley's Ridge in Arkansas. Currently, most slides are related to man's activities and the majority occur along roads, in old quarries, and in areas recently logged. Stream cuts, valley walls, and loess canyons account for most of the rest. Degradation and revegetation of new slides is rapid, removing most primary indications of the features within a few months to a few years. The potential for larger earth displacements seems to increase to the south where the loesses are the thickest.

Four landslide types or combinations thereof are indicated by the features of the currently observable landslide set. They are fall, flow, slide, and creep. The falls tend to be small in volume and are most common along the near vertical walls of roadcuts and old quarries but are seen associated with the cut banks of streams and the walls of loess "canyons" and amphitheaters. Flows are a frequent component of many of the other landslide types and dominate the displacement process at a few sites. The slides exhibit block glide, rotational slump, and carpet slide forms. Creep is a likely component of most steep slopes but specifically identified as the transportive process only for certain localized strata. Some landslide sites on the Ridge reactivate from time to time by episodic displacement of the same slide blocks over a period of years or by development of new landslides in the same general area. All currently observable landslides are on slopes steeper than 20^{o}.

INTRODUCTION
Landslides in all their various forms, sizes, and causes are the most ubiquitous geologic hazard modern man faces. In the United States today landslide damage may cost over a billion dollars and twenty-five lives each year (Nilsen and Turner, 1975; USGS, 1982; Brabb, 1984; NRC Task Group, 1984). Reduction of these losses can only be accomplished by becoming more systematically familiar with this hazard in the local regions. If we can observe the patterns of occurrence, develop a deeper understanding of the processes involved, learn to recognize the factors that promote landslide development, design new facilities that circumvent the dynamics of the features, avoid susceptible sites, and enact policies that promote warning and mitigation, we can significantly reduce their impact on the citizens of our land.

Landslides are a relatively common phenomena on the Crowley's Ridge, occurring every year to a greater or lesser degree. In dry cycles the events are typically few, small, and little noticed. In wet cycles the landslides are more common and somewhat of a public nuisance. By investigating the type, distribution, earth material components, and physical parameters of the current set of observable landslides on Crowley's Ridge, this initial study has attempted to develop some insights into the potential for landsliding and modern slope vulnerability.

CROWLEY'S RIDGE
Crowley's Ridge in Arkansas is a 145 mile long (235 km), 1/2 to 11 mile wide (1 to 18 km) gently arcuate, disharmonious highland extending from the northeast corner of Arkansas north of Piggott to the town of Helena along the Mississippi River in east-central Arkansas (Figure 1). It commonly stands one hundred to two hundred and fifty feet above the surrounding topographically unremarkable lowlands and is the only significant relief in eastern Arkansas. The Ridge is traditionally considered an erosional remnant formed

during the Pleistocene when the ancestral Mississippi and Ohio rivers eroded the Mississippi Alluvial Plain (Embayment) sediments to either side (Call, 1891; West, Rutledge, and Barber, 1980; Guccione, Prior, and Rutledge, 1990). However, recent studies by VanArsdale (1991) suggest that the Ridge may have a tectonic origin in part.

The Ridge is composed primarily of weakly consolidated to unconsolidated clay, silt, sand, and lignite of Eocene age overlain by sand and gravel of Pliocene age and capped by Pleistocene loess (Saucier, 1974; Haley, 1976; Guccione, Prior, and Rutledge, 1990). However, outcrops of well lithified Eocene sand can be found in a few small areas along the west side of the northern portion of the Ridge. The loess cap thickens significantly to the south: it is quite thin and often missing from the hilltops at the northern end of the Ridge, but becomes more than 50 feet thick near the south end. The margins and slopes of the ridge are commonly draped with colluvium made up of silt, sandy silt, and gravelly silt to various thicknesses.

Many of the topographic details of Crowley's Ridge have been generally altered by man's activity. The softness of the soils encourages topographic modification upon clearing or logging, thereby destroying the details of any evidence of past catastrophic geomorphic alteration. Bulldozers are used to smooth land cleared for pasture or crop. Loggers force new trails and unwittingly change local drainage patterns via the gouges made by their log skids. Disruption or destruction of the vegetation mat allows unchecked erosion of the colluvium, many loesses, and some of the sands and gravels. Even the natural erosional processes work to the disadvantage of the preservation of landslide features. The plant cover seems to be the major retardant to erosional destruction. Where it has been disturbed soils losses are rapid and land forms ephemeral.

PROCEDURES

Low altitude aerial photographs (1:20,000-scale, winter stereo coverage, taken in 1985-90) from the Arkansas Highway and Transportation Department were used in the initial survey of the Ridge. First the photos were examined for classic landslide indications: arcuate scarps, truncated slopes, disrupted or hummocky topography, and ponded drainage. Later the photos were reexamined for any anomalous features. All discerned anomalies were noted and compiled on 1:24,000-scale topographic base maps for field checking.

As observed by Jibson and Keefer (1984) it was found that features smaller than about 200 feet in minimum dimensions, especially in forested areas, could not be successfully mapped by this technique. Somewhat smaller features could be resolved on cleared acreage. Only one unequivocal landslide was found via the aerial photography. Subsequent field surveys showed that most modern landslides are too small and too ephemeral to be explicitly captured by routine air photo reconnaissance.

With the failure of photo reconnaissance to provide guidance a program of "Ridge running" was undertaken. Initially all roads were selected, however as experience was gained, limits were placed on the territory covered. Most of these roads were driven in the Winter and early Spring prior to the Spring "leaf-out". This allowed visual observation of the landscape to either side of the road usually limited only by the topography. All significant landslide features (i.e. more than a few clods of dirt) were indicated on field sheets for later detailing. Some selected areas were retraced after "leaf-out" because the heavy rains during April, 1991 induced so many new landslides. The located features were all later revisited, most briefly described, and some sampled. Selected features were revisited several times during the course of this study to observe the rate and style of feature degradation.

A few small valleys and streams were randomly selected for foot reconnaissance. These were chosen to compare streamside, gully, and valley-wall features and processes "back in the woods" with features visible from the roads. Several of the larger lakes on the Ridge were sampled from multiple points of view. Binocular examination of the shorelines of these lakes and interviews with local fishermen provided data on lakeshore caving. Railroad right-of-ways were not investigated because of the manner in which service equipment cleared ditches adjacent to the tracks by piling excavated debris atop the right-of-way cuts thereby modifying beyond recognition any potential landslide features.

Among the laboratory tests the Arkansas Highway and Transportation Department normally uses to determine the stability of a roadbase are the Atterberg limits (Dave Lumbert, personal communication). Recognizing that the amount of water in a fine-grained soil has dramatic effects on its properties, we performed Atterberg limit tests on selected samples of loesses from various failed and unfailed sites. In order to assure that our test results would produce the same types and kind of numbers that the highway department would get themselves, we used their laboratory, tools, methods (AHTD Test Method 353 & 354), and report forms. Liquid limit, plastic limit, and plastic index were determined for samples of displaced material from 15 sites and samples of undisturbed material from 23 sites. Ignoring those samples that were indeterminate for the values tested, we found no significant differences between the failed and unfailed silts (figure 2). Samples of the collapsed material did appear to have a more limited range of test values than the unfailed materials, however.

LANDSLIDES

Non man-induced landslides are the result of several interrelated factors. Degree of slope, nature of the soils, vegetation cover, but most especially excessive water play a role in their development (Nilsen and Turner, 1975; Killey and others, 1985; Keefer et al, 1987, Baskerville, 1991). Almost all of the displacements we observed occurred on slopes steeper than 20 degrees. Most of the landslides occurred in fine-grained materials, usually loess, although some sand and gravel were the major displaced components in a few cases. Landslides on undisturbed wooded hillsides were rare; most displacements were found associated with slopes that recently had been naturally or culturally modified. Few fresh landslides were observed on Crowley's Ridge until the rainy period that began in early April (figure 3) suggesting that water is the proximal causative agent of landslides under normal conditions.

Judging from what we observed during this study, the most common landslide type to occur is the carpet slide, a shallow displacement usually involving only a few inches to a few feet of the hillside soils (Table 1). Most carpet slides occurred on the walls of gullies where thin vegetation mats of mosses, ferns, and other small plants would slide into the floor of the gully to be washed away with the next rain. Individually these slides typically carried small volumes, but in aggregate a great deal of soil was displaced. Some carpet slides did occur on wooded hillsides, but in almost every case some recent modification of the local geoenvironment could be demonstrated.

Generally following organizational guidelines proposed by Simonett (1968), Varnes (1978), Keefer (1984), and Killey et al (1985), the landslides observed on Crowley's Ridge were broadly classified principally on their character of movement and degree of internal disruption. These categories are: fall, flow, slide, and creep. Although these are clear divisions, in the real world one type often evolves into another as the landslide progresses. The materials involved in most of the displacements are similar, usually a silt soil, sometimes with sand or gravel. The term "soil" is used in this report to mean any aggregate of particles that is relatively loose and unconsolidated (or nearly so); organic content is not considered. The phrase "vegetation mat" is used to mean the upper several inches of soil bound by plant roots, the dense root network itself, and the associated aerial floral infrastructure. "Root mat" is used to describe just the root-bound portion of the vegetation mat.

FALLS. A fall is when a block of coherent sediments falls freely from a steep or undercut bank. This type of landslide was most commonly seen in active and inactive quarries, vertical roadcuts, and along the cut banks of streams. Its most common expression was as a fall of loess.

The larger of these loess falls involved "sheets" or "strips" of silt that collapsed from a cliff onto the talus slope below (normally the talus is previously fallen loess). Most of the fallen material disaggregates and becomes indiscernible from the previously fallen silt, but some holds together in blocks that may persist for a short while. Weathering soon diminishes these blocks. Where the collapse falls into a road ditch or stream, the loess is swiftly washed away. Where the loess falls onto preexisting talus the silt may cover some colonizing plants, thereby leaving a buried horizon marker.

Many of the dirt roads on Crowley's Ridge are not dressed with gravel: the roadbed is whatever the underlying material is. The repair of these roads is usually accomplished by just scraping the roadbed smooth. (In some cases, however, if the roadbed becomes very soft and if it is a well used road, a load of gravel may be applied.) Where the roadbed is made up of fine soils, these roads may become incised over the years anywhere from a few inches to over 20 feet. If deeply entrenched, they become mini-canyons with sheer walls. Soil falls are a common feature of these vertical cuts, but usually involve only a small amount of silt. The greatest hazard from this roadcut silt-fall process seems to be a temporary fill of the road edge. The roads are normally crowned in the center to force water runoff down the road edges. With this blockage the runoff is directed out into the road and into the traffic stream. Not infrequently the runoff entrenches the road bed, sometimes to impassable depths.

FLOWS. Flow is the downslope displacement of incoherent soils in the manner of a viscous fluid. Flows are a common minor component of slides and falls, but in a few cases seem to be the dominate displacive process. Flows tend to extend well beyond the boundaries of the site of failure.

Many of the flows observed were extensions of slides or falls. Frequently the internal disruption of the toe of a slide would increase to a state of near complete disaggregation into individual soil grains resulting in continued downslope movement as flow. Once flow is established continued and reactivated transport was observed to usually continue by the same manner. In some cases the viscosity of the flow was sufficiently great to form bulging lobes toeward. These lobes sometimes oversteepened and developed secondary slumps and flows. Dry falls of loess usually disaggregated upon collapse and became nearly pure dry flows.

In a few cases the failure of the slope material seemed to be dominated by flow failure. In these cases the conversion from stable slope to flow seemed to be quite sudden. Intermediate landslide types may have been involved in these processes but were short-lived and not preserved. The runout of these flows indicated that the muds were generally quite fluid and in at least one case capable of erosive action (Site 3). In each of these cases the flowing muds extended well beyond the failure site. An abundant water supply seems to be one of the most important factors leading to flowage.

Some flow-type soil failures were observed whose resultant expressions suggests other geomorphic processes. Features that mimic streamcut benches and abandoned channels were observed to be formed by the failure of the soils below the root mat and above some lower, more stable horizon. In each observed case the soils in the failed interval had apparently flowed out into a creek or ditch to be washed away. As the soils were lost the vegetation mat sank more or less uniformly downward, thus protecting the disturbed surface from outside erosion. This tended to obscure the timing of the event. There was no observed displacement of the underlying materials and usually little disruption of the vegetation mat. Even small trees settled in so well that they showed little indication that they had been disturbed and they continued to grow in a normal manner. Marginal scarps were clearly delineated, arcuate, and provided the only suggestion of a fresh surface. The revegetation of the scarp quickly masks this distinguishing landslide detail. Proper identification of these founder-features is apparently dependant on observation at a critical time period during or shortly after their formation. Water seems to be the main agent causing this type of failure. In all cases observed to date the failed material has been a loess or very silt-rich deposit with little clay. The underlying strata seem to act as an aquatard.

Small loess amphitheaters (Sites 2 & 19) were observed that appear to be the result of this foundering process. Their appearance connotes flowage of mud from a narrow outlet of a larger, foundered area upslope. As with the streamside developments, the vegetation mat subsided more or less uniformly downward with little signs of disruption. Initial drainage tended to be ponded and a few marginal trees later collapsed into the resultant depression. Large loess amphitheaters are frequently found along the southern segments of the Ridge. These observations may indicate the initial processes leading to their development are related to this foundering. Long-term observation should provide some understanding of the development of this geomorphic feature.

SLIDES. A slide is the downslope displacement of coherent masses along one or more well-developed failure planes. The slide category involves several forms of expression on Crowley's Ridge: slump, either rotational (where the units are few and commonly display backward rotation) or translational (where there are several coherent units displaced in step-like fashion); carpet slide, where the displaced material is restricted to the upper layers of soil; and, block glide, where the mass is dislocated along a very shallow plane with little internal disruption of the main block. Commonly the toes of slides became internally disrupted to the point of flowage. The most common types of landslides observed on Crowley's Ridge were carpet slide and rotational slump forms.

Most roadside failures observed were rotational slumps. These usually involved just the banks of the road cut. The most general outline of these slumps was a near equidimensional one although long, relatively low displacements occurred in several places. The near equidimensional slumps came in various sizes but all tended to be slightly higher than the feature was wide at the base. Usually the main scarp was highly arched and generally formed a continuous surface. The long, low, roadside rotation blocks extended laterally several times their height. All these roadcut rotational slumps were generally shallow and usually observed to displace only the material between the road bed and the lip of the roadcut, a distance of 5 to 25 feet. Shortly after failure the local road crews would remove that part of the slump that intruded onto the road leaving a small fresh scarp at the top of the bank as the only indication of displacement. Displacements were usually only a few inches to a few feet and were observed to become reactivated after significant rainfall. Trees up to 18 inches in diameter were seen displaced by a few of these slumps suggesting long-term stability. In general the tilt of tree trunks was not an obvious indication of recent movement. Trees often grow at odd angles or shift their orientation along these steep banks and slopes due to incipient wind loads, snow and ice loads, light direction (breaks in the forest canopy), disease, human activity, and gradual loss of supportive soils around their root systems during the life of the tree.

The carpet slide form of landslide was the most common hillside, valley wall, and gully wall expression although rotational slumps were noted in some of these places. The failures called carpet slides are shallow-based slumps that quickly degenerate basally or toeward to incoherent flows carrying the vegetation mat. This slide type is similar to Keefer's "disrupted soil slides" (1984) but seem to differ in the material displaced and the coherence of the vegetation mat. On Crowley's Ridge the vegetation mat of a carpet slide is sometimes kept nearly entire, just wrinkled a bit. At other times it is broken into rafts of various sizes.

Block glide displacements (Simonett, 1968) seem to be a rare form of landslide on Crowley's Ridge. Only one example of this form was observed and there is little doubt that it is the result of artificial conditions. The initial displacement of the Lake Austell landslide (site 46a) occurred in the spring of 1980 and was first ascribed to a shallow seismic event (Lowman, 1980; Zollweg, 1980) but later investigations dismissed this suggestion in favor of the failure of a confined strata due to increase pore pressure (McFarland and Stone, 1982; Dow and Schweig, 1990). This feature is developed on a northeast facing slope that forms the south abutment of Lake Austell Dam. Lake Austell is an 1100 acre-foot lake in Village Creek State Park near the southern boundary of Cross County. It was constructed in the late seventies and first filled in 1979. The slide developed in two phases: one in 1980 and the other at least a year and a half but not more than 48 months later. (Note: Dow and Schweig [1990] indicated a failure of the second block within a few days of the first block. This is in error.) In the first phase a large coherent block slid on a near horizontal plane out into the valley floor pushing up a series of pressure ridges. A graben developed behind this laterally translated block that vertically displaced the crest of the ridge spur some 40 feet in places. The second phase of the feature may have been initiated during this time but displayed no discernable offset for at least the next 20 months. However, by 48 months after the initial failure, another equally large block was observed to have moved in essentially the same fashion, extending the main scarp to within a few 10s of feet from the foot of the dam.

CREEP. The process of creep is a very slow displacement where downslope movement is detected only over a period of years. Practically all of the colluvium blanketing the Ridge's steeper slopes is undoubtedly undergoing some measure of incipient creep (intermixed with occasional, small to large scale, more catastrophic earth movements). Nevertheless, the orthoquartzite boulders at Site 51 and in that vicinity are the only specifically identified examples of creep in this investigation. There the large blocks of sandstone are becoming detached from the outcrop and slowly migrating downslope. The outcrops of sandstone from which these boulders are derived are quite limited due to the lack of persistence of the well lithified sands: these Tertiary sand strata may be hard orthoquartzites on one hill and only loose sand on the next. As there is no other local source of dimension stone in Green County some of the creeping rocks have been "harvested" and the outcrops quarried.

BANK CAVING

Several small lakes and numerous ponds have been built on Crowley's Ridge. To a greater or lesser degree all of these impoundments show some signs of bank caving. Bank caving is the incremental collapse of the shoreline of a water reservoir typically by fall, flow, or slide. All individual incidents of bank caving observed were small scale, but the cumulative effect has produced some problems. Where the forest comes to a lake's edge the trees are being undermined. Each year more trees fall into the water as the lake margin advances landward. Although these trees provide cover for the fish and therefore are not entirely unliked by the people who use these lakes for recreation, they are indicative of a problematic erosional process. Lakeside or pondside cleared regions erode at an alarming rate in numerous spots. All of this places a significant quantity of clay and silt into the lake. There it causes turbidity and a shallow, muddy bottom. This causes loss of lakeside recreational space, water quality, and/or usefulness as a stock pond. Also, some private ponds regularly show severe erosion of their dams necessitating frequent repairs. Riprap, hauled at significant expense, is frequently used with limited success to attempt bank stabilization on both ponds and lakes.

THE HAZARDS

As Bak and Chen (1991) point out "interactive systems naturally evolve toward a critical state in which a minor event can lead to a catastrophe." The steeper slopes of Crowley's Ridge seem to be in a critical state and even failure of a portion of a slope by landslide does not significantly change that state or the potential for other landslides. Extrapolating from the present understanding of the landslide potential of Crowley's Ridge, however, is highly speculative. Our observations of this year coupled with discussions with local residents suggest that protracted periods of rainfall will continue to provoke landslides similar in character to those reported herein. Seismic loads from events along the New Madrid Seismic Zone would undoubtedly encourage additional slope displacements, especially if the earthquake occurred during the wet season.

Roads. Luckily, most of the state highway system roads run north/south just off the ridge margins with only occasional east/west traverses across the Ridge. Only one highway was observed to have been partially inundated by a landslide during this investigation (Site 16). The banks of the highways that run along the foot of the Ridge on the east side of the middle portion of the Ridge also showed numerous small-scale displacement features. For the most part the road cuts for state highways are wide enough to accept these minor landslides without effecting the roadway and the failure potential of the roadbed by landslides seems low. Although the danger of landslide damage to the state highways seems low the same cannot be said for many county roads. Typically county roads are narrow and commonly have high, close banks. Some of the roads on the east side of the southern portion of the Ridge were blocked during 1991 for a few hours to a few weeks by landslides. Clearly, these roads could be closed again.

Communities. Most of the population concentrations are in areas of reduced slope thereby reducing the hazard potential. Vertical cuts for roads and buildings around these communities, however, do present a commonplace hazard, albeit one of very local scale. Normally there is sufficient room between the highwall and the cultural feature to absorb simple collapse or flow. No housing or commercial developments were observed on steep slopes or near any of these steeper slopes except in Helena. Broad-scale earth flows and lateral spreads were not encountered in this study but do offer a conceivable problem. Keefer (1984) notes that these types of landslides can occur on slopes of less than 1 degree. That possibility will have to await further study.

Gravel Mines. Gravel has been mined on Crowley's Ridge for as long as modern development has taken place. Increasingly the abandoned gravel mines are being used as landfill sites. White and Kyriazis (1968) suggest that the effluent from these landfills as well as septic systems and other concentrated sources of chemicals can cause certain sediments, notably clays, to become unstable increasing the risk of failure by landslide. We did not observe any suspected relationships between the two but this is a concern. Also, active gravel mines pump large amounts of water from working pits to inactive areas promoting localized anomalous hydrostatic heads. By their removal of the forest cover these mines allow localized increased rates of infiltration. Both these factors cannot help but lower the additional conditions necessary to bring about slope failure in nearby down-slope regions. One of the largest slumps we observed in this study was on a wooded slope directly below an active gravel mine (site 16).

Lakes. The growth in population along Crowley's Ridge has placed a great demand on the current water supply sources. Currently most of the water supply is from wells, but surface impoundments are being increasingly discussed by many public leaders. The Lake Austell experience clearly suggests caution in this kind of development. Additionally, the ubiquitous bank caving problem on all existing lakes causes one to wonder if seismic loads under certain other preconditions might be dramatically catastrophic to a lake.

THE FUTURE

Our study of the landslides of Crowley's Ridge has just begun. Our understanding of the landslide dangers and potentials is very rudimentary at this time. To bolster our systemic comprehension an electronic database is being established to keep track of the landslides identified in this study and other landslides to be investigated in the future. Continued visits to the Ridge are envisioned to document the rate of formation of new landslides and degree of degradation of old features. Over time this database will broaden our insight into the landslide hazard of Crowley's Ridge. An earthquake large enough to cause landslides is in our future: of this there can be no doubt. Through long term observation and data collection we will be better prepared to understand the effects of seismic loads on the hillsides, carry out post-earthquake investigations, and project those lessons to future seismic shocks.

Acknowledgments

This research was supported by a grant from the U. S. Geological Survey under the National Earthquake Hazards Reduction Program New Madrid Seismic Zone Announcement 7642 (award number 14-08-0001-G1937) and the Arkansas Geological Commission. John Hill and Nathan Adams provided field and laboratory assistance and are gratefully acknowledged for their attention to detail, insightful observations, thoughtful discussions, and above all their hard work. I wish to thank Dave Lumbert and Jake Clements of the Arkansas Highway and Transportation Department who gave advice on AHTD procedures and loaned us the use of the AHTD laboratories. Lastly, I wish to thank the staff and management of the Arkansas Geological Commission for their support.

National Oceanic and Atmospheric Administration, 1991, Arkansas Climatological Data, Vol. 96, Nos. 1-5.

Bak, P. and Kan Chen, 1991, Self-organized criticality: Scientific American, v. 264, no. 1, pp. 46-53.

Baskerville, C. A., 1991, Northern New England landslides: 42nd Annual Highway Geology Symposium, Program with Abstracts.

Brabb, E. E., 1984, Minimum landslide damage in the United States: U. S. Geological Survey Open-File Report 84-486, 4 p.

Call, R. E., 1891, The geology of Crowley's Ridge: Annual Report of the Geological Survey of Arkansas for 1889, Vol. II, 283 p.

Dow, Ronald T. and Eugene S. Schweig, III, 1990, The Lake Austell landslides, Village Creek State Park, eastern Arkansas: *in* Guccione, M. J. and E. M. Rutledge, eds., Field guide to the Mississippi alluvial valley, northeast Arkansas and southeast Missouri, Friends of the Pleistocene, South-central Cell, p. 115-126.

Guccioni, M. J., W. L. Prior, and E. M. Rutledge, 1990, The Tertiary and early Quaternary geology of Crowley's Ridge: *in* Guccione, M. J. and E. M. Rutledge, eds., Field guide to the Mississippi alluvial valley, northeast Arkansas and southeast Missouri, Friends of the Pleistocene, South-central Cell, p. 23-44.

Haley, B. R. et al, 1976, Geologic map of Arkansas: Arkansas Geological Commission.

Jibson, Randall W. and David K. Keefer, 1984, Earthquake induced landslides in the central Mississippi Valley, Tennessee and Kentucky: USGS Proceedings of the symposium on "The New Madrid Seismic Zone", November 26, 1984, Open File Report 84-770, p. 353-390.

Keefer, D. K., 1984, Landslides caused by earthquakes: Geological Society of America Bulletin, v. 95, no. 4, p. 406-421.

Keefer, D. K. et al, 1987, Real-time landslide warning during heavy rainfall: Science, v. 238, pp. 921-925

Killey, Myrna M., Jennifer K. Hines and Paul B. DuMontelle, 1985, Landslide inventory of Illinois: Illinois Department of Energy and Natural Resources State Geological Survey Division Circular 534, 28 p.

Lowman, Larry P., 1980, Seismic activity at Village Creek State Park: Little Rock, AR, Special Document, Arkansas Department of Parks and Tourism, 59 p.

McFarland, J. D. and C. G. Stone, 1981, A case history of a major landslide on Crowley's Ridge, Village Creek State Park, Arkansas: Contributions to the Geology of Arkansas, Vol. 1, pp 45-52.

Nilsen, Tor H. and Barbara L. Turner,, 1975, Influence of rainfall and ancient landslide deposits on recent landslides (1950-71) in urban areas of Contra Costa County, California: United States Geological Survey Bulletin 1388, 18 p.

Saucier, Roger T., 1974, Quaternary geology of the lower Mississippi valley: Arkansas Archeological Survey, Publications on Archeology Research Series No. 6, 26 pages.

Simonett, David S., 1968, Landslides: *in* Rhodes W. Fairbridge, ed, The Encyclopedia of Geomorphology, p. 639-641.

Task Group on Landslides and Other Ground Failures, 1984, Ground failure hazards: Ground Failure, (a publication of the National Research Council Committee on Ground Failure Hazards), no. 1, Winter 1984-85.

U. S. Geological Survey, 1982, Goals and tasks of the landslide part of a ground-failure hazards reduction program: U. S. Geological Survey Circular 880, 49 p.

VanArsdale, Roy B., 1991, Preliminary Study of the subsurface structure of Crowley's Ridge, northeast Arkansas: *Abs.*, 63rd Annual Meeting Eastern Section Seismological Society of America Program and Abstracts, p. 58.

Varnes, D. J., 1978, Slope movement types and processes, *in* R. L. Schuster and R. J. Krizek, eds., Landslides - Analysis and control: Transportation Research Board Special Report 176, National Academy of Sciences, p. 11-33.

West, L. T., E. M. Rutledge, and D. M. Barber, 1980, Sources and properties of loess deposits on Crowley's Ridge in Arkansas: Soil Science Society of America Journal, Vol. 44, No. 2, March-April 1980, p. 353-358.

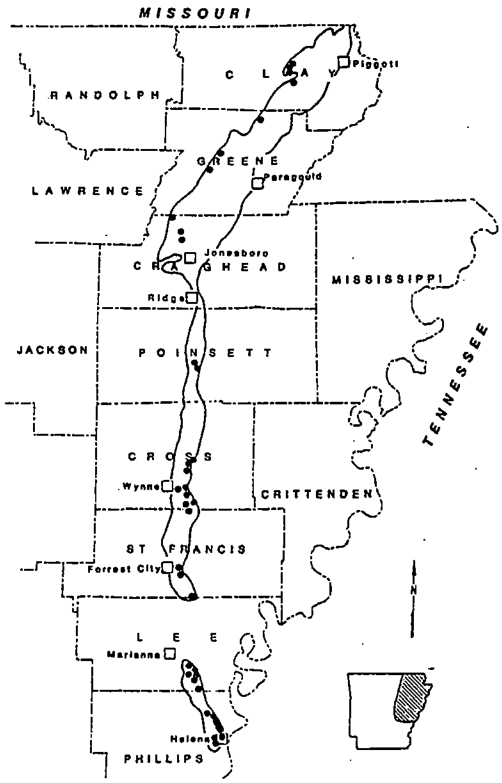
White, W. A. and M. K. Kyriazis, 1968, Effects of waste effluents on the plasticity of earth materials: Illinois State Geological Survey Environmental Geology Notes 23, 23 p.

Zollweg, J. E., 1980, Preliminary report: Lake Austell seismic events and associated problems and recommendations: Memphis, TN, Tennessee Earthquake Information Center Special Report, 3 p.

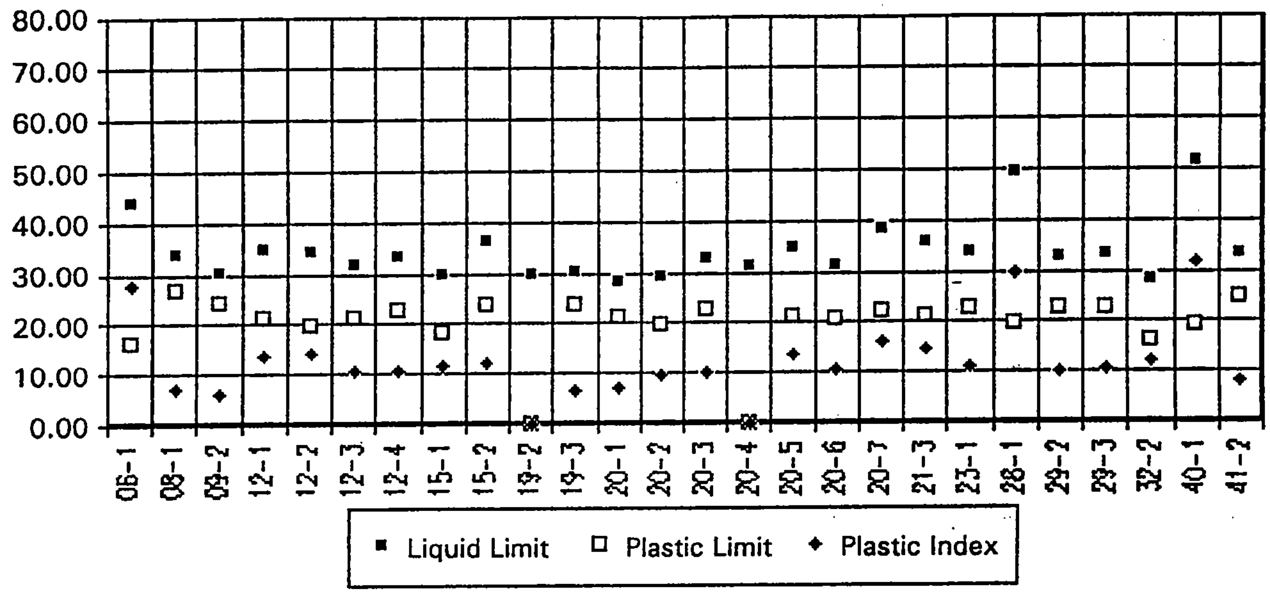
Figure 1. Crowley's Ridge, Arkansas. The open squares represent towns. The filled circles show the locations of landslides studied during the course of this investigation.

Figure 2. Atterberg limits for loess samples collected from Crowley's Ridge. The first numbers of the labels across the base of the graph are the site numbers. A) Samples of failed material. B) Samples from unfailed sites. Some samples of unfailed material were collected adjacent to landslides.

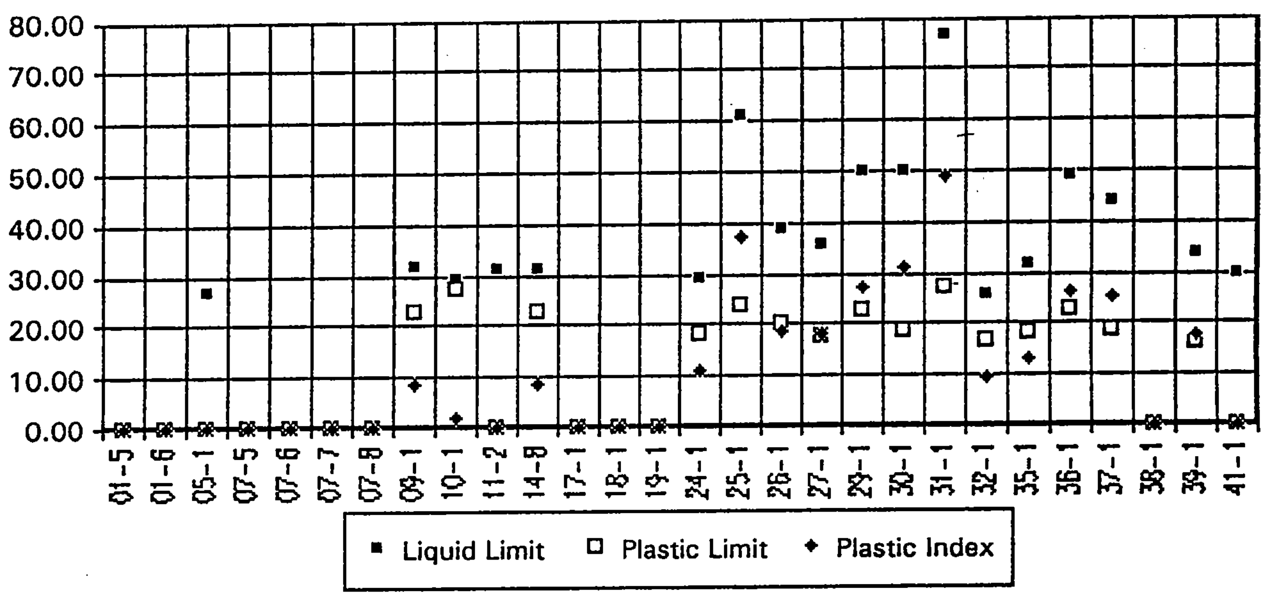
Figure 3. Weekly precipitation for Helena, Wynne, and Paragould for the first five months of 1991 (NOAA, 1991). Most of the landslides that developed this year occured during or shortly after the period of increased rainfall in April.


MISSOURI
CLAY
Piggott
RANDOLPH
GREENE
Paragould
LAWRENCE
Jonesboro
CRAIGHEAD
MISSISSIPPI
Ridge
JACKSON
POINSETT
TENNESSEE
CROSS
Wynne
CRITTENDEN
ST FRANCIS
Forrest City
N
LEE
Marianna
Helena
PHILLIPS

Loess from landslides
Crowley's Ridge



Loess NOT from landslides
Crowley's Ridge



Rainfall 1991
Rainfall by Week

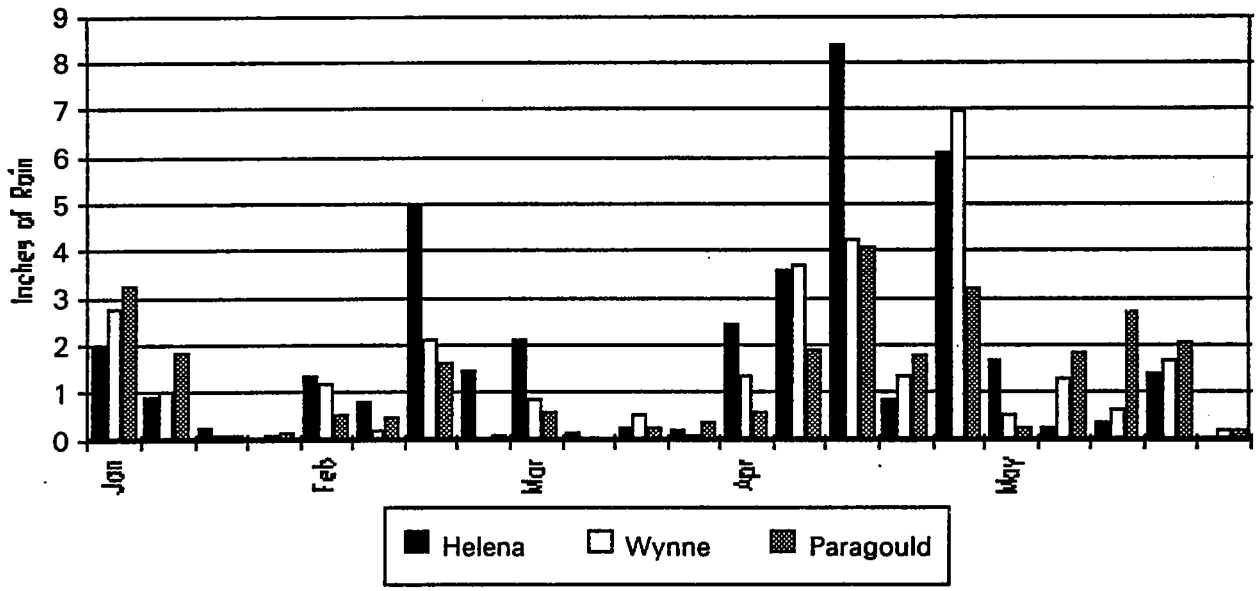


TABLE 1
1991 Landslide Site Locations on Crowley's Ridge

Site #	County	Quad	UTM Location	Type
1	Phillips	Helena	720830e/3827370n	slide
2	Phillips	Helena	720795e/3827670n	flow
3	Phillips	Helena	720220e/3829290n	flow
4	Phillips	Helena	720060e/3829745n	slide
5*	Phillips	Helena	719640e/3824065n	fall/flow
6	Phillips	Helena	720040e/3823230n	fall/flow
7*	Phillips	Helena	720470e/3828630n	flow/slide
8	Lee	LaGrange	712120e/3841780n	fall
9*	Lee	LaGrange	712910e/3843650n	slide
10*	Lee	LaGrange	712450e/3844820n	slide/flow
11a	Lee	LaGrange	712340e/3845180n	slide
11b	Lee	LaGrange	711970e/3846060n	slide
11c	Lee	LaGrange	711910e/3846070n	slide
11d*	Lee	LaGrange	711000e/3846860n	fall
12	Lee	Dansby	709940e/3864270n	fall
13	Cross	Wittsburg	708120e/3902100n	flow?
14	Greene	Walcott	711040e/3989770n	slide
15*	Cross	Wittsburg	706080e/3900670n	flow
16	Cross	Wittsburg	708760e/3900380n	slide
18	Phillips	LaGrange	716715e/3834880n	slide
19*	Lee	LaGrange	714735e/3835910n	slide/flow/fall
20*	Cross	Wittsburg	710530e/3896300n	fall
21	Cross	Princedale	708490e/3907170n	slide/flow
22*	Cross	Princedale	708540e/3910860n	slide/flow
23	Poinsett	Harrisburg	712040e/3933300n	slide
28	Craighead	Lorado	705240e/3977600n	slide
29*	Craighead	Lorado	704860e/3978930n	flow/slide
32	Greene	Delaplaine	713100e/4003650n	slide
38	Greene	Lafe	724480e/4013560n	slide
40	Clay	Boydsville	734740e/4024290n	slide
41	Clay	Boydsville	734110e/4027600n	slide
42	Clay	McDougal	734165e/4028520n	slide
43	Clay	McDougal	734800e/4029070n	slide
45	Craighead	Bono	701690e/3980920n	flow
46a	Cross	Wittsburg	707900e/3892150n	slide
46b	Cross	Wittsburg	707880e/3893040n	slide/flow
46c	Cross	Wittsburg	707980e/3893340n	slide/flow?
46d	Cross	Wittsburg	708090e/3893870n	slide/fall
47	Cross	Princedale	708680e/3812450n	slide
48	Phillips	Helena	721220e/3824950n	flow
49*	St Francis	Madison	706530e/3875370n	flow/slide/fall
50*	St Francis	Dansby	706400e/3875000n	slide/flow
51*	Greene	Walcott	712030e/3999670n	creep
52Lsc	Phillips	Helena	719000e/3830500n	flow/slide/fall
52Lbc	Lee	LaGrange	711000e/3843000n	flow/slide/fall
52La	Cross	Wittsburg	707500e/3892200n	flow/slide/fall
52Ld	Cross	Wittsburg	709000e/3894000n	flow/slide/fall
52Lp	Poinsett	Harrisburg	711000e/3934000n	flow/slide/fall
52Lf	Greene	Lorado	706000e/3984000n	flow/slide/fall

Table 1. These selected sites are the locations of most of the more significant of the many landslide features identified by this study. Many small roadside, stream cut, and gully-wall landslides were noted but due to their omnipresence and insignificant nature they were not given a site number. Other sites that were used to obtain stratigraphic enlightenment, soil grab samples, or some other non-landslide background observation were given site numbers in the course of this investigation but are not listed here. The sequence of numbers tends to skip around geographically due to the time of the slide, time of discovery, order of investigation, and/or caprice. The UTM locations presented here are from the 1000-meter Universal Transverse Mercator grid, zone 15, 1927 North American Datum. (The changes in UTM locations brought about by the 1983 North American Datum grid will alter the locations listed here by less than 10 meters.) For an extended site (indicated by *) the location given is usually some central point and may include more than one landslide feature. An "L" followed by initials after a site number indicates a lake.

SLOPE FAILURES ON HIGHWAY 71 RELOCATION PROJECTS

I-40 - FROG BAYOU

by

JONATHAN ANNABLE

AND

JOHN SHARUM

presented to

43rd HIGHWAY GEOLOGY SYMPOSIUM

FAYETTEVILLE, ARKANSAS

AUGUST 19-21, 1992

INTRODUCTION

The Arkansas State Highway and Transportation Department is currently in the process of designing and constructing a 42-mile relocation of U.S. 71 in the northwestern part of the state between Interstate 40 (Alma) and Fayetteville (see Figure 1). Since the highway is to be constructed to interstate standards, it will be the first such type of road to traverse a very mountainous area of the state. Consequently, the new highway will require the highest fills and deepest cuts ever constructed in Arkansas.

The relocation is divided into several construction segments. This paper will discuss Geotechnical, Geology, Construction and Slope Design information for Job Numbers R40032, R40033, R40075 and R40086. This will include grading and structure projects that have been completed from I-40 to Frog Bayou, Station 0+00 to Station 432+00, (see Figure 2).

The Department conducted a subsurface investigation for each project to provide sufficient information of soil and rock quality for backslope recommendations, embankment designs and for estimation of earthwork quantities. Special notes were made of springs and marshy areas where observed.

GEOLOGY

Job numbers R40032, R40033, R40075 and R40086 lies within the northern limits of the Arkansas Valley (Ouachita Province) and the foothills of the Boston Mountains (Ozark Province) (see Figure 3). The Boston Mountains stand higher than any of the other sediments of the Ozark Province and

HWY. 71 RELOCATION AREA

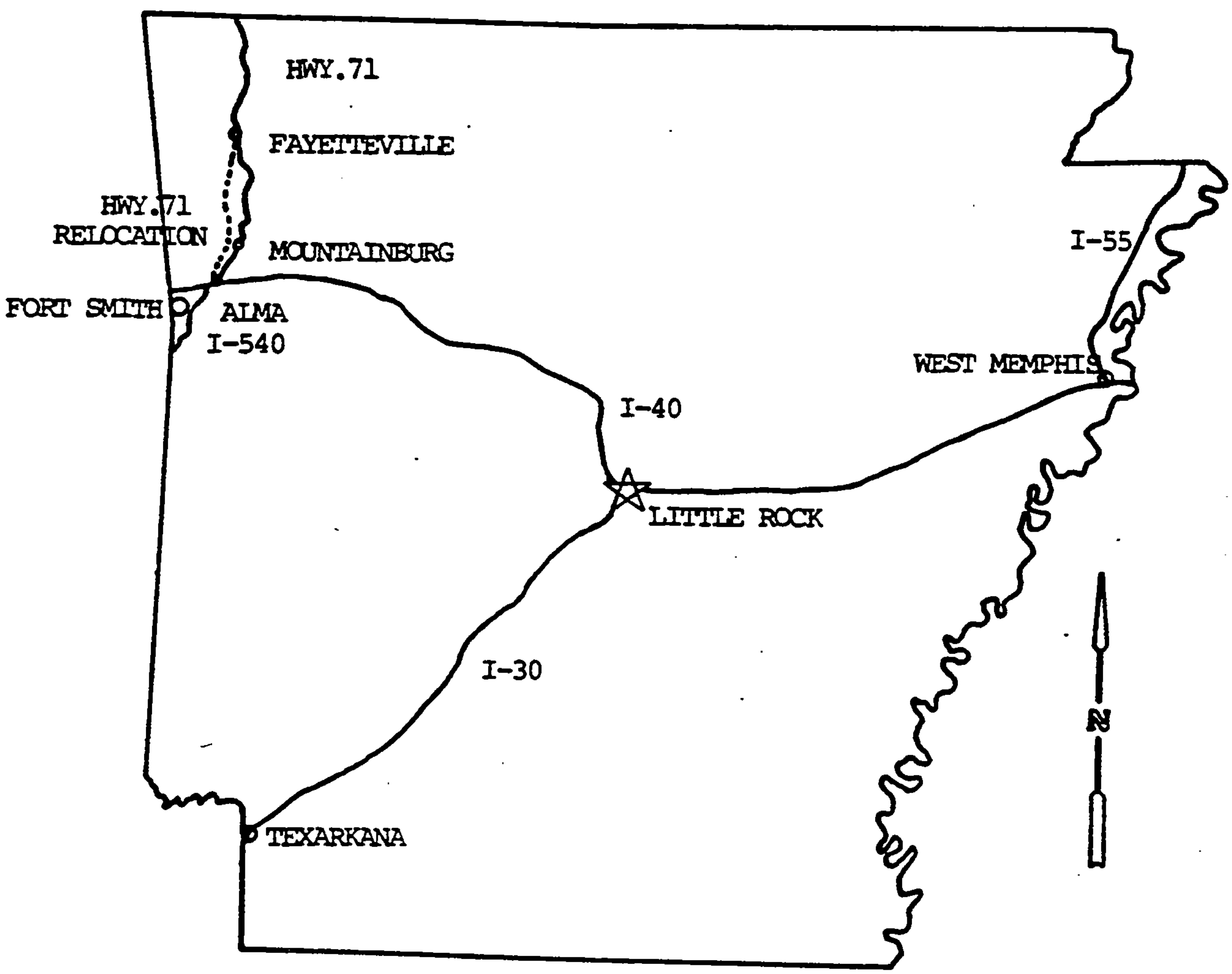


FIGURE 1

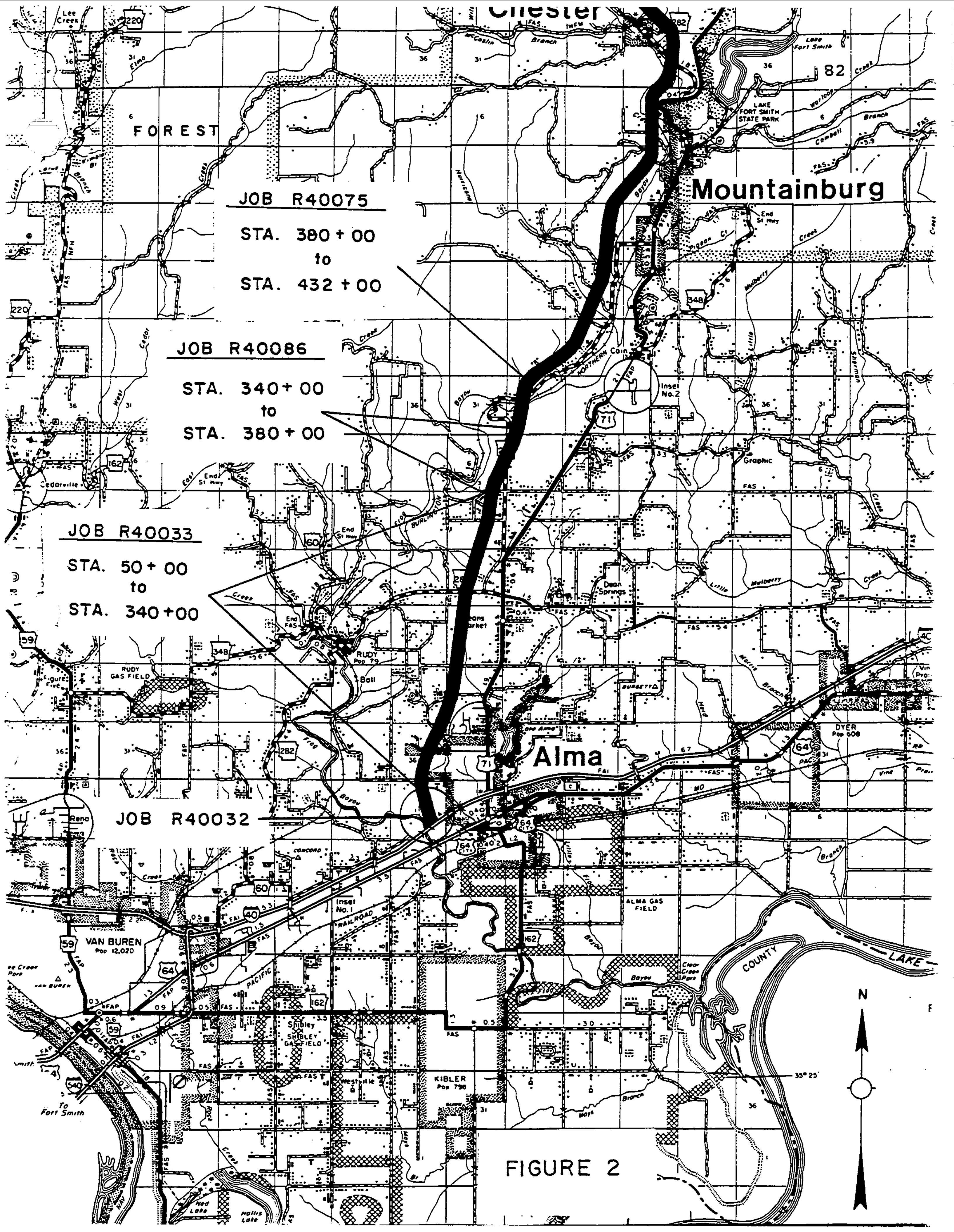

Chester
Mountainburg
Alma
FOREST
JOB R40075
STA. 380 + 00
to
STA. 432 + 00
JOB R40086
STA. 340 + 00
to
STA. 380 + 00
JOB R40033
STA. 50 + 00
to
STA. 340 + 00
JOB R40032
LAKE FORT SMITH STATE PARK
Lake Fort Smith
Cedarville
RUDY
RUDY GAS FIELD
Dean Springs
Graphic
DYER
VAN BUREN
ALMA GAS FIELD
SHIBLEY GAS FIELD
KIBLER
Westville
To Fort Smith
COUNTY
LAKE
N

FIGURE 2

rise to an average height of 2,400 feet east of the relocation. The mountains now constitute a long, highly dissected plateau with floors and valleys of main streams 500 to 1400 feet below higher elevations. The beds that form the mountains have experienced only a small amount of folding and faulting (see Figure 4).

The section of rock outcropping throughout this area is the Atoka Formation of middle Pennsylvanian time. The Atoka consists mainly of dark silty shales, very light to medium gray sandstone, and light to dark gray siltstones. Also, a few dark colored, fossiliferous, sandy limestone and limey sandstones are found in the Atoka throughout Crawford County.

Faults in this report area are normal and dip generally south at 45 . Maximum displacement across the faults within the Atoka ranges from 10 feet to 2,500 feet. However, due to the lack of deep well information in this area, displacements of mentioned faults are difficult to predict. The Geologic Commission indicates that northwest Arkansas is seismically stable and that no movement is expected along the very old faults present.

The faults located within these job segment areas are discussed as follows:

<u>Station 79+00</u>

A normal fault crosses the centerline at station 79+00 which is in a fill section. Just upstation, natural weathering processes have affected the rock quality more than any damage from previous fault movement.

MAJOR PHYSIOGRAPHIC REGIONS OF ARKANSAS

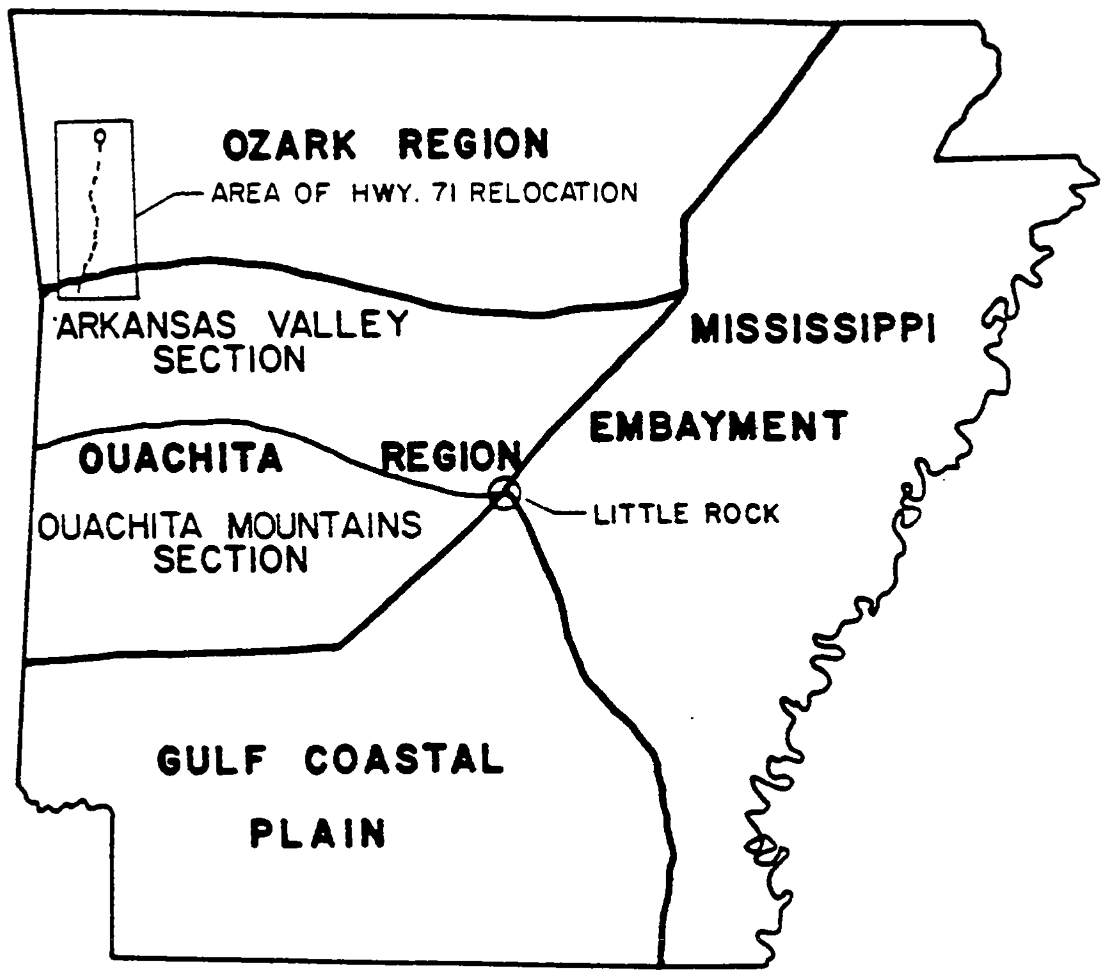


FIGURE 3

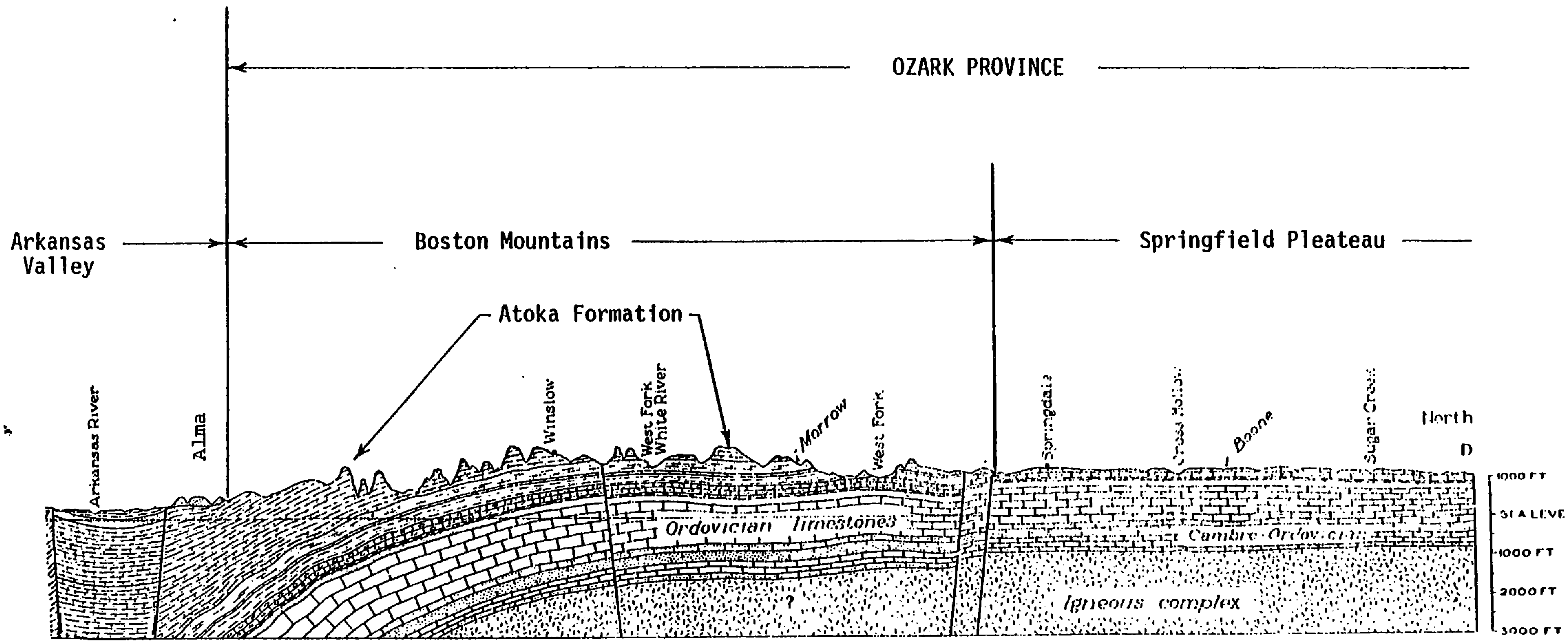


Fig. 4 North-south section showing structure of the Paleozoic area of the Boston Mountains (From Croneis)

Station 140+50

Two faults cross the centerline at approximate station 140+00. These faults cross within a fill section at Rudy Road Cutoff. At the time of the investigation, the area around the Rudy Road Cutoff is quite boggy and it may be that a significant amount of water is seeping along the fault zones.

Station 394+00

A normal fault crosses the centerline at the above station. This fault dips generally to the southeast at about 45 degrees. Due to the absence of any deformation of cretaceous rocks within this area which overlie the Paleozoics suggests no movement since Pennsylvanian time.

The primary importance of this station is the nearness to the south end of the bridge, crossing Frog Bayou, Burlington Northern Railroad and Highway 282. The bridge ends are between Stations 399+18 and 422+06. Also, during construction, cut slope failures occurred within this area and are discussed in the construction segment of this report.

FIELD AND LABORATORY INVESTIGATION

The drilling program was outlined by utilizing the preliminary plan, profile and cross-sections sheets furnished by Roadway Design Division. Selected Fill and Cut sections were investigated utilizing 4-inch continuous flight augers, hollow stem augers and rotary wash borings. Preliminary descriptions of the materials encountered were recorded in the field but all samples recovered were brought to the laboratory for more complete evaluation and selected testing.

Numerous field inspections were conducted by the geotechnical staff with special care taken to note springs and unusual soil conditions. Field inspections were conducted at other sites within the Boston Mountains in order to observe similar fills and cuts in the Atoka Formation.

All of the samples obtained were brought to the laboratory and visually classified to serve as a recheck by experienced lab personnel of field identification, but mainly to provide better uniformity of descriptions. Physical limit tests were conducted on selected samples obtained from the soil survey. Strength tests and Slake Durability Tests were conducted on various representative samples and recommended R-values for subgrade support.

GENERAL DESIGN RECOMMENDATIONS

The rock strata in the Boston Mountains are basically level or only slightly tilted, rock cut backslope stability is controlled by rock quality and water seepage. It is well known that in the Atoka Formation, individual rock beds can "pinch out" within a very short distance. Also, as previously mentioned, there are several major faults and many small ones common in these job segments. The natural "pinching out" and faults make strata correlation very difficult even with closely spaced borings.

The bedrock encountered within these job segments was composed of shales and sandstones with interbedding common. Because of the interbedding, sandstone fragments were usually present throughout the residual soil overlying bedrock. The sandstones proved to be quite permeable which contributed to the weathering process to a considerable depth. Sandstone quality varied widely but they seemed to be most durable when covered by shale. When not covered and especially at the hill sides, the sandstones tended to be highly weathered.

It was much more difficult to assess the quality of the shales. As previously mentioned, the permeability of the sandstones contributed to the weathering of shales to a considerable depth. Even seemly hard shales are susceptible to deterioration after being exposed to the elements. This deterioration may not take place immediately but eventual stability problems are well documented nationwide.

For recommendations to construct the cut slopes for shales three types were defined:

1. Soil Like - Shale that is rippable (Highly Weathered)
2. Intermediate - Shale that may be rippable (Weathered)
3. Rock Like - Shale that will require blasting (Medium Hard to Hard)

Backslope cut heights are considered to extend from the highest point of the cut of natural slope to the bottom of the ditch line as shown in Figure 5. Durable sandstone or rock-like shale can be cut on near vertical slopes (1/4:1 for Hard Sandstones and 1/2:1 for Hard Shale with SDI>95). Intermediate shales with SDI 50 to 94 are to be set on slopes no steeper than 1:1. Soil-like shales (SDI<50) and other soil types were set on slopes no steeper than 3:1.

Soil and soil-like shales were recommended to have

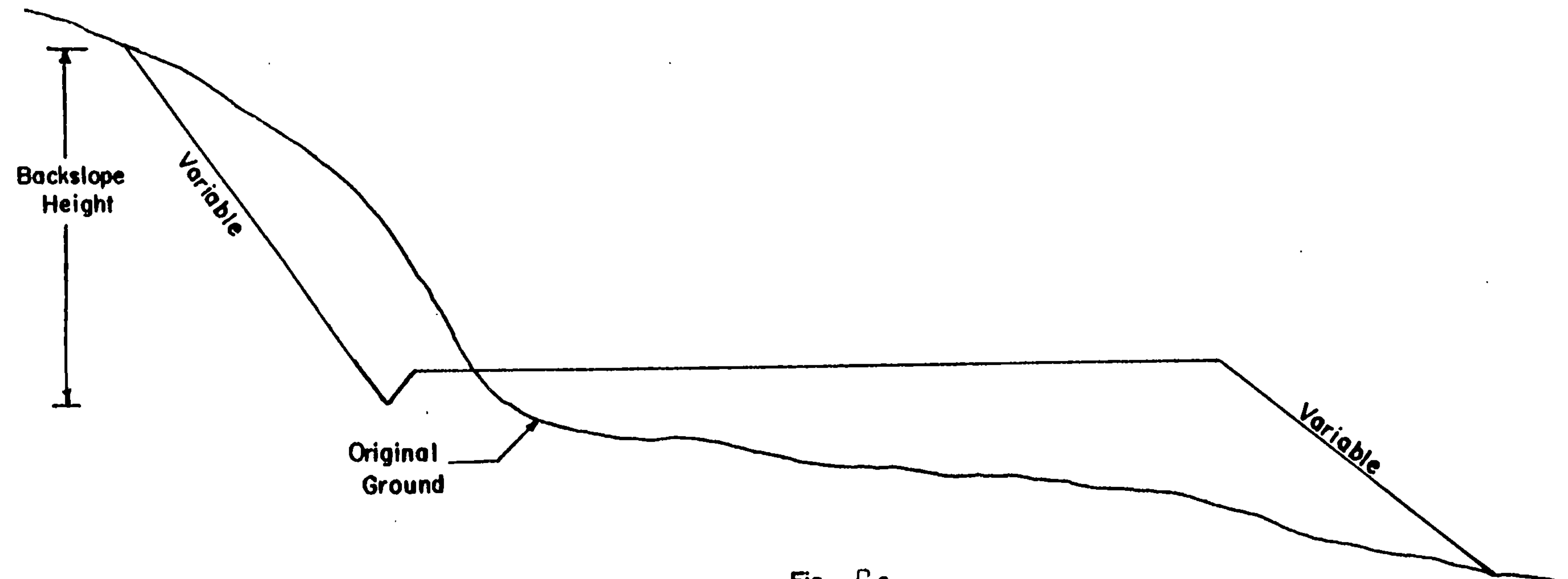


Fig. 5a.

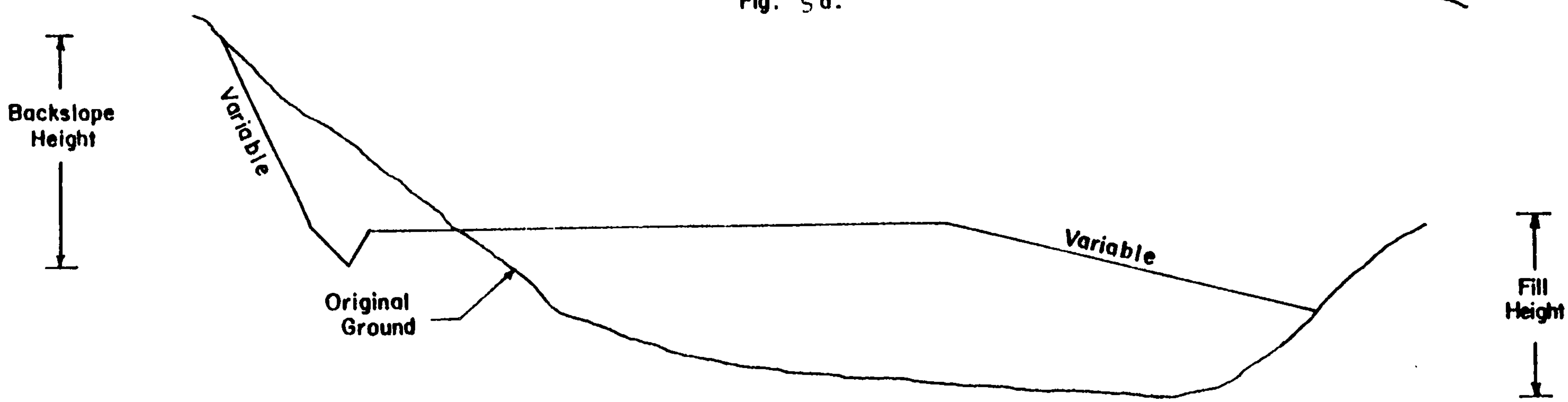


Fig. 5b.

FIGURE 5

minimum bench widths of 10 feet with all other rock types with a minimum 15 foot width. Ten foot catchment basins were recommended at the ditch line. Bench height recommendations ranged from 25 feet for rock-like shale to 50 feet for medium hard to hard sandstones.

Fill heights are considered to extend from the highest point of the filled roadway to the lowest point of the intersection or lowest point of the original ground below a fill area. Fills of 25 feet in height or less can be composed of unspecified materials and should be stable when designed for 3:1 or flatter slopes (see Figure 6). Only granular material is recommended below the 25 feet of unspecified material. The granular material can be divided into two types. The first type (Type I) includes the rock-like shale, durable sandstone, and alluvial sand and gravel. The second type (Type II) of granular material includes the intermediate shale and nondurable sandstone. Type I granular material is recommended for drainage of sidehill, transition sections and typical high embankments over 25 feet in height (see Figures 9, 10 and 11). The Type II granular material can generally be utilized up to a maximum of 50 feet but will require a minimum of 3:1 slopes. The Type I granular material can be placed on 2:1 side slopes and can always be utilized as replacement of Type II material . The unspecified material and Type II material when set on 3:1 side slopes should be plated with top soil to promote the growth of vegetation. The 2:1 side slopes for the Type I material

TYPICAL UNSPECIFIED FILL SECTION

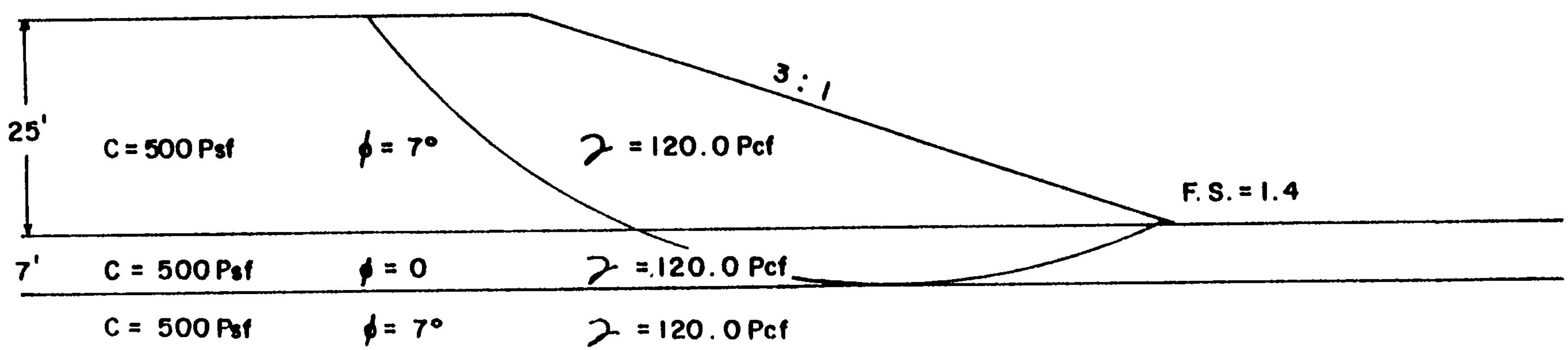


FIGURE 1.6

should not be plated because it would hinder internal drainage. A typical High Fill is illustrated in Figure 7.

A major concern of the highway industry when shale is utilized as an embankment material is the potential, long-term deterioration of the material especially when exposed to water. Shale embankments which have experienced the least problems are those where sufficient drainage and adequate compaction is provided. Therefore, a minimum of 5 feet of the Type I material was utilized underneath the Type II material for embankments over 25 feet high in order to prevent saturation of the fill.

Springs and seeps, both intermittent and year-round, are common within the job limits. Although every effort was made to note them during preliminary design stage, provisions were made in the design for unexpected water sources encountered during construction. This would include additional quantities of Type I material to be used at the discretion of the Engineer.

Construction Techniques and Problems

In the design stages of the projects the cut sections specified typical slope configurations. It was decided that it would be impossible to slope stake the project and insure the final slope configurations would match the necessary slopes required for the stability of the material based on SDI tests performed on the shale and the strength of the sandstone. A method of exposing the geological features was added as a note to the plans that required the contractor to

TYPICAL HIGH FILL

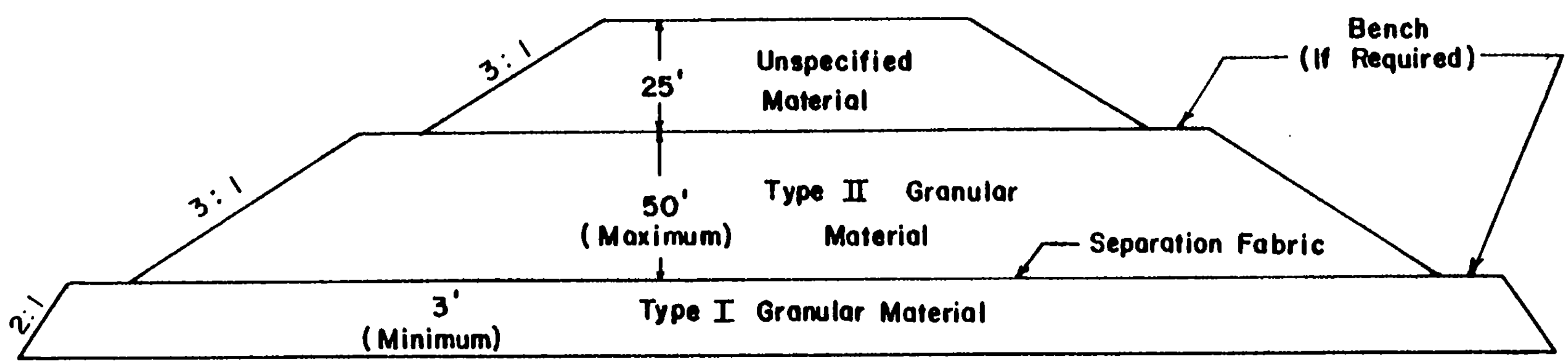


FIGURE 7

excavate a nearly vertical face at the ditchline. This allowed inspection of the exposed material for each cut section and the slope configuration that was deemed most suitable was established and then the slope was excavated. With all this effort to provide the ideal slope configuration we were still constrained in several instances by the available right-of-way.

Contractor concerns about the safety of excavating nearly vertical slopes on future projects with the cuts approaching 200 feet in depth were considered. It was decided to modify the requirements for exposing the geological features by allowing alternatives which would include drilling at the ditchline on a per station basis.

The fill sections of a height greater than 25 feet called for a special construction feature in an attempt to reduce settlement and potential for slides. This feature incorporated a drainage blanket that is three to five feet thick and composed of a Type I granular material. The Type I material was composed of a sandstone with no boulders greater than 18" or a shale with a SDI greater than 95. The fill above the Type I material to a point 25' feet below the final grade was composed of a Type II material. The Type II material could be composed of a shale with a SDI of 50 to 95 or any rock-like material. If the shale with a SDI of 50 to 95 was utilized a special processing was required that would help accelerate the breakdown and facilitate compaction. This involved implementing a gang disk with a minimum wheel diameter of 24 inches. This disk is used to uniformly incorporate the water throughout the entire lift. The amount of water equalled the amount necessary to achieve a moisture content of optimum +/- 2.0% as determined by AASHTO T-99. This moisture content requirement had equal weight with the density requirements when determining the acceptability of a lift. Each lift received a minimum of 3 passes with a static roller followed by blading and a minimum of 2 passes with a vibratory roller. The minimum density was 95% of optimum.

In the construction process for these cut and fill sections one problem encountered required a coordination of the excavation and embankment construction on the project beyond what is normally required. The material for the Type I and Type II materials was at the bottom of the excavations. The materials needed for the bottom of the fill sections was the Type I and Type II. In order to keep from moving the excavated material twice the contractor had to remove the overburden in the cut sections with the Type I and Type II at the bottom and utilize this overburden in fills that were less than 25 feet in height. This sometimes required a longer haul than would normally be required. This was recognized as a potential problem and therefore overhaul was stipulated as a subsidiary item and not paid for directly. This allowed the contractor freedom to utilize the material as he best felt it could be used. Because of this requirement for special materials it also required an educated guess at how much of each material was available and a balancing act to insure the proper material was available for the proper position in the fill.

The most extensive slide in our projects at the present time is from station 368 to 395. The original plan typical section was starting at the ditchline a 1/2:1 approximately 25 feet tall with a 20 foot berm then a 1:1 for approximately 20 feet height and then a 3:1 to the top of the slope. Upon examination after the exposure of the geological features the slope configuration was changed to a 1-1/2:1 approximately 35 feet tall and a 2-1/2:1 or 3:1 to the top of the slope. This slope configuration was restricted by the right-of-way for this area and would have been constructed using flatter slopes if that had been possible. The slide started at an area were a pond had existed at station 390 that was drained. The pond was a spring fed and the area remained wet after the pond was drained. The slide repair for this immediate area involved constructing a special 3'x 2' rock drain composed of a class B concrete aggregate and a filter fabric in a ditch on the slope to intercept this water and take it to the roadway ditch.

In repairing these slides there were several problems to overcome. One of these problems is how to excavate the slide prone material and keep it stable enough to build the rock buttress. The most obvious solution to this problem is to excavate this material in the summer months when the rainfall is minimal and therefore keep the slope stability at it's highest potential. In the area of the spring fed pond an extra effort had to be made to intercept this spring in order to increase the stability of the repaired slope.

The next problem is availability of material to construct the rock buttress. The highest quality material would be a shot rock from a commercial quarry site but due to the hauling cost this proved to be an extremely expensive option. For the slide repairs made from stations 374 to 395 the projects that we had ongoing in this area had enough rock remaining to construct these buttresses. This material had a slightly lower quality than the commercial source but not enough to be detrimental to the construction and durability of the buttress. The future slide repairs at stations 368 to 374 and stations 343 to 347 do not have this advantage of having a project with rock excavation. The material to repair these slides will have to come from a commercial quarry or a borrow pit site. These slides are to be repaired during the construction of the pavement. The contractor for this project is attempting to start a quarry at a borrow site they have established at the diamond interchange for highway 282 in the Deans Market area.

Another problem in repairing the slides is access to the slide area. The typical sections for these cut areas involved a two berm cross-section with the bottom berm of either shale or sandstone usually on 1:1 slope with a height of approximately 20 feet and then a soil slope on a 2-1/2:1 on top. The soil slope is where the stability failure has occurred and is creating a mud flow down the face of the lower berm to the ditchline of the main lanes. The difficulty is the impossibility of running equipment up and down the 1:1 slope on the lower berm. The solution to this problem required the construction of a haul road in front of the lower berm and then removing this road after the slide repair is complete. The method of construction on these highway 71 projects has proven to be a very wise choice by constructing the grading and structures projects first and then performing a pavement project at a later date. This method allows us to discover any maintenance problems and repair them with the pavement contract and not have to contend with a road that is already under traffic and the inconvenience to the travelling public by having to close portions of a roadway.

These slides are not something that could accurately be predicted. Their exact locations were not able to be predicted.

On future projects I understand that we will be acquiring right-of-way with additional width greater than the current projects in order to allow a little more flexibility in selecting slope configurations. The availability of Type I and Type II material, the depth of cuts and fills, and the terrain and geology of the new alignment will continue to make the new Highway 71 projects an engineering challenge.

SLOPE MAINTENANCE AND SLIDE RESTORATION

BY

Andy Muñoz, Jr., P.E.
Geotechnical Engineer
FHWA

The maintenance of slopes and the correction of slides has been identified by many highway agencies as a major and continuing problem involving considerable expenditures of funds. Each year U.S. highway agencies spend millions of dollars in maintaining highway embankments, slopes and other earth structures, removing rock falls and soil debris from roadways and repairing landslides. Activities from maintaining highway slopes and restoring landslides often cause traffic slow down and stoppage that creates serious safety hazards and consumes significant highway maintenance and construction funds. In addition, economic losses due to the inconvenience to the traveling public is often immeasurable.

A training course has been developed by the Federal Highway Administration (FHWA) to provide guidelines and aid maintenance supervisors in making decisions regarding highway slope movements and maintenance activities and correction of small slope failures.

An executive summary of this one-day NHI course, titled "Slope Maintenance and Slide Restoration," is presented. This course presents a basic explanation of why and how slopes fail, provides information on recognition and identification of slope distress features for early identification of problems, identifies maintenance activities to prevent and minimize slope failures and discusses stabilization methods for economic repair of small slope failures.

A manual, titled "GUIDELINES FOR SLOPE MAINTENANCE AND SLIDE RESTORATION," is available from the FHWA. This is Report No. FHWA-TS-85-231.

REPAIRS TO ROCK SLOPES AT THE US 22/SR 7 INTERCHANGE IN STEUBENVILLE, OHIO

By J. R. Graham, Ohio Department of Transportation
P. C. Ingraham, Golder Associates Inc.
R. W. Humphries, Golder Associates Inc.

INTRODUCTION

The Ohio Department of Transportation is constructing a major interchange for Ohio Route 7 and U.S. Route 22 on the west side of a new cable stay bridge over the Ohio River in Steubenville, Ohio. This project will complete the $138 million Steubenville bypass program which is shown in Figures 1 and 2. The roadway is functionally classified as urban expressway with a design average daily traffic estimated at 44,060 vehicles per day.

The construction of the interchange includes seven tieback walls, three bridges, a tunnel, the construction of 125,000 square yards of concrete pavement, and repairs to a 250 foot high, 3,000 foot long section of steep rock slopes. One of the retaining walls, Wall No. 5 is one of the largest tieback anchored walls in the world. It has 2140 tieback anchors with capacities up to 245 kips and free lengths up to 115 feet, a maximum height of 130 feet, a length of 2840 feet, and a pressure relief tunnel to reduce groundwater pressures on the wall (Reference 1).

GEOLOGIC SETTING

The natural slopes in the vicinity of the project on the west bank of the Ohio River are steep, forming an escarpment with slope angles of up to 45 degrees, slopes heights of 350 feet, and thin soil cover.

The project is located in the Appalachian physiographic province which is characterized by thick cyclothemic sequences of sedimentary rocks of Pennsylvanian age. The outcrops at the site belong to the Conemaugh Group and are composed of thick, flat lying sequences of sandstones and shales, with minor coal seams and limestone beds.

A geologic cross-section through the hillside at the north end of the project site is presented in Figure 3. The Morgantown and Buffalo sandstones are generally massive, hard and durable, while the Cow Run sandstone is less massive and its thin bedding and intercalated siltstone laminations prominent. The shale units throughout the slope vary considerably from hard, massive gray shales to weaker red shale units which have well defined bedding partings and some slickensided joints. The coal units are associated with thin layers of underclay. All the shale units slake rapidly when exposed on the surface. The Cow Run sandstone weathers significantly over time, while the Morgantown and the Buffalo sandstones are relatively weather resistant.

The main structure of the rock at the site is the horizontal bedding. There are two subvertical orthogonal joint sets which strike at approximately 45 degrees to the road alignment. A third joint set, prevalent throughout the site, was formed by stress relief parallel to the river. These stress relief joints dip at between 70 and 85 degrees. Schematic views of the rock structure are shown in Figure 4 and Photograph 1.

NATURE OF THE ROCKFALL PROBLEM

As mentioned previously, the sandstone units are generally resistant to weathering while the shale units slake significantly. The net result is differential weathering of the steep natural slopes which causes undercutting of the more competent units and the development of overhangs in these sandstone units. When the undercutting process proceeds to a point where the intersecting orthogonal joints or the stress relief joints are exposed, failure of blocks occurs as indicated in Figure 4 and Photograph 1. Rockfalls of this nature have occurred from the Morgantown sandstone, the Cow Run sandstone, and the Buffalo sandstone.

Construction of this section of highway in the 1950s entailed steepening of the slopes below the Cow Run sandstone and construction of the pavement immediately at the toe of the slope. There are consequently no natural benches below the Cow Run sandstone and very little catch ditch width. Therefore, blocks that have fallen from the Cow Run and Buffalo have fallen directly onto the highway. While the Morgantown sandstone was not affected by the 1950s construction, there have been recent rockfalls, predominantly in the southern half of the project, from this unit also. However, many of these blocks have stopped on flatter slope benches immediately below the Morgantown or have been arrested by large trees on the lower slopes.

Three separate areas were identified on the rock slopes of the project that need to be addressed separately. These were:

- The northern half of the project where the slopes were steepened in the 1950s and recent rockfalls have occurred from very steep cut slopes. The rockfalls were generated mainly from the Cow Run sandstone but also in the Buffalo sandstone. This area represented the most significant hazard to traffic and was assessed to have the highest priority for repairs.

- In the southern half of the project where the natural slopes are shallower overall because of the presence of benches in the Cow Run and Buffalo Sandstone formations, a number of blocks of Cow Run sandstone have broken loose and shifted a few feet downslope on weak underlying shales. The rock in this area is very friable and the blocks are likely to break up if they move further or may topple and cast rocks downslope if they are left in place.

- The thick Morgantown Sandstone Formation that forms steep bluffs at the top of the slope has a number of very large overhangs that cantilever up to 25 feet over the slopes below. There is evidence of recent rockfalls onto the benches immediately below the Morgantown. Some of the falling blocks have rolled to the base of the slope down steep drainage swales though many have been stopped by trees and by natural benches.

ROCK BOUNCE ANALYSES

Significant advances have been made in the development of rock bounce simulation programs in the last few years. Currently, the most widely used rockfall program is the Colorado Department of Highways Rockfall Simulation Program (CRSP). This is used by many departments of transportation and design professionals and has been checked against a number of field tests in Colorado.

The CRSP program yields very similar results to a program developed by Golder Associates, ROCKFAL2, (Reference 2) however, the ROCKFAL2 program is better able to simulate surface roughness and tangential forces and angular momentum of falling blocks.

For the Steubenville project, both programs were calibrated using observed rockfall displacements downslope and rock bounce heights based on measured scars on trees. Multiple simulations were run on both the CRSP program and the ROCKFAL2 program. The analyses included simulations of rock falling from the Morgantown and the Cow Run sandstone units at a large number of cross-sections for several slope configurations. Both existing slope, construction phase slope and remediated slope configurations were simulated with varying sizes of rock blocks. These analyses have proved useful in assessing the viability of alternative configurations of slope recontouring and requirements for protecting existing works during construction. However, the science of rock bounce simulation is still in its infancy and it was recognized that results of the simulations should be used to guide the design and not to finalize the design. Final design was tempered by observations of the past rockfall behavior on the natural slopes which included:

- The arrest of most large blocks and blocks with length to width ratios of more than 2 on trees, benches or other fallen blocks downslope;
- The preferred rockfall path comprising drainage swales; and
- The predominant rock block size reaching the roadway of less than 2 feet nominal dimension.

The lack of vegetation and benches, and the poor condition of the rock at the north end of the project necessitated significant remediation. It was recognized for the southern slopes, however, that significant rockfall protection was afforded by the natural slope conditions and only minimal enhancement would be required.

DESIGN OF REPAIRS TO SLOPE

Northern Half of the Slope

Northern slopes were found to require the most immediate repairs. A number of alternatives were examined to remediate this area including laying back the overall slope, constructing a retaining wall, recontouring the slope with multiple benches, and recontouring the slope with one straight cut face.

The solution that was selected as shown in Figure 5 involves excavating a minimum of 15 feet (horizontal) to remove the weathered rock surface, at a slope of 0.35H:1V from the top of the Cow Run sandstone to the top of the Buffalo sandstone, constructing a 30 foot (minimum) wide bench at the top of the Buffalo sandstone, constructing a slope again at 0.35H:1V to just below roadway level and constructing a 50-foot wide rockfall catch ditch at roadway level with a bottom cushion layer of crushed stone to absorb impacts. The 0.35H:1V faces are being protected with shotcrete and rockbolts to prevent further differential erosion. Thicker shotcrete and more closely spaced dowels are being installed on the shale faces while the dowel spacing is being increased and the shotcrete occasionally eliminated on the sandstone faces, depending on field assessment of rock conditions once exposed.

This is a compromise solution which does not include repairs to the overhangs in the Morgantown sandstone at the top of the slope because of land acquisition problems. While there still may be significant rockfalls from the Morgantown sandstone in this area, the risk of these blocks reaching the road is greatly reduced by the bench at the top of the Buffalo sandstone and the rockfall catch ditch at roadway level. A concrete barrier constructed between the catch ditch and roadway shoulder will further prevent rocks from reaching the road.

Southern Slopes

The natural slopes are less steep overall in the southern half of the project due to the presence of benches in the Crown Run and Buffalo Sandstones. However, the toe of the slopes will be excavated vertically for the new intersection at the bridge and the slope will be stabilized by the construction of Wall No. 5. This excavation effectively removes the run-out/arresting areas afforded to past rockfalls by the lower slopes and benches. The two areas of concern are the loose blocks which dilate and topple from the Cow Run sandstone at midslope and the large overhangs in the Morgantown sandstone at the crest of the slope.

After examining many options for stabilizing these areas the selected solution is to erect a high capacity rockfall catch fence along the top of Wall No. 5 as shown in Figure 6. As with the northern slope, this solution addresses retention rather than source removal because of land acquisition problems at the top of the slope for the Morgantown bluffs, and access difficulties to the central section of the slope where stability problems created by the construction of an access road would be as great as the existing problems of leaving the blocks in place.

CONSTRUCTION OF REPAIRS TO THE SLOPES

Construction of the overall project began in 1990 and is due to be complete in 1993. The repairs to the northern half of the slope started in March 1992 and should take approximately one year. The rockfall catch fence in the southern half of the project will be erected in late 1993.

The vast majority of the work for the repairs is concentrated at the northern half of the project. Because the slopes are steep in this area, the only practical method of construction was to cut a pioneer road to the top of the excavations for the slope recontouring, and to excavate the slope from the top down, casting the excavated material over face of the slope.

To maintain the flow of traffic on the highway, it was necessary to divert the traffic onto the outer two lanes, excavate a small amount at the toe of the slope and construct a temporary high capacity rockfall fence beside the construction haul road as shown in Figure 7 and Photograph 2. Even though extensive rock bounce analyses were run on this configuration, some small rocks have bounced over the top of the temporary high capacity fence as large blocks shattered on impact with the fence or catch area. It was necessary to erect a second lower energy (chain link) fence adjacent to SR 7 to prevent these small rocks from landing on the roadway. It has become evident that rock bounce analyses cannot model the effects of rock shatter; the energy transfer during the impact of a large rock to its broken pieces can be significant.

The shotcrete and dowels for slope protection are being applied from construction benches as the construction progresses. A dry mix shotcrete method is being used and the dowels are untensioned epoxy coated #8 rebar (Grade 60) bent to an "L" shape at the face to hook over steel weld mesh, thus eliminating the need for threaded ends, face plates and nuts at the rock bolt heads. ODOT had considered specifying steel fiber reinforcement for shotcrete, but finally elected to use wire mesh. The decision was based primarily on lack of documented local, long term performance of fiber reinforced shotcrete on steep rock slopes.

While it would appear that steep, narrow cuts and rock dowel-shotcrete systems are costly, lack of easily acquired right-of-way and waste areas for large excavations will continue to promote this type of remediation on future remediation projects.

Acknowledgements

The authors would like to thank Ohio DOT for permission to publish this paper. They would also like to thank the staff at Ohio DOT, Golder Associates, and URS Consultants for their contribution to the engineering at the site.

References

1. Humphries, R.W.; Elliott, G.M.; Caferelli, G.; and Geiger, E.; "Analysis and Design of Tieback Wall No. 5 in Steubenville, Ohio" Proceedings 41st Highway Geology Symposium, Albuquerque, NM, August, 1990.

2. Elliott, G.M.; and Rippere, K.H., "Performance Analysis in Rockfall Simulation", Proceedings 41st Highway Geology Symposium, Albuquerque, NM, August, 1990.

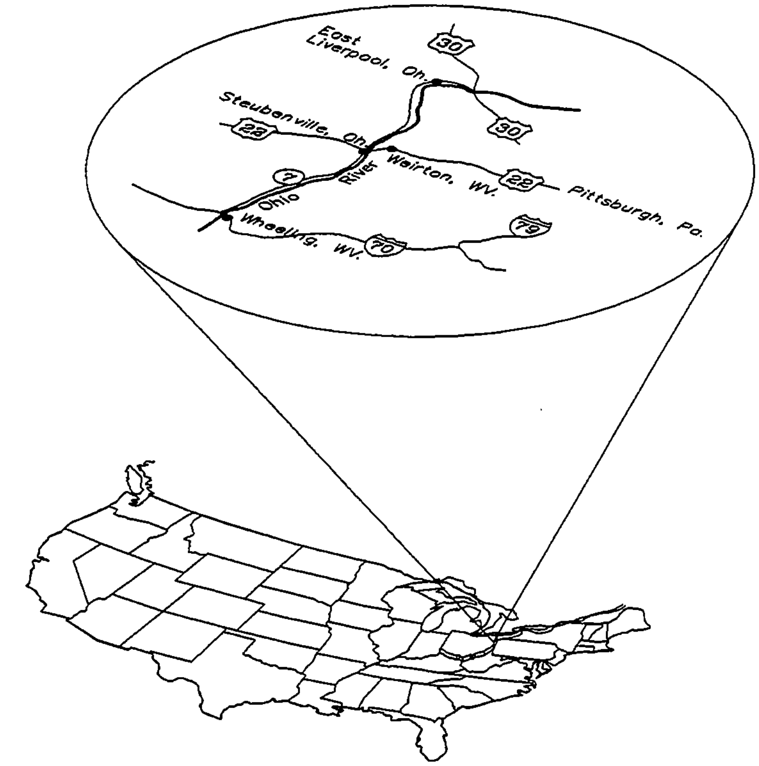


FIG. 1 - SITE LOCATION MAP

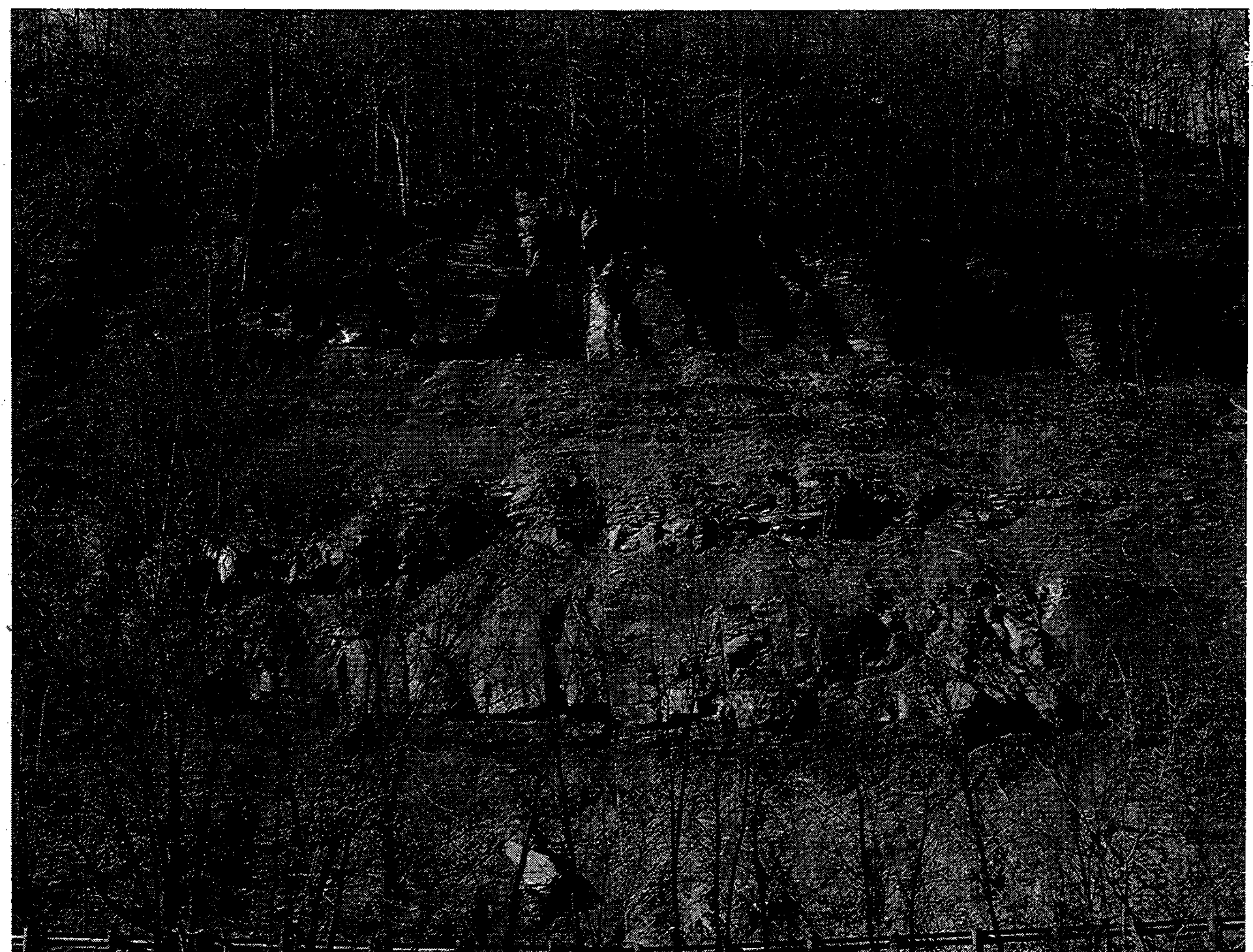


PHOTO 1

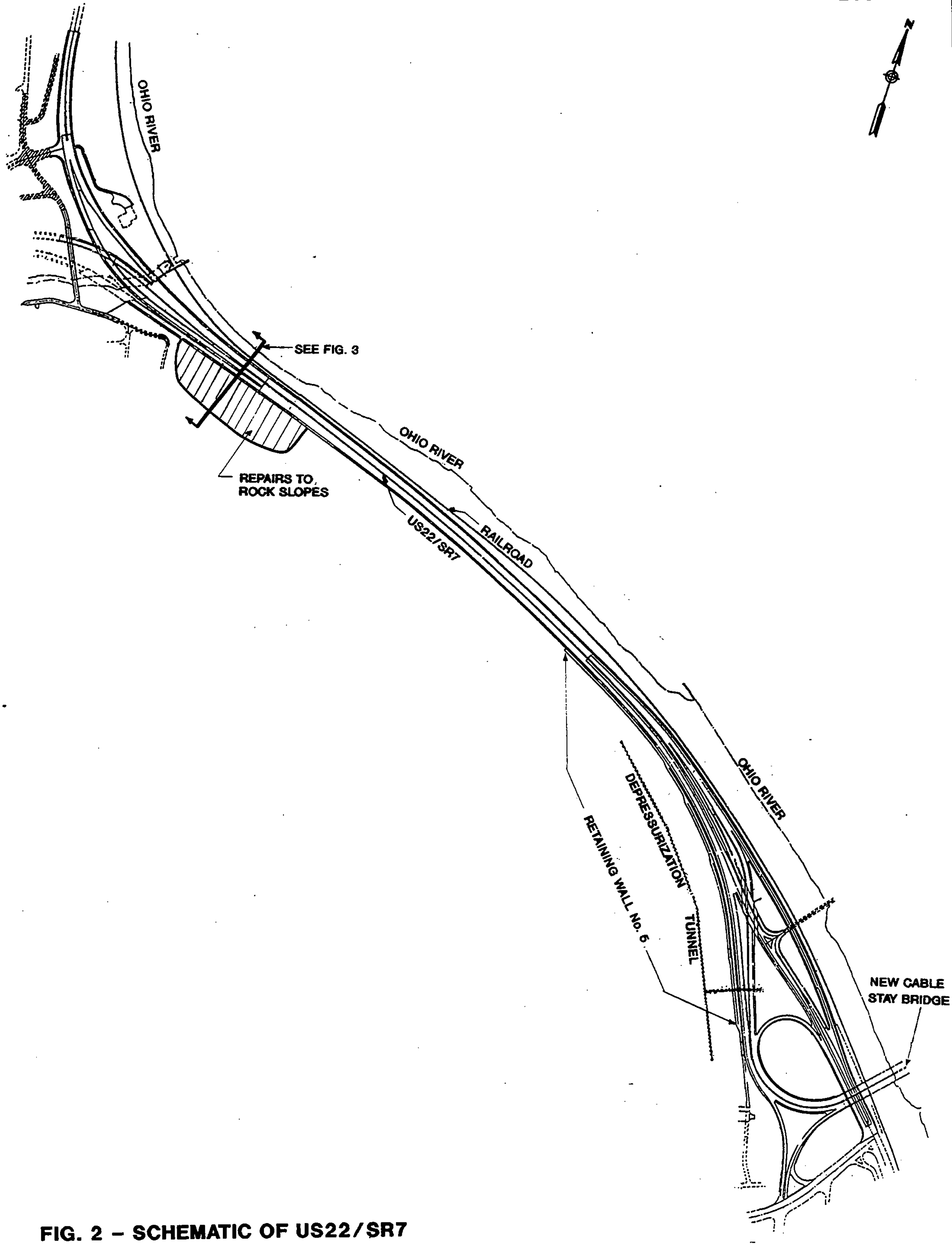


FIG. 2 – SCHEMATIC OF US22/SR7 INTERCHANGE PROJECT

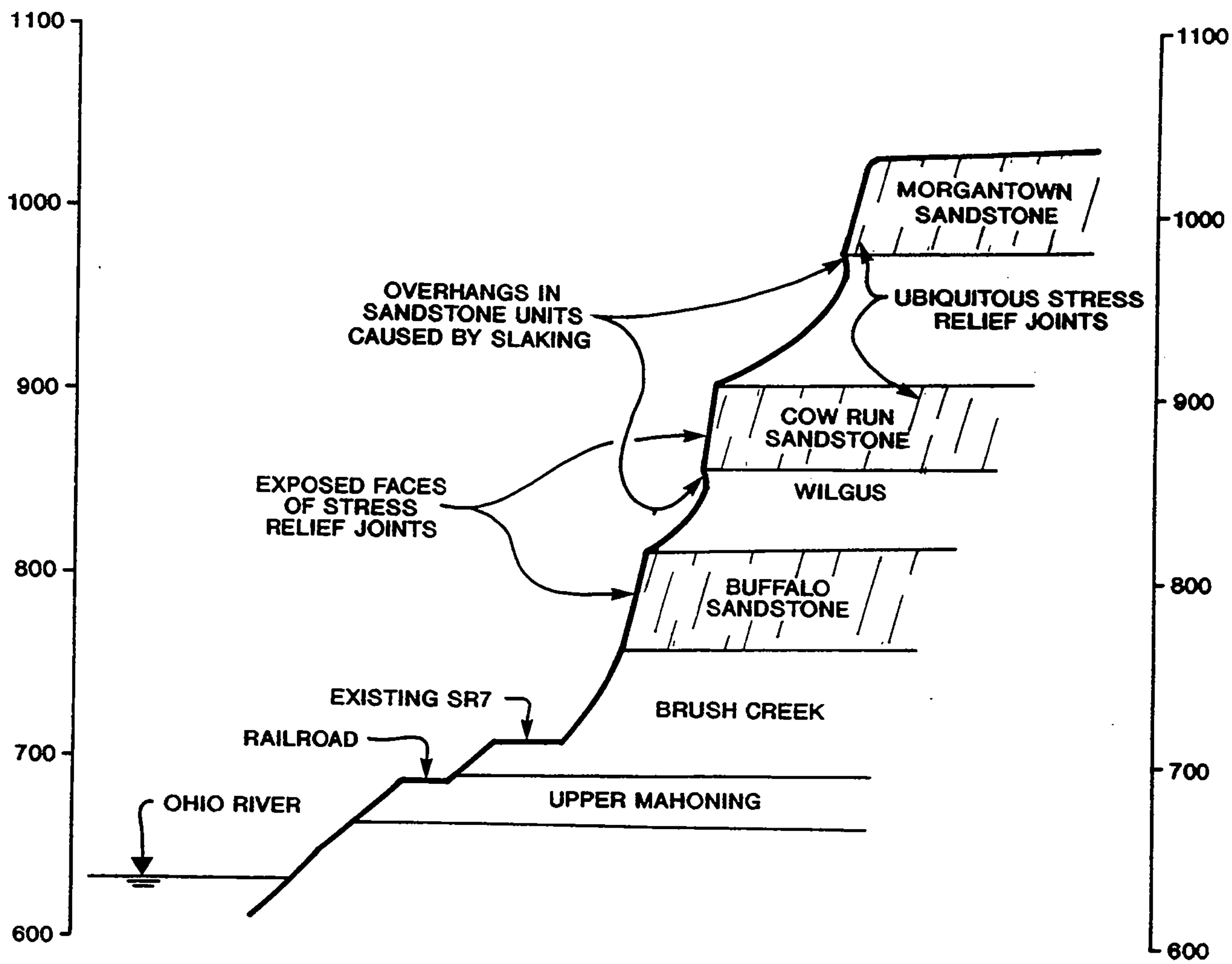


FIG. 3 – PRECONSTRUCTION CROSS SECTION OF SLOPES AT STUEBENVILLE, OH.

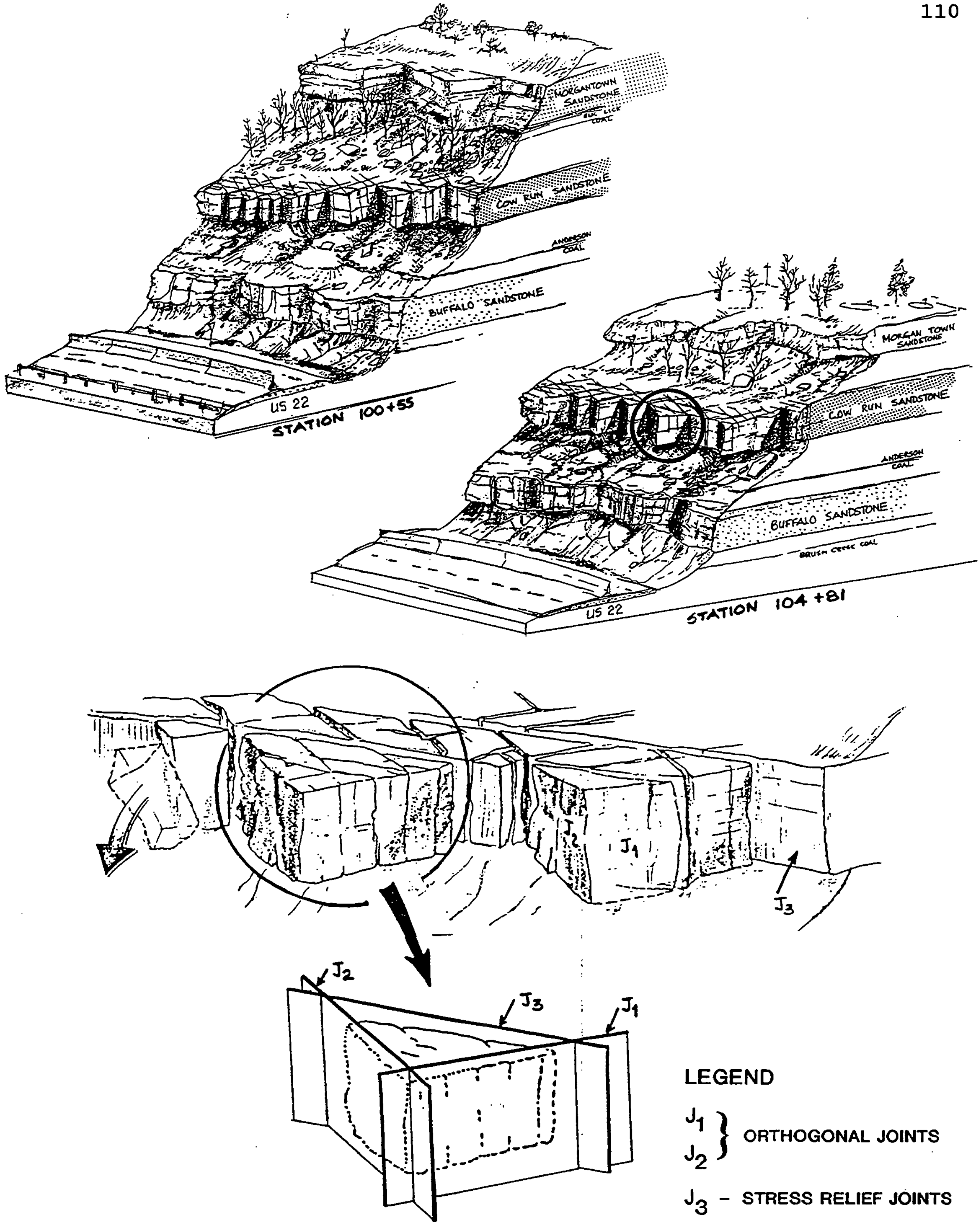


FIG. 4 – SCHEMATIC OF ROCK STRUCTURES AT PROJECT SITE.

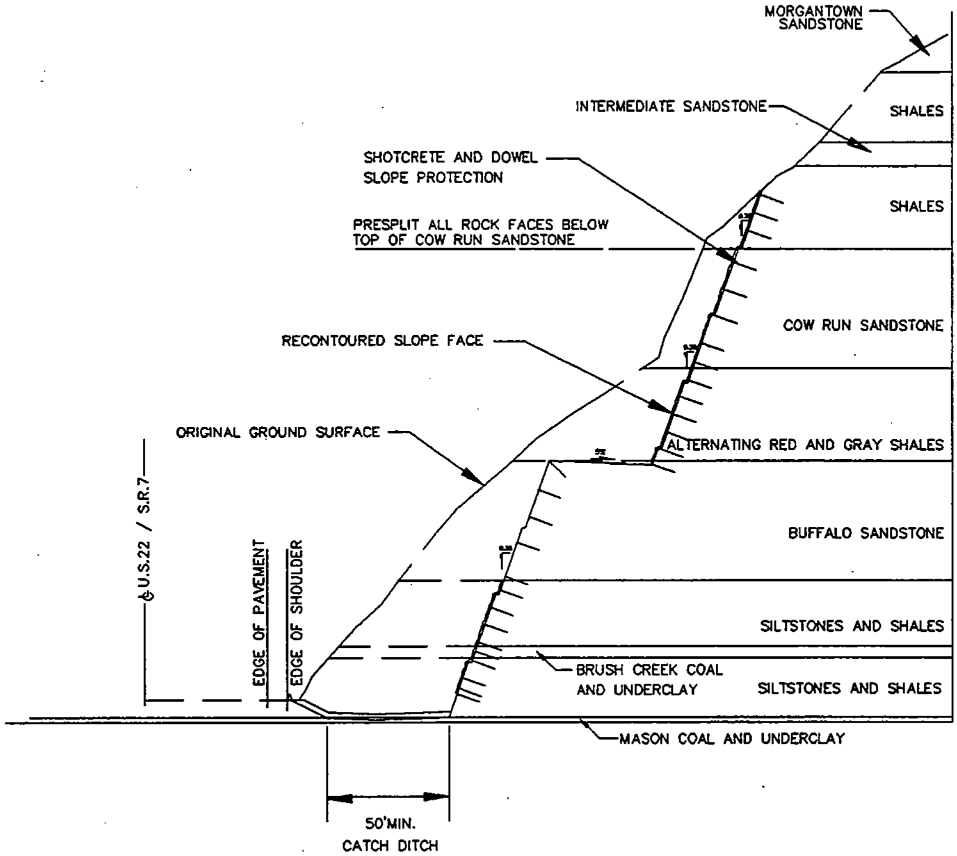


FIG. 5 - CROSS SECTION OF REPAIRS AT NORTH END OF PROJECT

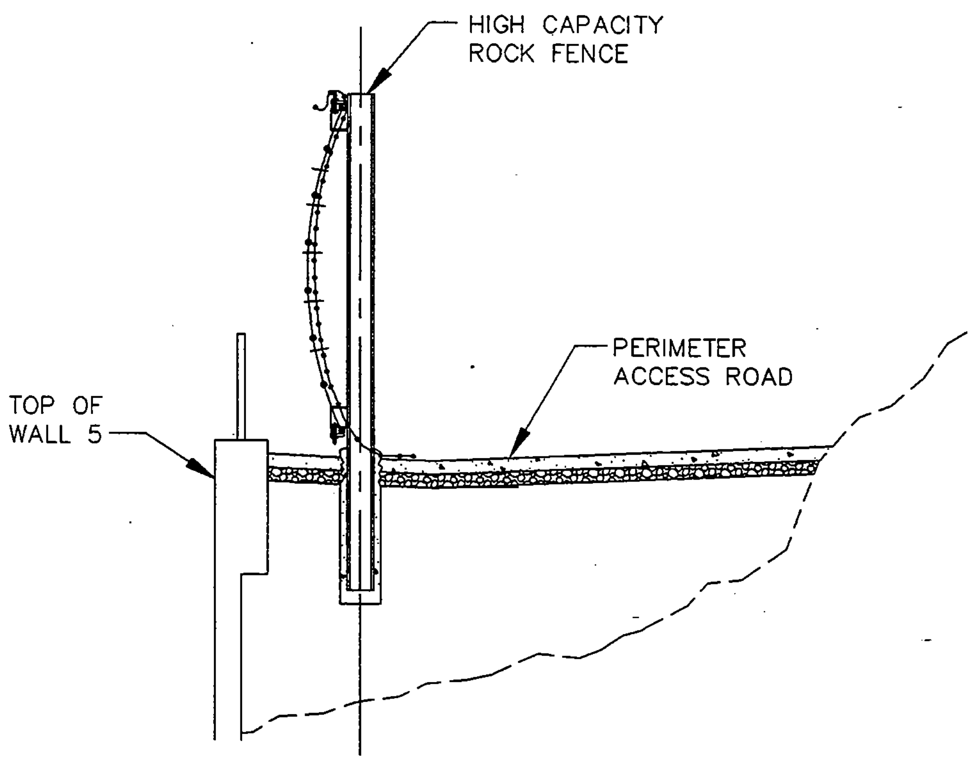


FIG. 6 - POSITION OF HIGH CAPACITY ROCK FENCE ABOVE WALL No. 5

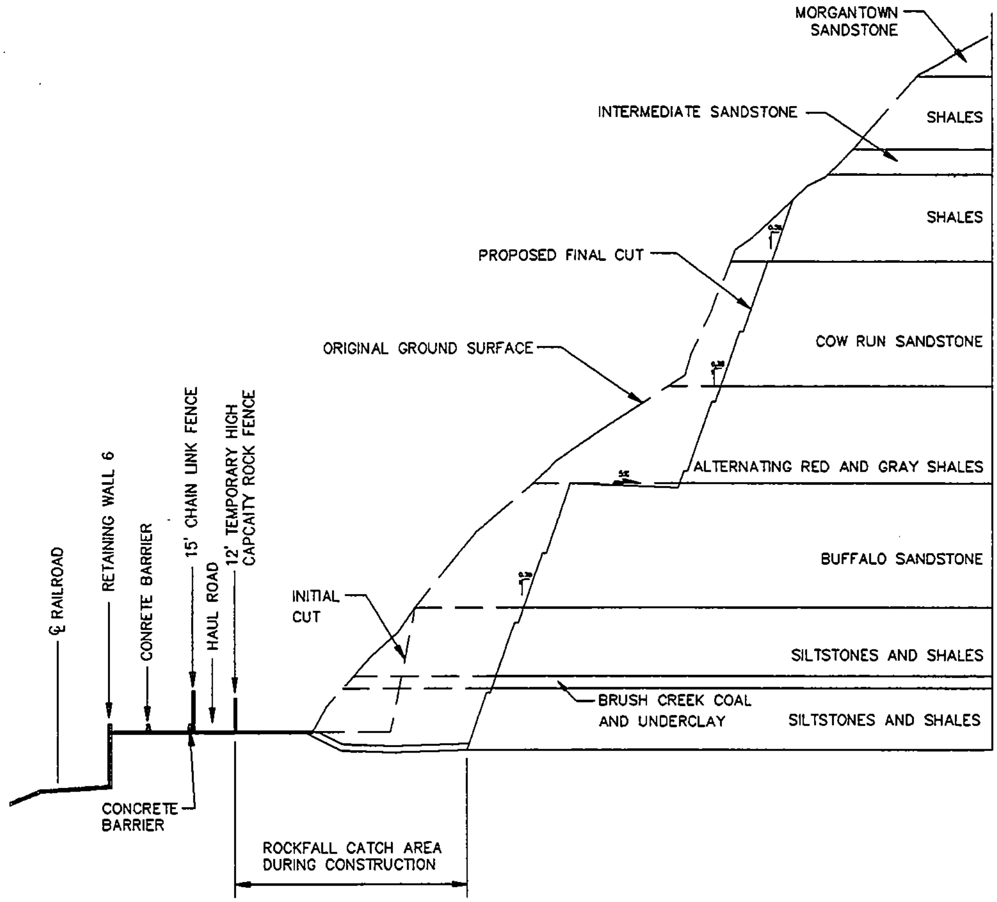


FIG. 7 - SLOPE CONFIGURATION DURING EXCAVATION

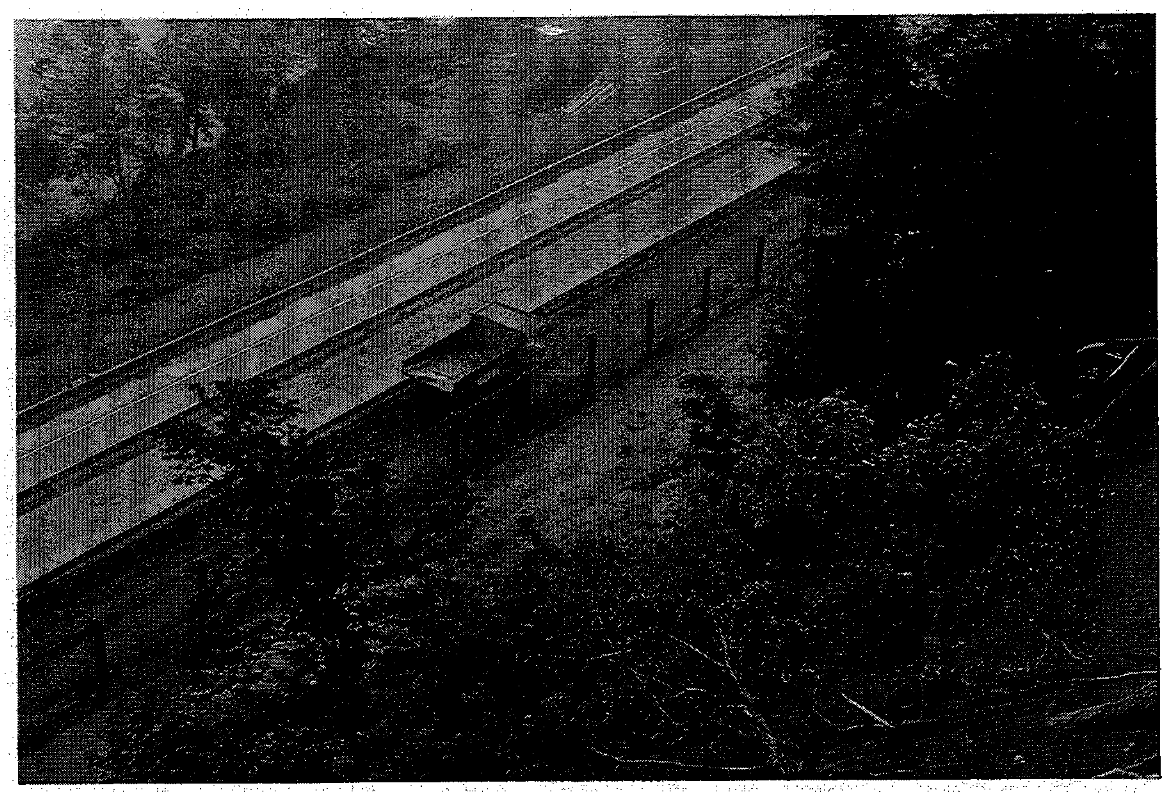


PHOTO 2

Cannon Creek Embankment Instrumentation

Sam I. Thornton - University of Arkansas
Michael S. McGuire - Wal-Mart Engineering
Boon K. Thian - AHTD

ABSTRACT

Cannon Creek is the site of a 76 feet high by 800 hundred feet long geogrid reinforced embankment instrumented with Piezometers, Extensometers, Inclinometers, Strain gages, Moisture-Temperature Indicators, Psychrometers, Soil Matrix Potential gages, and Tensiometers. Built in 1988, the embankment is located about 20 miles southeast of Fayetteville.

Piezometers, Extensometers, Inclinometers, and Strain gages all provided useful information about the performance of the embankment. The useful life for the Extensometers was three years. Half of the Strain gages were still in use after 3 years.

INTRODUCTION

The Arkansas Highway and Transportation Department (AHTD) has constructed a 76 feet high by 800 feet long geogrid reinforced embankment on state highway 16 at Cannon Creek (Figure 1). The embankment, located about 20 miles southeast of Fayetteville, Arkansas, (Figure 2) is the tallest geogrid reinforced embankment in the USA built by a Department of Transportation.

A report that the embankment was stable after 2 years of service was presented to the Highway Geology Symposium in 1990 (Thornton and McGuire). Details of the construction are contained in a paper by Hayden, Schmirtmann, Qedan and McGuire (1991).

INSTRUMENTATION

Built in 1988 as a demonstration project, the Cannon Creek embankment was instrumented at a cost of $30,000 to monitor its performance and stability (Qedan, 1988). Included in the instrumentation were three Piezometers, three Extensometers, three Inclinometers, seventy-six Strain gages, seven Psychrometers, five Soil Matrix Potential gages, eight Moisture-Temperature Indicators and two Tensiometers.

Results

Piezometers, Extensometers, Inclinometers, and Strain gages have provided useful information about the performance of the embankment. McGuire (1990) identified three potential failure surfaces based on the stresses, deformation and water pressures measured by the instruments. All of the potential failure surfaces had factors of safety over 1.3. In addition, the maximum measured reinforcement strain was only 1.3 percent, well

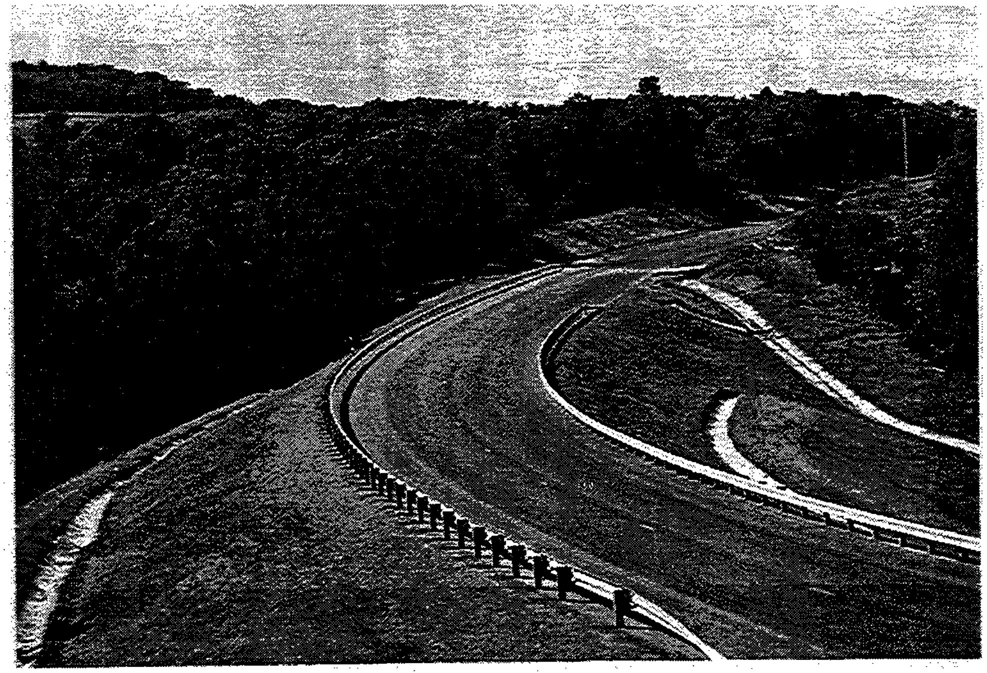


Figure 1

The Cannon Creek Embankment

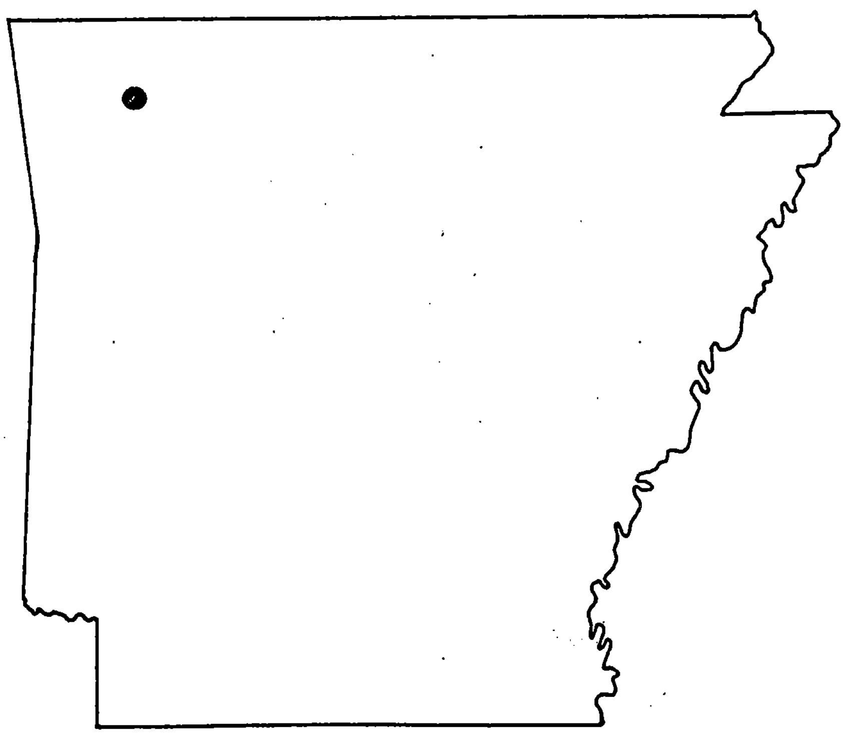


Figure 2

Cannon Creek Embankment Location

within the predicted maximum of 10.0 percent.

For a variety of reasons, however, Psychrometers, Soil Matrix Potential gages, Moisture-Temperature indicators, and Tensiometers, have provided little useful information about the performance of the embankment.

The Psychrometers and Soil Matrix Potential gages proved difficult to read. Numbers from the readout display jumped over a wide range on these two gages.

Moisture-Temperature Indicator and Tensiometer data was used only to confirm that the soil mass was not changing substantially with time. Moisture-Temperature Indicator data is thought to be valid, showing a moist condition in the soil mass. Tensiometer data indicates the soil had no suction (zero readings) until June, 1992.

Expected Life

The useful life of the gages varied widely. Table I contains the history of the failure rate of the gages.

Table I
Useful Life of the Gages

Number of Gages in Service

Gage	Installed	1988	1989	1990	1991	1992
Piezometer	3	2	2	2	2	2
Extensometer	3	3	3	3	0	
Inclinometer	3	3	3	3	3	3
Strain Gage	76	68	58	53	37	
Moisture-Temperature	8	8	8	6	6	6
Psychrometer	7	7	7	7	7	7
Soil Matrix Potential	5	4	3	0		
Tensiometer	2	2	1	1	1	1

CONCLUSIONS

1. Instrumentation has provided useful information in the performance monitoring of the Cannon Creek geogrid reinforced embankment.

2. Piezometers, Extensometers, Inclinometers and Strain Gages were useful in monitoring the Cannon Creek embankment.

REFERENCES

Hayden,R.F., G.R. Schmirtmann, B.A. Qedan and M.S. McGuire, 1991, "High Clay Embankment Over Cannon Creek, Constructed with Geogrid Reinforcement", Geosynthetics 91 Conference, Atlanta GA

McGuire, Michael S., 1990, "Reinforced Earth Embankment," Thesis at University of Arkansas, Fayetteville.

Qedan, Bashar A., 1988, "Construction Report on the Cannon Creek Embankment," Arkansas Highway and Transportation Department, Research Station, Box 2261, Little Rock, AR 72203.

Thornton, Sam I. and Michael S. McGuire, 1990, "Geogrid-Expansive Clay Embankment," Proceedings of 41st Annual Highway Geology Symposium, Albuquerque, NM.

DESIGN, CONSTRUCTION AND MONITORING OF A 76' HIGH GEOGRID REINFORCED EARTH EMBANKMENT

by

David W. Lumbert

and

Boon K. Thian

Arkansas State Highway and Transportation Department

Planning and Research Division

A paper presented to the

43rd HIGHWAY GEOLOGY SYMPOSIUM

FAYETTEVILLE, ARKANSAS

AUGUST 19-21, 1992

Design, Construction and Monitoring of a 76' High Geogrid Reinforced Earth Embankment

ABSTRACT:

This paper will present an overview of the design process and focus on the results of the monitoring to date. The purpose of the said reinforcement was to improve the embankment stability thus obtaining a satisfactory factor of safety. Originally, the overall stability of the 76' high geogrid reinforced earth embankment was minimized due to physical characteristics of the high plasticity indices of the insitu clays used as fill material. The average plasticity limit of the fill material used was 26. Geogrid reinforcement consisted of three different types of primary reinforcement and one type of secondary reinforcement.

While under construction, monitoring instruments were installed at different locations throughout the downstream side of the embankment. Instrumentation consisted of multi-point extensometers, inclinometers, strain gages, settlements stakes, tensiometers, pneumatic piezometers, soil matric potential sensors, and moisture-temperature indicators. Instrumentation with resolved monitoring of the embankment has provided useful information on the effective of the reinforcement in terms of embankment stability, cost and, long term maintenance projections.

The design and stability analysis of the embankment was implemented using a factor of safety of 1.3, soil phi angle of 20 degrees and soil cohesion of 50 psf.

PROJECT DESCRIPTION AND BACKGROUND

Arkansas State Highway 16 is a two lane secondary road which links from Brashears to Fayetteville over the Boston Mountains (Figure 1). In 1985, a realignment project involved the replacement of a hairpin curve and a one-lane bridge over Cannon Creek with a straightened roadway section, an embankment with a maximum side slope height of 76 feet, and a four barrel concrete box culvert.

In 1986, slope failures occurred during the early stages of construction of the embankment which eventually led to the cancellation of the project. In order to obtain a greater factor of safety against slope failures, several alternatives were considered . In 1987, construction resumed under a new contract which included the use of geogrid material as embankment reinforcement. Furthermore, the new contract included instrumentation and monitoring plans to evaluate the performance of the reinforced embankment.

SOIL PROPERTIES

The material for embankment fill was a highly plastic clay (AASHTO type A-7-6). Attenberg limits test performed on 9 samples recovered from borings in the borrow areas resulted with the high average plasticity index (PI=35). The clay was believed to be expansive. Grain size analysis indicated that all samples had at least 70% by weight finer than the #200 sieve. The result of the undrained unconsolidated (UU) triaxial tests on compacted samples of the clay indicated an undrained shear strength of about 900 psf.

DESIGN

The design was performed by the supplier of the geogrid using three primary reinforcement grades, designated types 1, 2, and 3, assumed to have allowable 120-year design strengths of 1000, 2000, and 3000 lb/ft, respectively, in one material direction. It was assumed in design that the reinforcement was placed in continuous horizontal layers. Specifically, total geogrid strain at the allowable design load was limited to 10% over 120 years, in order to limit long-term embankment deformations.

The design specified that continuous horizontal layers of intermediate reinforcement, consisting of lightweight geogrid extending 4.5 ft. into the slope, be placed on 1 ft. vertical intervals over the entire slope face. Intermediate reinforcement is believed to be a necessity when dealing with expansive clay soils.

CONSTRUCTION

The fill was spread in nominal 8 in. lifts. Sheep-foot rollers were used to compact the clay fill to the specified minimum 95% relative compaction according to the AASHTO T-99 standards. Specified moisture contents ranged from optimum to 4% dry of optimum for the lower 30 ft. of embankment and from +2% to -2% of optimum in the upper embankment section.

INSTRUMENTATION AND MONITORING

Due to the limited experience with geogrid reinforcement of high, plastic clay fills, an extensive instrumentation array was installed in the downstream face of the embankment structure during construction.

The instrument was designed to measure the performance of the geogrid-reinforced embankment with respect to horizontal and vertical movement of the soil, horizontal strain in the geogrid, and soil pore pressure and moisture content. Monitoring will continue for five years after completion of construction. The installed instruments are listed below under headings of the response to be measured:

Horizontal soil movement:-
Multipoint extensometer - 3
Inclinometers - 3

Vertical soil movement:-
Settlement stakes

Horizontal geogrid strain:-
Electrical resistance strain gages - 67

Soil pore pressure:-
Tensiometers - 2
Pneumatic piezometers - 3
Soil matrix potential sensors - 5

Soil moisture content:-
Moisture-temperature indicators - 8

MEASUREMENTS OF HORIZONTAL MOVEMENTS AND STRAINS

Extensometers:

Three horizontal multipoint rod extensometers were installed in the downstream face of the embankment in the locations indicated in Figures 2, 3 and 4. The interval between anchors was 5 ft. near the slope face and 10 ft. for the remaining length. Relative movement between each anchor and the slope face was measured to 0.001 inch at various times during construction with the first readings taken when the embankment crest was about 0 to 10 ft. above each extensometer elevation.

The total horizontal soil strains detected in each anchor interval during approximately 26 months after installation are presented in Figures 5, 6, and 7. The primary indication from the extensometer data is that horizontal soil extension strain is greatest in the outer 10 ft of the embankment. Specifically, average measured strain in the outer 10 ft. is approximately 4.3%, while the average strain in the zone 10 to 50 ft from the face is 1.7%.

The significant higher horizontal strain near the slope face is consistent with the inclinometer data which shows dramatic increases in measured horizontal soil displacement within the 5 to 10 ft depth from the slope face. Large movements in the near-face soils also seem consistent with the use of expansive clay fill given moisture access at the slope.

Inclinometers:

Three slope inclinometers were installed in the downstream face of the embankment in the location shown approximately in Figure 8, 9, and 10. Installation was performed when the embankment fill reached the planned inclinometer elevations.

The total outward horizontal movements detected by each inclinometer during approximately 43 months after installation are shown in Figures 11, 12, and 13. Points of local maximum values on the graphs, for example at depths of 13 and 25 ft for inclinometer #1, at depths of 20 and 35 ft for inclinometer #2, and at depths of 9 and 21 ft, can be interpreted as locations of zones of relatively high horizontal displacement. These graphs also suggest large horizontal movement within the surface to 15 ft depth from the slope face.

Strain Gages:

Sixty six strain gages were glued to the ribs of the geogrid reinforcement the downstream slope of the embankment. The first set consisted of 38 gages on type 3 geogrids at elevation 1326 ft. and 15 to 70 ft. behind the slope face. The majority were placed on the geogrid tensile load carrying ribs, but four were placed on the geogrid cross bars perpendicular to the load carrying direction. A plan view of a representative geogrid structure showing typical strain gage location is shown on Figure 14. Strain gage set #2 consisted of 28 gages on type 2 geogrids at elevation 1352 ft. and 30 to 50 ft. behind the slope face. All of this set were placed on the geogrid load carrying ribs. Strains occurred during gage installation, geogrid placement, and initial soil placement and compaction cannot be evaluated because the zero gage readings were made after.

A summary of the total measured gage strains during the 33 months after the zero gage readings is tabularized in Table 1 for both gage sets. This summary includes only those gages which were working over the entire span time span. The strains measured with small gages glued on the geogrid are not generally equal to the overall geogrid strains. The reason may be that the glue itself is stiff relative to the polymer and may cause a local stiffening of the geogrid.

Furthermore, local strain measured at geogrid ribs may not equal the average strain in the whole geogrid structure.

CONCLUSIONS

1. The Cannon Creek embankment on State Highway 16 is stable to date.
2. From the measured strains, the embankment is performing well within the bounds of its predicted longterm design strain of 10%. The maximum measured strain by extensometer #3 is 6.2% in 33 months.
3. The instrumentation (inclinometer and extensometer) shows a shallow critical failure surface which is probably due to the swelling of the expansive clay materials in the embankment.
4. Instrumentation has provided useful information in the performance monitoring of the Cannon Creek geogrid reinforced embankment.
5. Extensometers, inclinometers, and strain gages were useful in monitoring the Cannon Creek embankment.
6. The rest of the instrumentation has provided little useful information about the performance of the embankment.

REFERENCES

1. Qedan, B. (1990), "Instrume1ntation of a 76 ft High Reinforced Earth Embankment, Cannon Creek Structures and Approaches (phase II): Post Construction Report", Arkansas State Highway and Transportation Dept., Materials and Research Division, Report No. FHWA/AR-90/001, January.

2. McGuire, M.S. (1990), "Reinforced Earth Embankment", MSCE Thesis, University of Arkansas, Fayetteville, Arkansas.

3. Thornton, S.I.; McGuire, M.S.; Thian, B. (1992), "Cannon Creek Instrumentation", Paper submitted for 43rd. Highway Geology Symposium.

4. Vishay Instrumens, Inc., "Correction Equation for Strain Gages on Low Modulus Material", 63 Lincoln Highway , Malvern, PA, 19355.

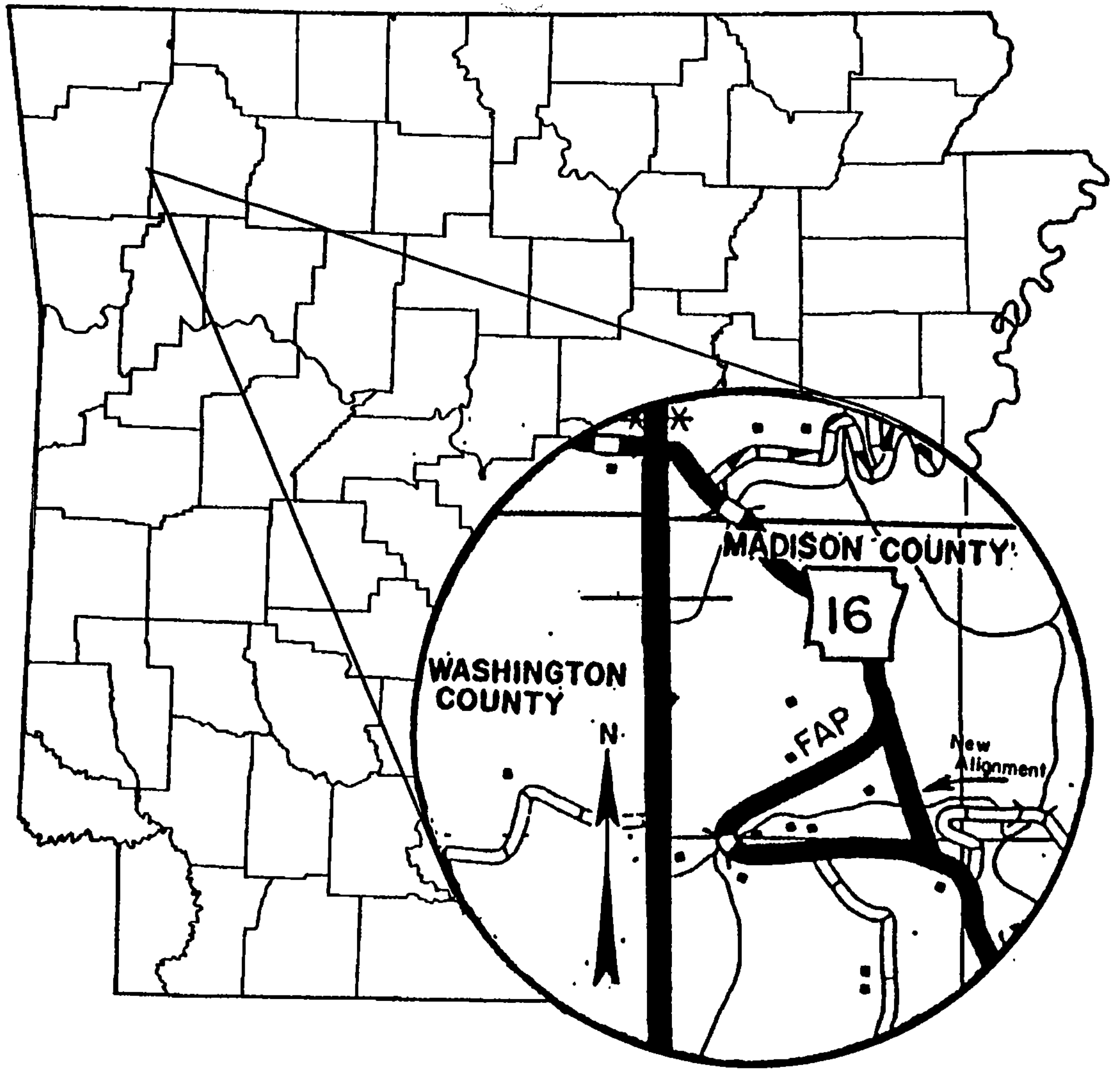


Figure 1 Project location

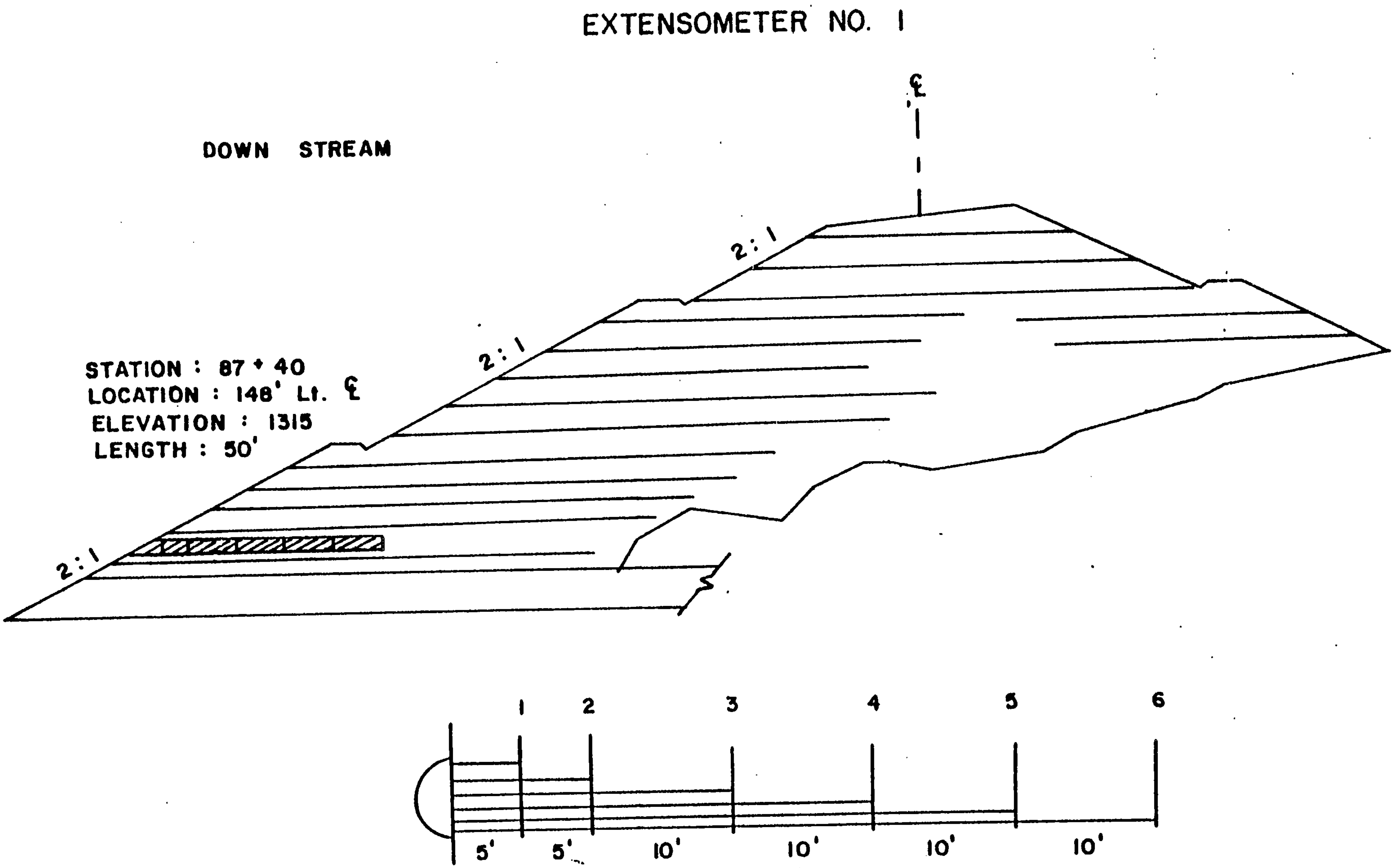

EXTENSOMETER NO. 1
DOWN STREAM
℄
2 : 1
2 : 1
2 : 1
STATION : 87 + 40
LOCATION : 148' Lt. ℄
ELEVATION : 1315
LENGTH : 50'
1
2
3
4
5
6
5'
5'
10'
10'
10'
10'

Figure 2

EXTENSOMETER NO. 2

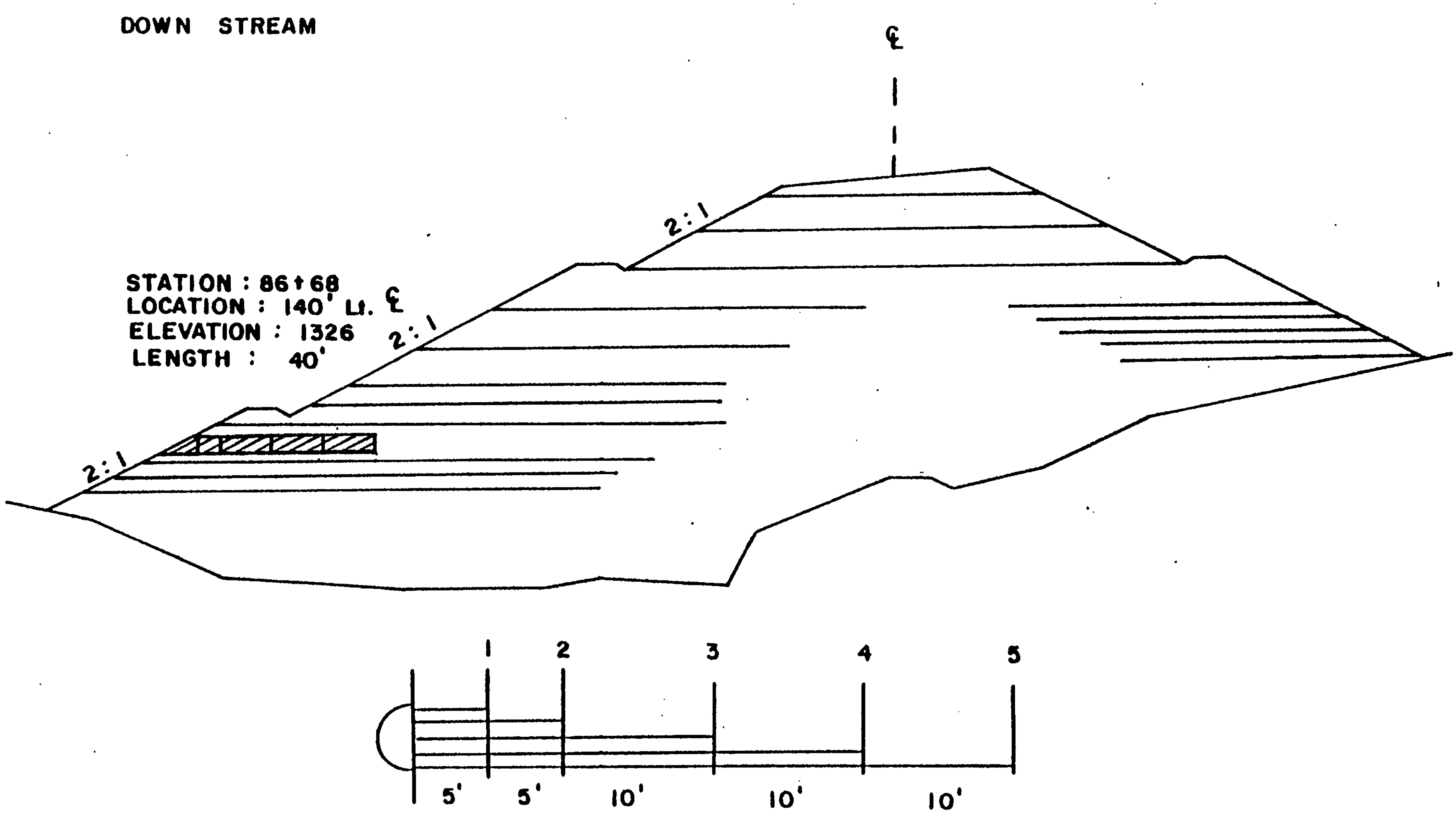
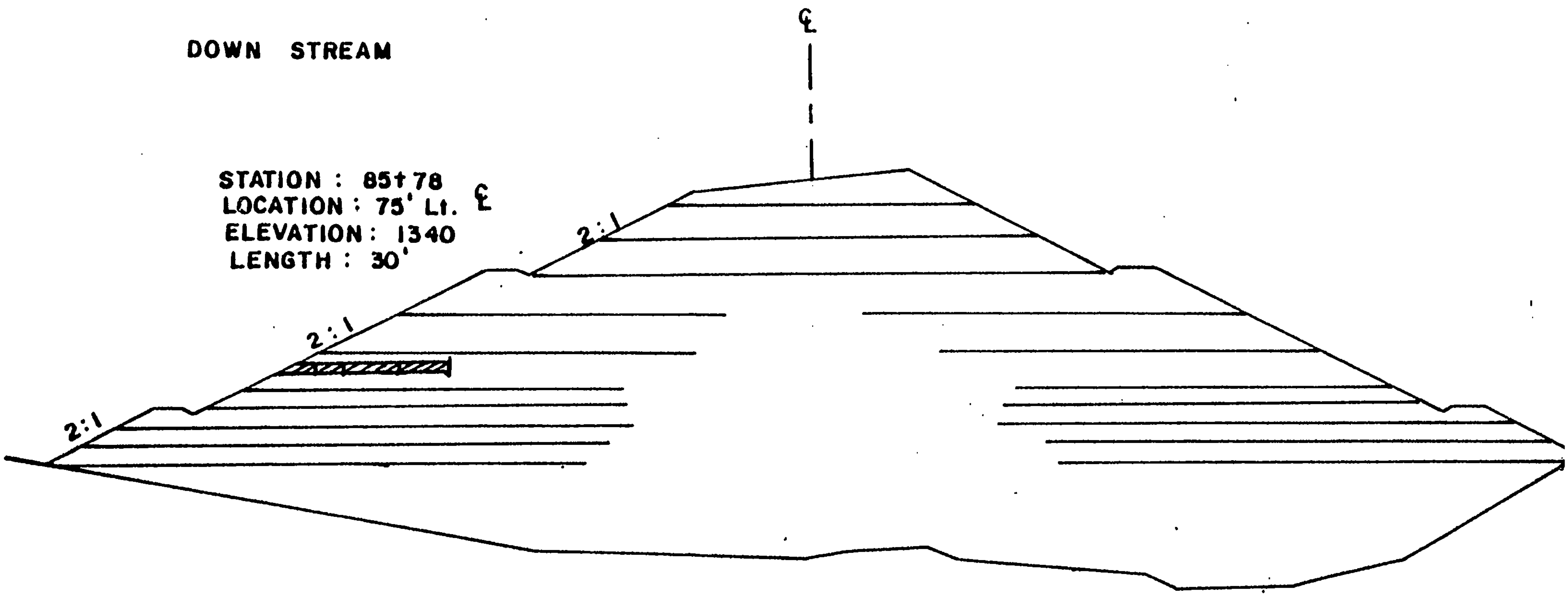


Figure 3

EXTENSOMETER NO. 3



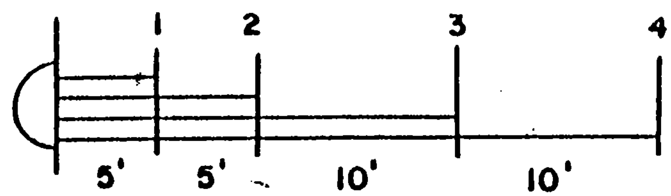


Figure 4

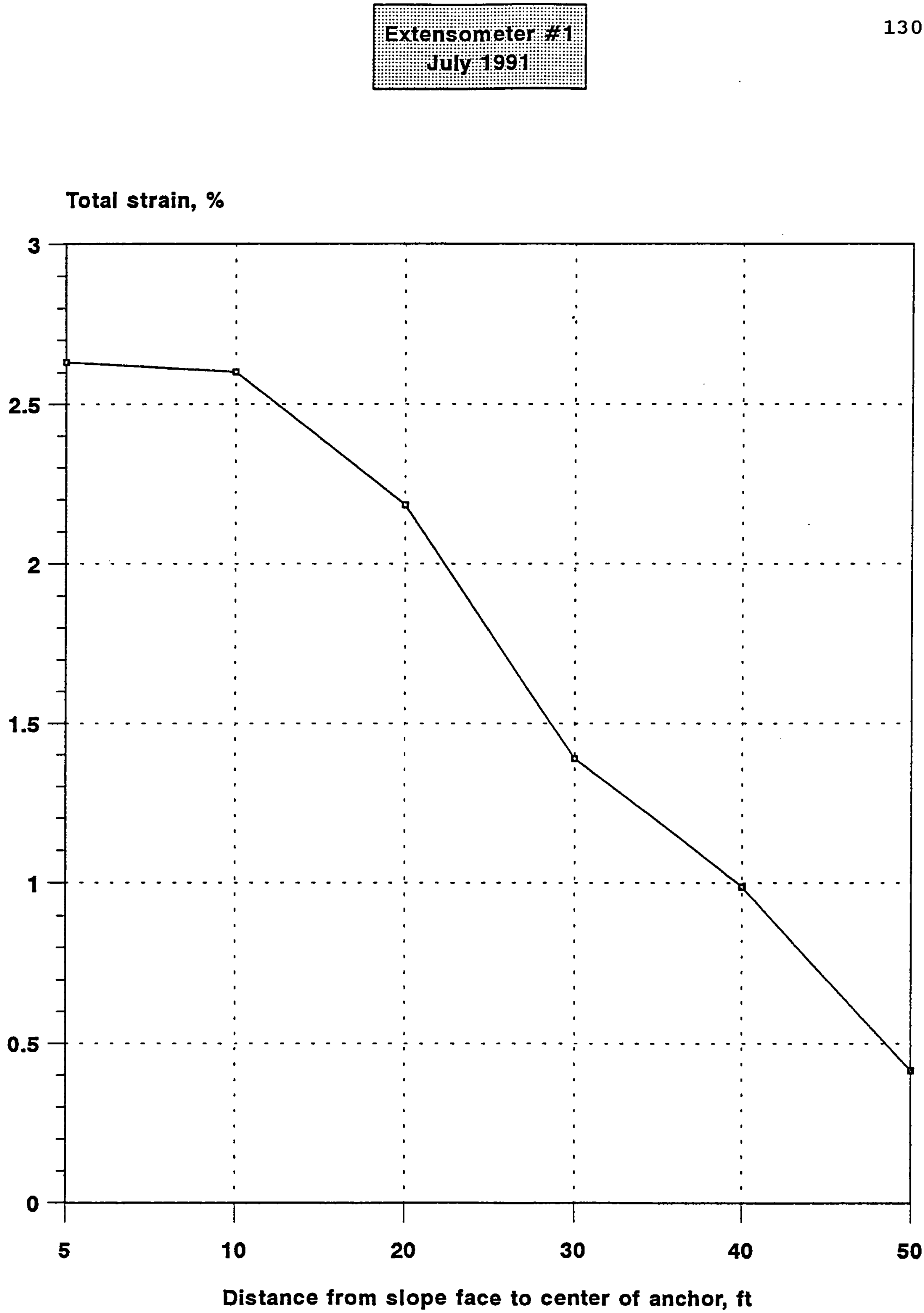

Extensometer #1
July 1991
Total strain, %
3
2.5
2
1.5
1
0.5
0
5
10
20
30
40
50
Distance from slope face to center of anchor, ft

Figure 5

Extensometer #2
October 1990

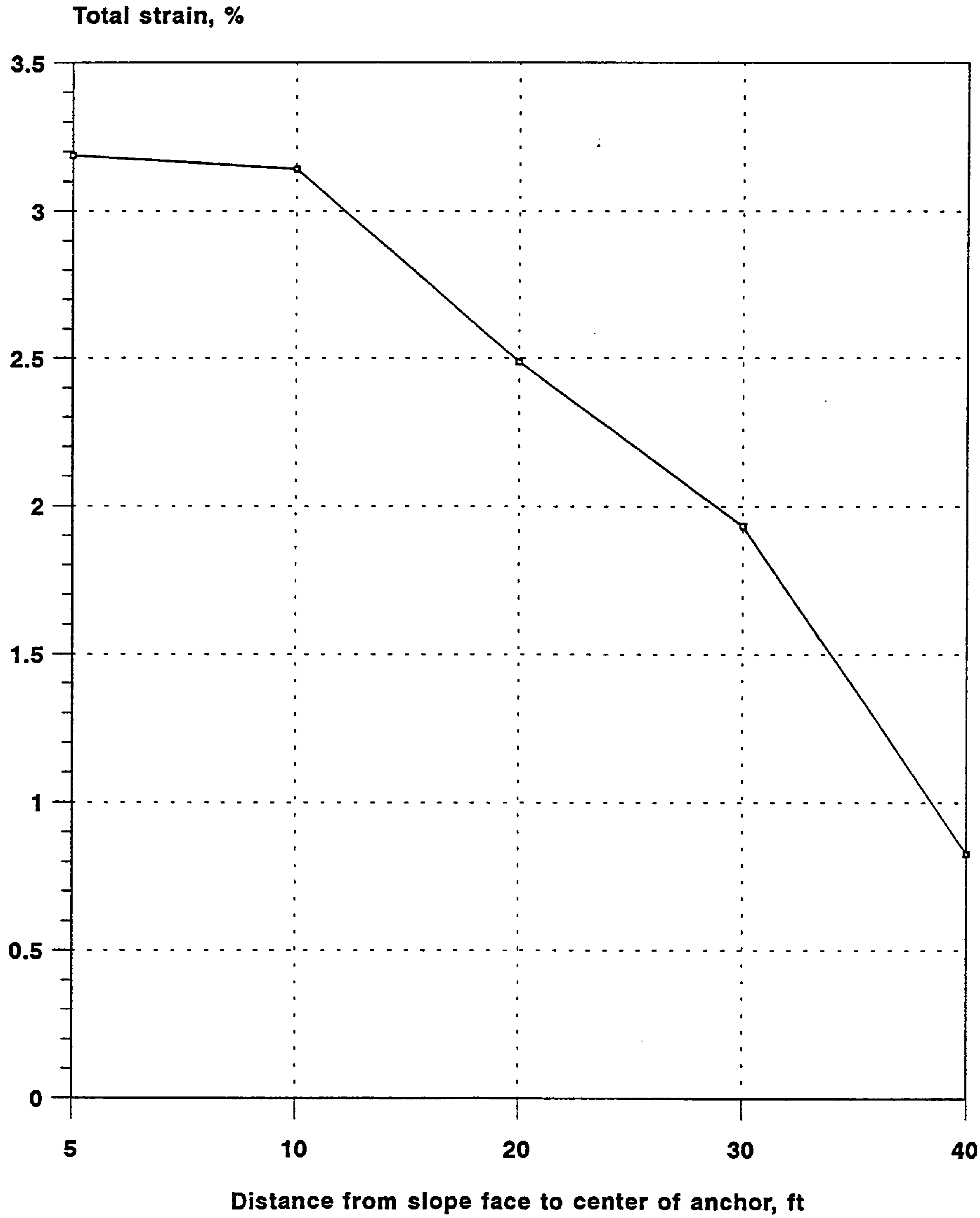


Figure 6

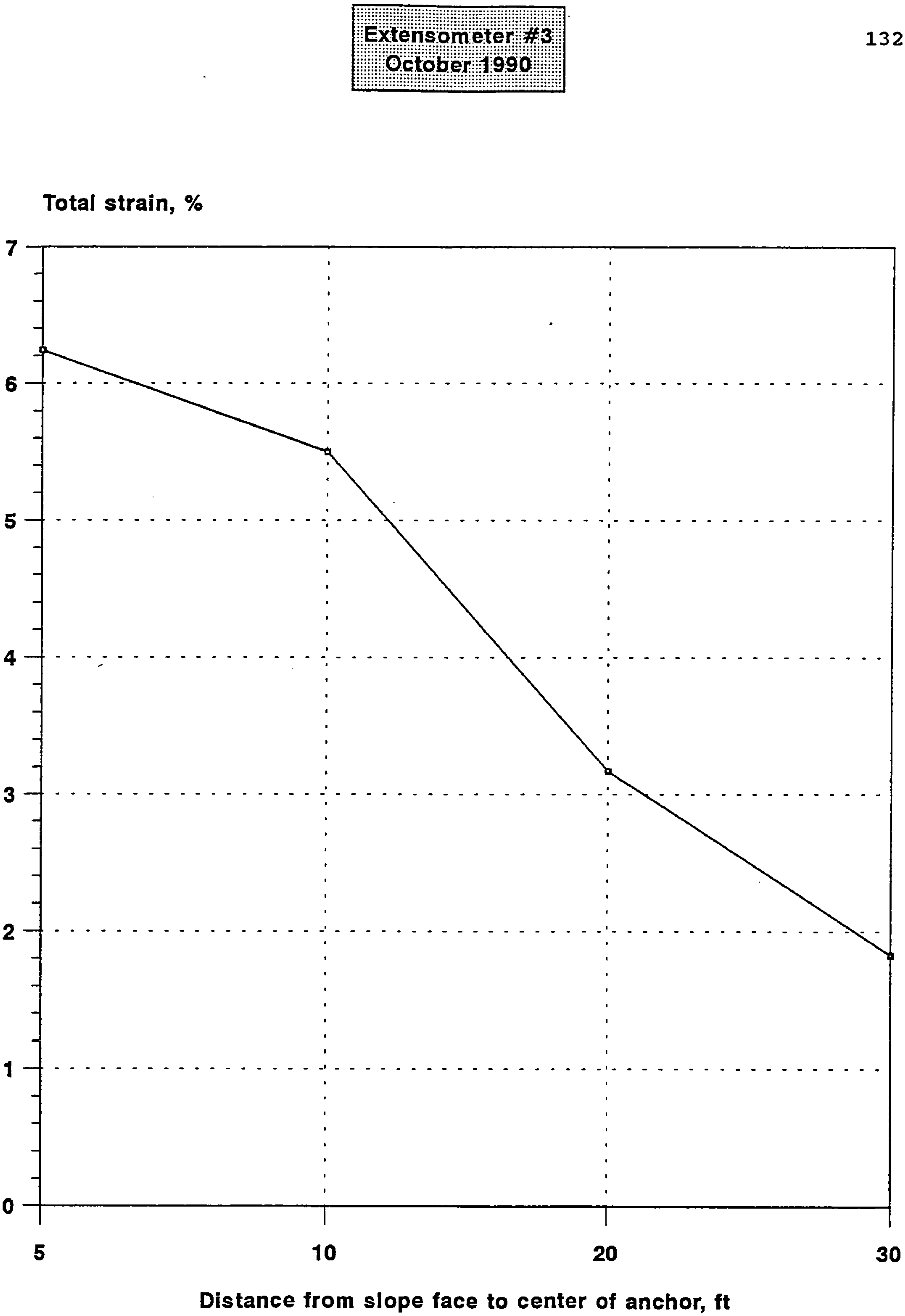

Extensometer #3
October 1990
Total strain, %
7
6
5
4
3
2
1
0
5
10
20
30
Distance from slope face to center of anchor, ft

Figure 7

INCLINOMETER NO. 1

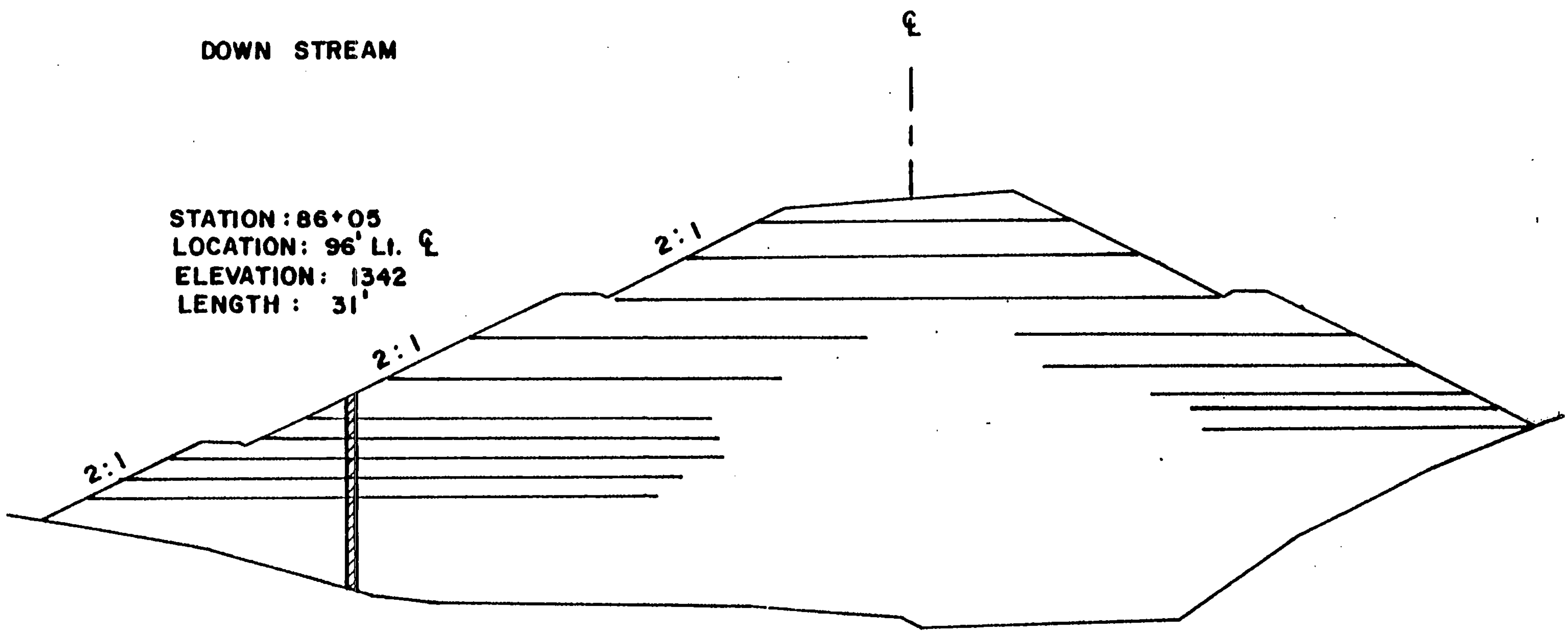


Figure 8

INCLINOMETER NO. 2

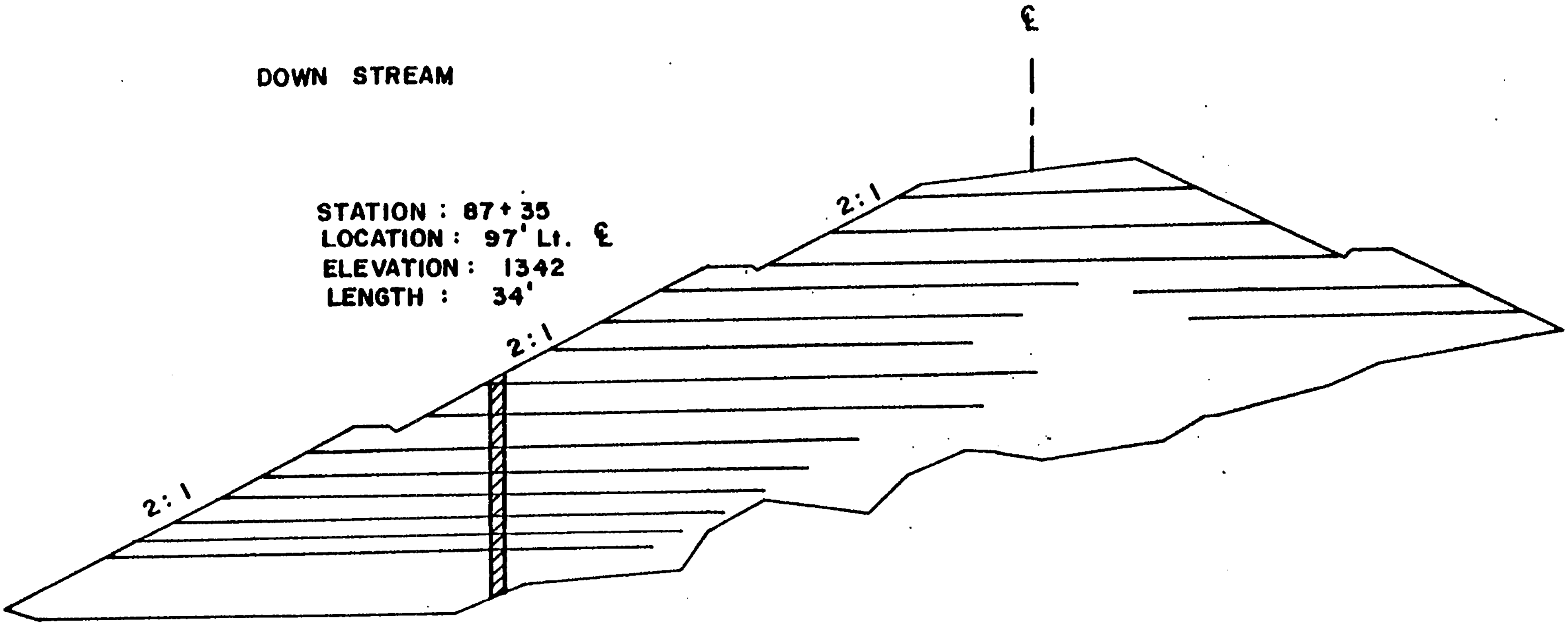


Figure 9

INCLINOMETER NO. 3

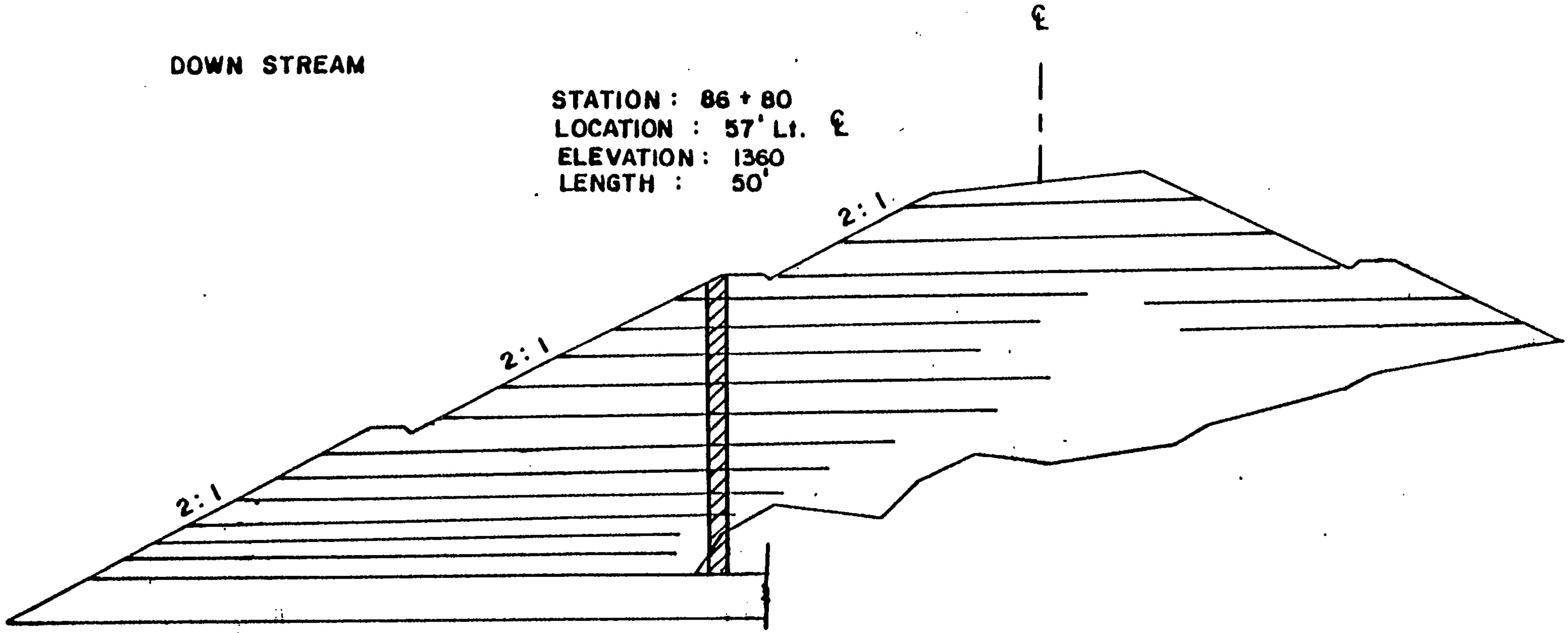


Figure 10

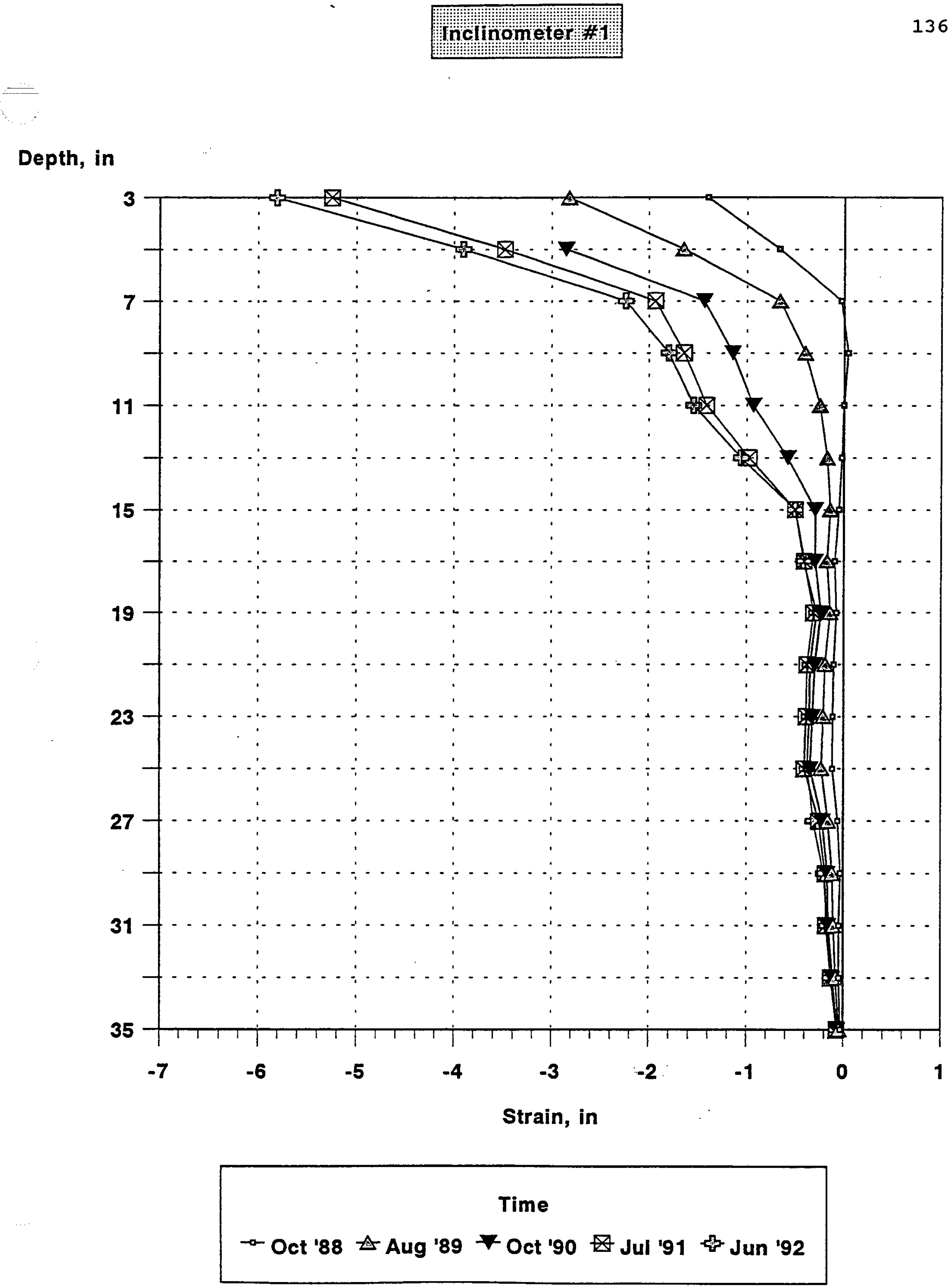

Inclinometer #1
Depth, in
3
7
11
15
19
23
27
31
35
-7
-6
-5
-4
-3
-2
-1
0
1
Strain, in
Time
Oct '88
Aug '89
Oct '90
Jul '91
Jun '92

Figure 11

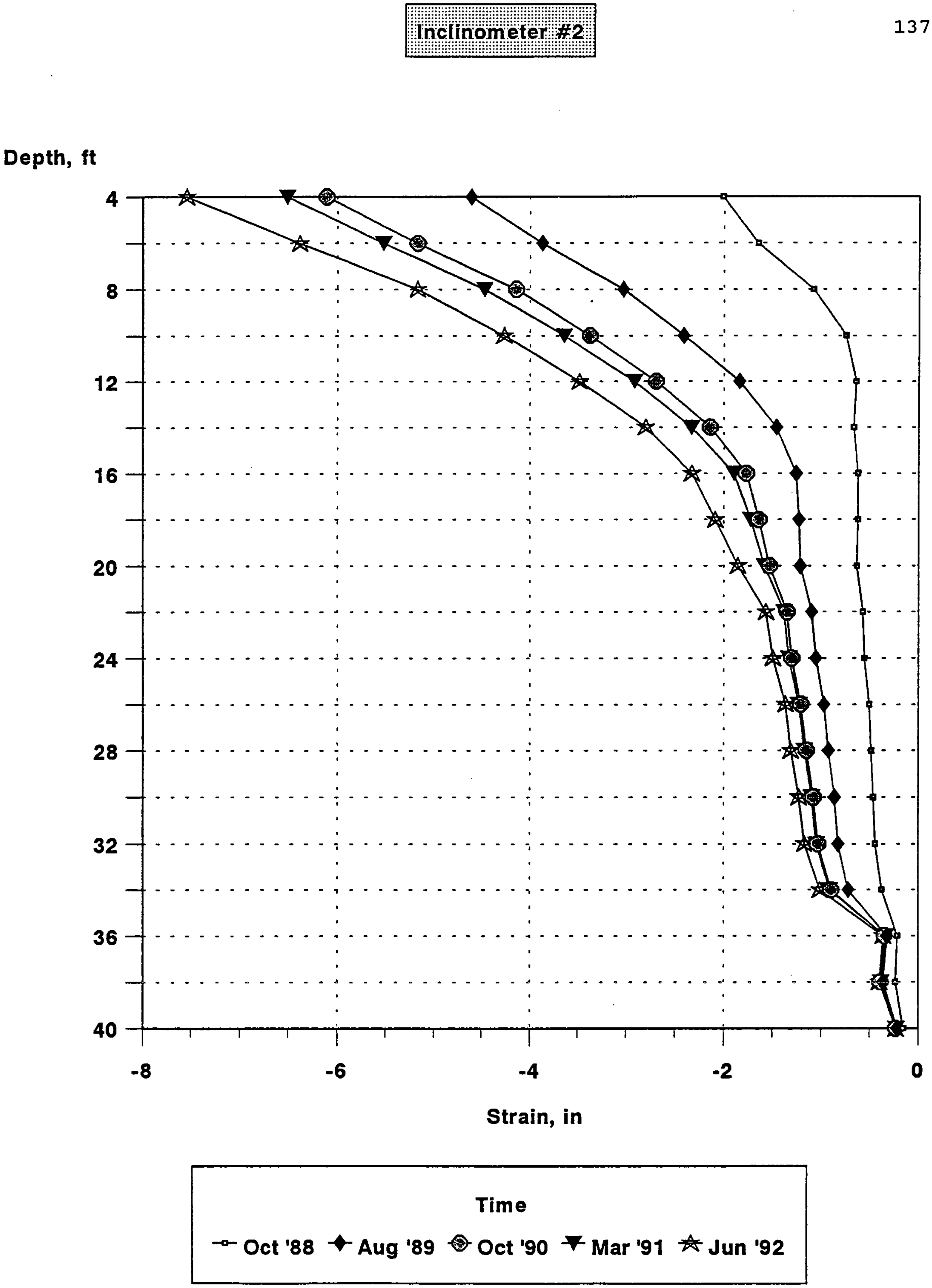

Inclinometer #2
Depth, ft
4
8
12
16
20
24
28
32
36
40
-8
-6
-4
-2
0
Strain, in
Time
Oct '88
Aug '89
Oct '90
Mar '91
Jun '92

Figure 12

Inclinometer #3

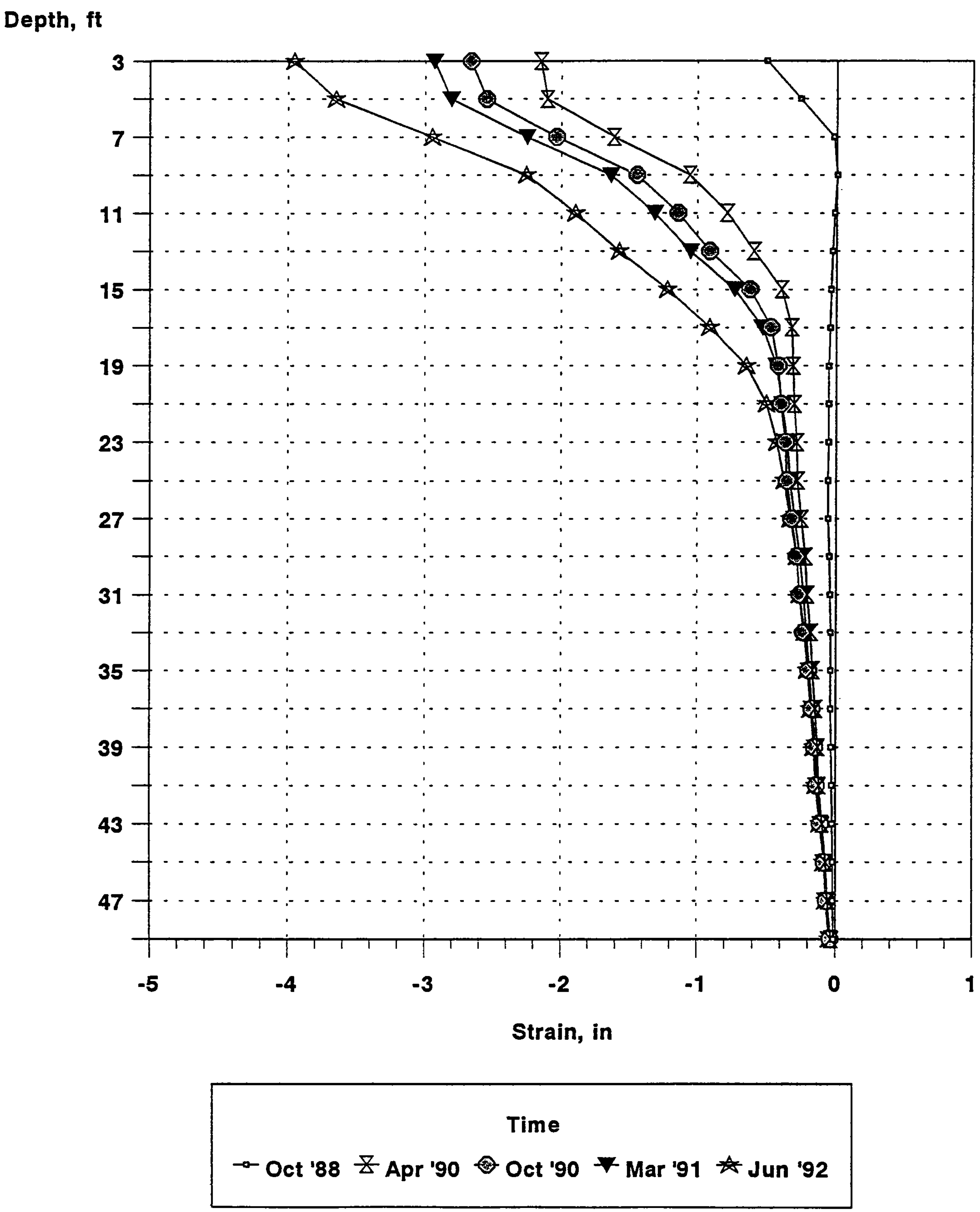


Figure 13

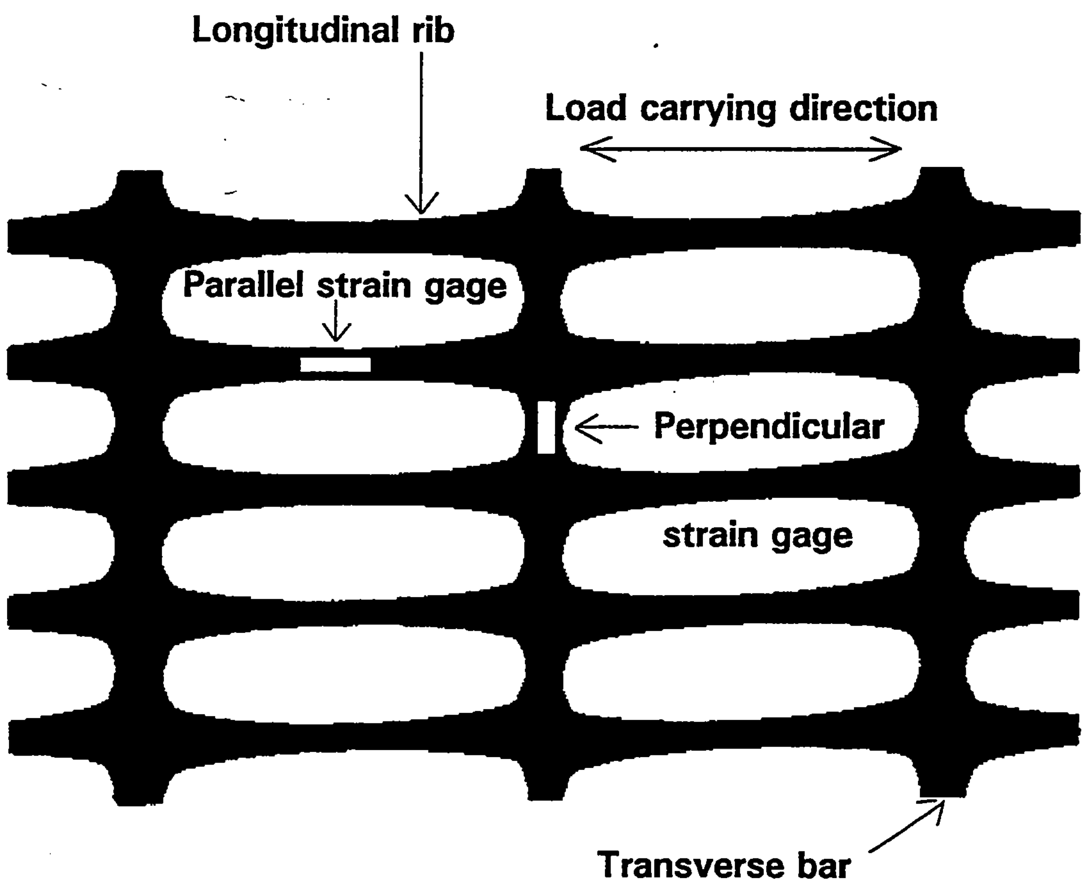


Figure 14 Strain gage installation positions on geogrid

TABLE 1

Gage Set #	Align Direc Ribs	Total # Gages	Total Work Gages	Date of Last Read	Measured total strain (%) to date			
					Avg	Std.Dev	Min	Max
1	Parallel	34	15	Nov 91	0.10	0.08	−0.05	0.29
	Perpend.	4	1	Nov 91	−0.03	–	–	–
2	Parallel	28	21	Nov 91	0.16	0.18	−0.25	0.72

"THE USE OF GEOMEMBRANES FOR MITIGATION OF PYRITIC ROCK"

By

Harry Moore
Tennessee Dept. of Transportation
Geotechnical Operations Section
P.O. Box 58
Knoxville, TN. 37901

ABSTRACT

The mitigation of pyritic containing rock along highways has in the past consisted of treatment with agriculture lime and/or encapsulation in a clay lined earth enclosure. In 1991, the Tennessee Department of Transportation elected to encapsulate 10,000 cubic yards of "hot" acid producing rock material by using synthetic HDPE Geomembrane.

The geomembrane was used on a construction project located in rural and mountainous Unicoi County in East Tennessee. The reconstruction of U.S. 23 (Future I-26) encountered several areas of acid producing rock that required extensive mitigation measures.

Mitigation procedures of the acid producing rock included treatment with agriculture lime and blending with non-acid rock, clay "covers" for treated material, and total encapsulation of subject rock material. Determination of acid potential was by net acid/base accounting.

A geomembrane cover was selected for use to replace the specified 6 feet of compacted clay (as per plans and based on previous experience) used in total encapsulation of acid producing rock. A total of 10,168.5 square yards of 60 mil geomembrane was used. The geomembrane encapsulation area was incorporated as part of the main roadway embankment.

Details concerning pyritic rock testing procedures, and mitigation design were developed and incorporated into the plans as elements of consideration prior to using the geomembrane encapsulation method. Total costs for geomembrane was $70,467.71 while total costs for the mitigation of acid producing rock was in excess of $600,000.00.

INTRODUCTION

The identification and treatment of acid producing rock material encountered along highway construction projects has been gaining more attention and documentation in recent years (Huckabee, et al, 1975; Byerly, 1981 and 1990; Winchester, 1981; Eskenasy and Dunnagan, 1985). The problem with acid producing rock materials is the creation of acidic drainage (usually sulfuric acid along with high metal contents) that results from the breakdown of the sulfidic rock when it is placed in highway embankments.

In the late 1980's, the Tennessee Department of Transportation undertook the design and construction of a new 4-lane 15 mile section of U.S. 23 (future I-26) through the mountainous Blue Ridge Province of Unicoi County, Tennessee. During the design phase the Tennessee D.O.T. Geotechnical Operations Section investigated the possible occurrence of pyritiferous based acid drainage problems as a result of the proposed construction.

Net acid-base accounting (NAB) analysis was used to estimate the potential for acid drainage. A number of researchers have adequately described the analysis procedure (Byerly, 1981; Skovsen et al, 1987; Brady, et al, 1989; Soobek et al, 1978).

As a result of the investigation, approximately 5,000 cubic yards of rock to be excavated was found to contain sufficient quantities of iron disulfides and the possibility of producing adverse acid drainage leachate.

The subject project (the first section consisting of approximately 4 miles in length from Station 644+00± to Station 896+50) was let to contract in March of 1990, with excavation work beginning immediately. Detailed sampling and testing of the rock as it was drilled and blasted for excavation was performed. As a result, a more definitive approach was established and accurate locations and quantities were derived.

LOCATION AND GEOLOGIC CONDITIONS

The project area is located in the mountainous Blue Ridge Province of East Tennessee. Specifically it is located along South Indian Creek in the Temple Hill section of Unicoi County, adjacent to U.S. 23. Steep rugged mountain slopes floored with a narrow stream valley comprise

LOCATION OF REPORT AREA

ALONG U.S. 23

UNICOI COUNTY, TENNESSEE

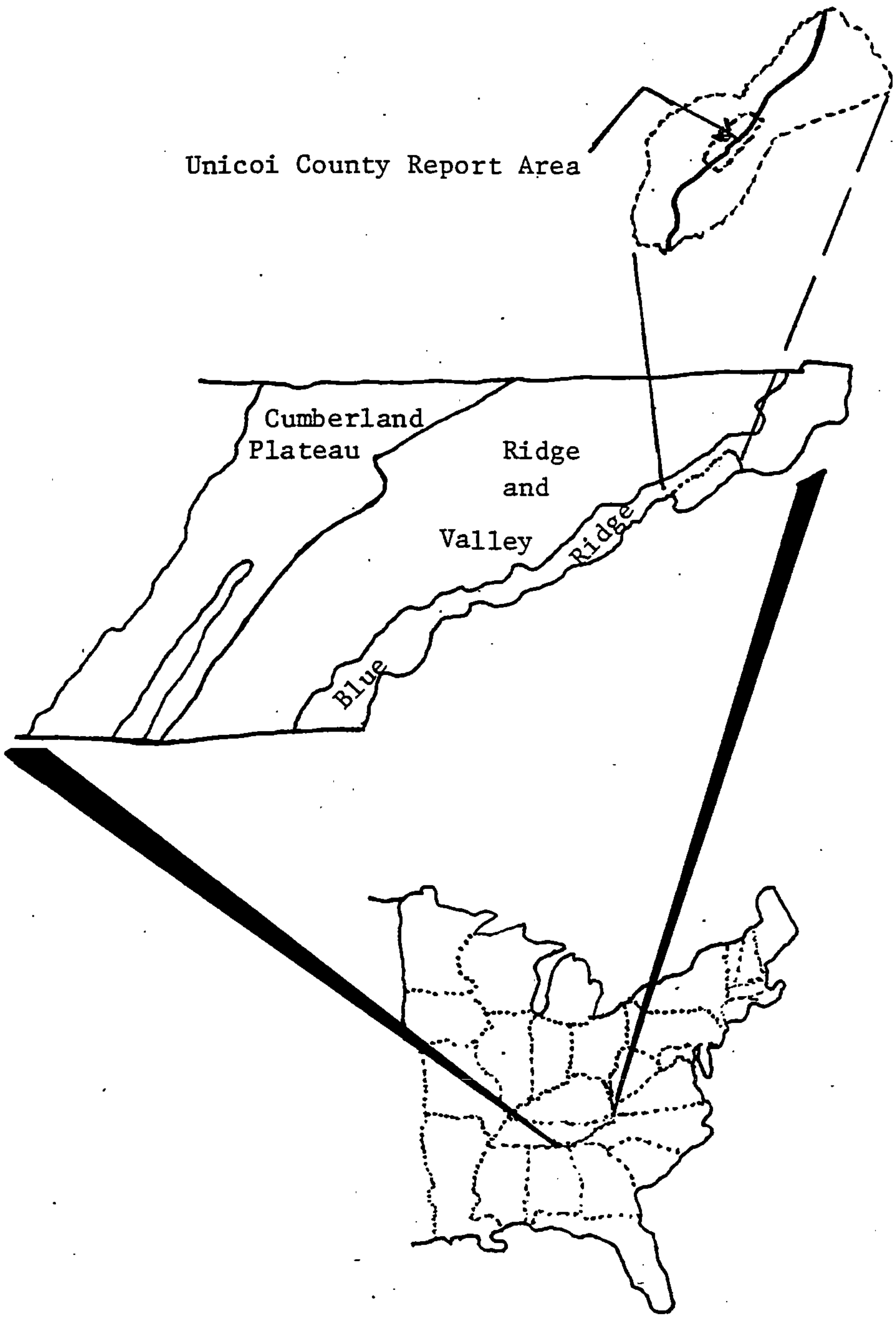


Fig.1 General location of project area.

the general topographic conditions. Geologically, the subject site is situated along the faulted contact between Cambrian sedimentary strata and Precambrian metamorphic basement material.

Structurally, the strata are folded and thus steeply inclined. Severe shearing of the strata occurs along the main thrust plate along with several en echelon shears.

The acid producing rock encountered on the subject project was located in a phyllite lithology of the Precambrian age Walden Creek Group (possible Snowbird Group). The subject lithology was found to consist of two parts: a gray to greenish-gray to black banded phyllite and slate with linear concentrations of crystal pyrite and chalcopyrite (parallel to foliation); and a black, slick-n-sided graphitic and silicious phyllite with high concentrations of "mobilized" pyrite as well as crystal pyrite.

The majority of the acid producing rock (90%) was located in the graphitic and silicious phyllite found along the major shear zone.

TEST RESULTS AND MITIGATION PROCEDURE

Mitigation details were included in the contract plans and called for the following:

* Testing of rock from production blast drilling for location of potentially acid producing rock.

* Tests were to include analysis for neutralization potential, acid potential, % sulfur, net acid/base accounting, and pH values.

Rock material with net acid/base values greater than 0.0 were not mitigated; rock material with net acid/base values from 0.0 to -5.0 were treated with agricultural lime (@ a rate of 3.38 tons of lime per 1,000 cubic yards of rock material); rock material with values of -5.0 and lower are to be encapsulated as shown on the project plans.

* Water quality testing for acid leachate was to be performed by a contracted consultant with results forwarded to Tennessee D.O.T. representatives.

Mitigation of the subject acid producing rock involved treatment or encapsulation (in accordance with Special Provision 107L - see Appendix A). A total of 200,000 cubic yards of rock material was treated with agricultural lime (@ 3.38 tons of lime/1,000 cu. yds. of rock) and placed in the roadway embankment from Station 657+00 to Station 660_00, Station 679+00 to near Station 686+00, and Station 700+00 to Station 706+00. Approximately 166,000 cubic yards came from the large cut (shear zone) near Station 650+00± and 34,000 cubic yards came from Ramp 1-"D" and Frontage Road "Z" (near the Temple Hill Interchange).

Approximately 15,000 cubic yards of rock was found to have a high potential for producing acid leachate and was subsequently encapsulated. The majority of the encapsulated pyritic rock came from the cut at Station 650+00 with smaller quantities coming from the footing excavation of the bridges at Station 665+00 and Station 690+00, Ramp 1-"D", and Frontage Road "Z".

The encapsulation process involved two methods: The original clay liner method as specified in the contract plans, and a geomembrane method, which was conceptualized after the project was underway. The location of the clay liner method is right of centerline Station 669+00±, off the roadway but within the D.O.T. right of way. The geomembrane method was used in the roadway embankment along centerline Station 671+50± to Station 676+00± (Temple Hill Interchange area).

Test results of the samples taken during the construction project were evaluated by the Tennessee D.O.T. Geotechnical Operations Section with the resulting required mitigation procedure being relayed to the construction field engineer. A total of 3,520 samples were taken and tested during the subject construction project. Statistical frequency distribution analysis of the test results indicated the following:

* <u>Net Acid/Base Values:</u> Approximately 1% of the samples tested resulted in values below 0.0.
* <u>% Sulfur:</u> Approximately 82.7% of the samples tested resulted in % sulfur values of 0.07% or less.
* <u>pH:</u> Approximately 68.2% of the samples tested had pH values from 8.0 to 9.0.

* The average values are as follows:

Net Acid/Base-------+33.57
% Sulfur------------0.07%
Paste pH------------8.14

Fig.2 Sampling production drill cuttings for acid/base testing during construction.

Fig.3 Construction of the clay liner encapsulation method showing limestone drainage pad and early stages of clay liner construction.

ENCAPSULATION PROCESS

As mentioned above, two encapsulation methods were employed on the project: A clay liner method (as outlined in the contract plans) and a geomembrane method (developed during the construction project).

The clay liner method followed accepted procedures as researched and implemented on previous Tennessee Highway Projects (Byerly, 1981). Details of the clay liner method are as follows:

Clay Liner Method - Approximately 5,000 cubic yds of pyritic rock was encapsulated using the clay liner method. Most of the pyritic rock came from the cut at Station 650+00. A small portion of the encapsulated material came from the excavation of the bridge footings at Station 665+00±.

A 5 foot thick limestone rock pad was placed on the natural surface of the site. A thin choker layer of crushed stone and a layer of filter cloth were placed on the rock pad. Next, a 6 foot thick layer of clay was compacted into place above the filter cloth and also tied into the natural ground along an adjacent hill. After placement and treatment of the subject rock, the 6 foot thick clay liner was brought around and over the top of the pyritic material, encapsulating the material on all sides. Finally, top soil was placed over the clay and seeded.

GEOMEMBRANE METHOD

During the construction project Tennessee D.O.T. Geotechnical personnel investigated the possible use of a synthetic geomembrane for encapsulating the acid producing rock. Specifications and samples of selected geomembranes were reviewed in preparation of the possible use of the material.

Fig.4 Side view of clay liner encapsulation during construction showing rock drainage pad, filter cloth and compacted clay liner.

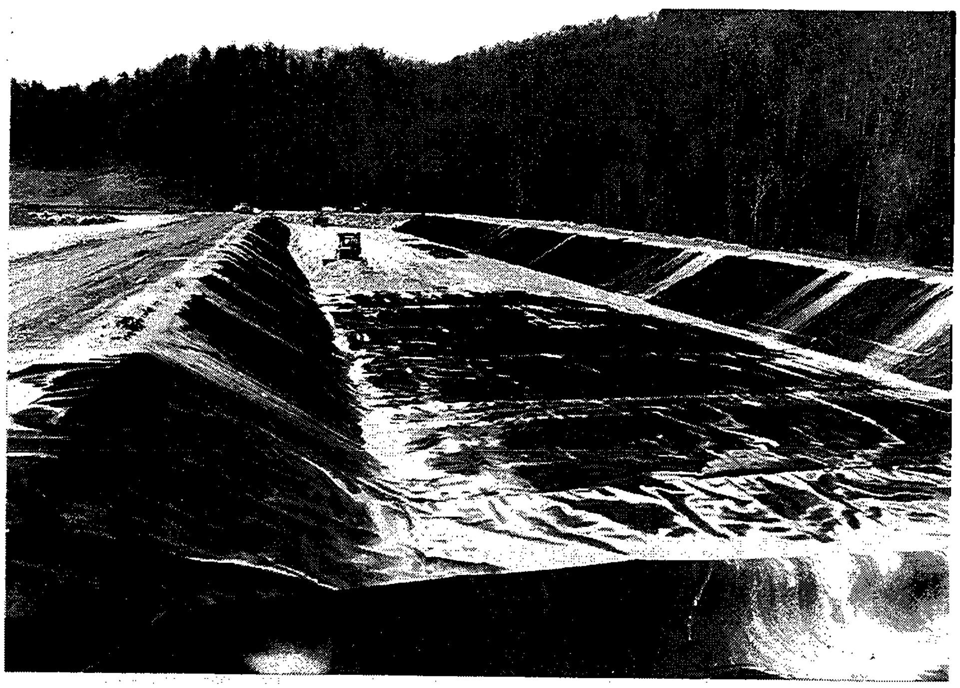


Fig.5 Geomembrane encapsulation area showing trench concept and placement of geomembrane; note geomembrane anchoring scheme along left side.

The material selected consists of a textured surface high density polyethylene membrane. The geomembrane specifications are as follows:

HDPE Friction Textured Geomembrane

Property	Test Method	Test Results
Thickness		60 Mil
Yield Strength	ASTM D 638	130 lbs./inch width
Break Strength	Type IV Specimen at 2 inch/min.	35 lbs./inch width
Yield Elongation		13%
Puncture Resistance	FTMS 101B Method 2065	70
Bonded Seam Strength	ASTM D 3083	65 lbs./inch width
Peel Adhesion	ASTM D 413	50 FTB lbs./inch width

The following outlines the geomembrane method of encapsulation:

The geomembrane method was used in conjunction with the construction of the roadway fill from Station 671+50± to Station 676+00±. Approximately 10,000 cubic yards of pyritic material were encapsulated at this site.

The lower 15± feet of the subject embankment fill was constructed of rock (lower 6-8 feet) and clay soil (next 7-10 feet). On top of this material the two outer sides of the embankment were constructed leaving the interior a "hollow" trench, measuring 450 feet long by 12 feet high and 64 feet wide (looking much like a ground silo). A 1 foot layer of very fine grained soil was placed on the interior of the trench, and compacted. All protruding rock fragments were removed in order to prevent puncture of the geomembrane.

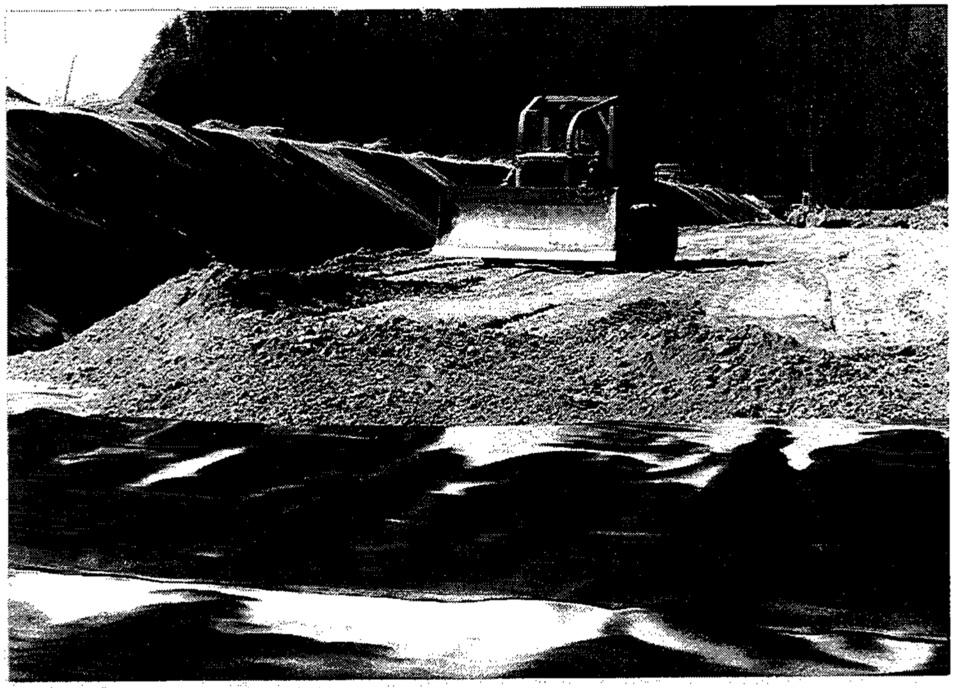


Fig.6 Placement of minimum one foot thick agricultural lime along bottom of geomembrane lined trench.

Fig.7 This photo illustrates the compacted nature of the acid rock material placed in the geomembrane encapsulation area.

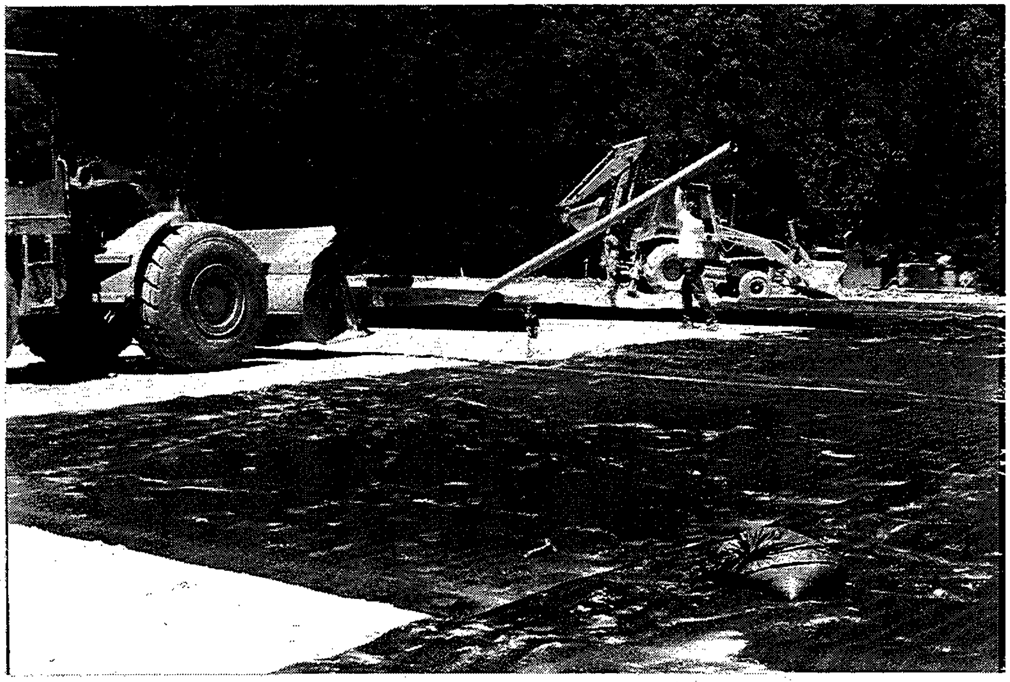


Fig.8 This view shows the placement of the top geomembrane sheets for the encapsulation area; the geomembrane is resting on a one foot thick layer of agricultural lime.

Fig.9 Geomembrane encapsulation site nearing completion; foreground slope is open-end of trench with agriculture lime exposed - soon to be enclosed in geomembrane.

The geomembrane used consisted of 60 mil thick high density polyethylene membrane with a friction textured surface (on both sides). The membrane material came in rolls 400 feet long and 22 feet wide. Poly-Flex, Inc. supplied the geomembrane and personnel to place and seam the material. Placement of the geomembrane began on March 25, 1991. The geomembrane was placed into the trench with the seams of each strip of geomembrane being perpendicular to the roadway centerline. All seams were double welded, wedge heat bonded and vacuum tested. A 1 foot thick layer of agricultural lime was placed along the bottom and sides of the trench, on top of the geomembrane. Pyritic material was then placed in 2 foot thick lifts and treated with the agricultural lime (3.38 tons of lime/1,000 cu. yds. of rock).

After placement of all the pyritic rock was completed, then a 1 foot thick layer of agricultural lime was compacted on top of the rock. The geomembrane was then sealed over the top and end, completely encapsulating the subject material.

Approximately 5 to 7 feet of clay material was placed over the top of the geomembrane bringing the embankment up to planned grade elevations.

Placement of all of the subject rock material took approximately 5 months, leaving some of the pyritic material open to the weather. Water samples of leachate coming from the subject encapsulation site were taken and tested. The results indicated no acid leachate was being generated from the site. The acid levels of the leachate were being eliminated by the agricultural lime used in the geomembrane encapsulation method.

The cost of the geomembrane was $6.93/sq.yd. which included the cost of the material, placement and seaming. A total of 10,168.5 sq.yds were used at a cost of $70,467.71.

CONCLUSIONS

After this first attempt at encapsulating acid producing rock using geomembranes, the results are very acceptable. There are several items that should be given attention should one choose to use the geomembrane encapsulation process. These include:

* The site should be relatively level; placing the encapsulation area on a side hill is not advisable.
* A good properly designed anchoring system for geomembrane installation is recommended; windy days should be avoided.
* The floor of the encapsulation area should be placed on about a 0.5 to 1.0 % grade, draining to an open end; this will provide for gravity flow of infiltrating surface precipitation during placement of the acid rock material, with the subsequent leachate draining out at one end. Once placement is completed and the geomembrane is sealed, the encapsulated area should be air and water tight.
* If used as part of a roadway embankment the acid producing rock should be placed in maximum 2 foot lifts and compacted according to standard specifications; if the rock is shale or phyllite then lift thickness should be from 8 inches to 12 inches in thickness.
* Monitoring of leachate is a must.

The total cost of mitigating the acid producing rock on this project can be divided into testing and mitigation costs. The following details these costs:

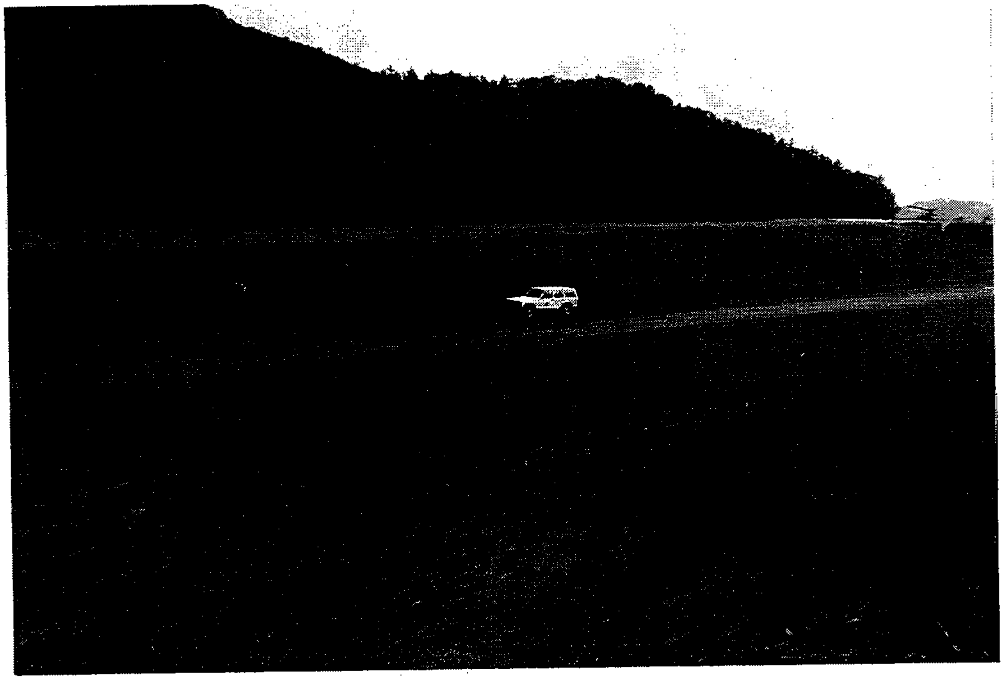


Fig.10 Completed geomembrane encapsulation site with five feet of compacted fill atop the encapsulated material; fill is to road subgrade.

* Laboratory Testing: A total of 3,520 samples were tested at a cost of $32.00 per sample. Total cost is $112,640.00.

* Mitigation:

Geomembrane- 10,168.5 sq.yds @ $6.93/sq.yd.
= $70,467.71
Agricultural Lime - 9,682.3 tons @ $40.00/ton
= $387,292.00
15,000 cu. yds of overhaul @ $2.69/cu.yd.
= $40,350.00
Non-roadway encapsulation @ $1.20/cu.yd.
= $6,000.00

*	Total Mitigation	= $504,109.71
	Total Testing	= $112,640.00
	Grand Total	= $616,749.71

The total cost of testing and mitigation ($616,749.71) does not include engineering costs. The project is still being monitored for acid leachate by water quality sampling and testing. As of this writing, no toxic acid leachate has been detected. Planned monitoring of the stability and performance of the two encapsulated areas is to continue for several years after completion of this project.

Three additional construction projects have been let to contract, completing the 15 mile stretch of new highway through the mountains. According to preconstruction geotechnical investigations, an additional 150,000 cubic yards of acid rock material (to be encapsulated) will be encountered on the remaining 11 miles of road. Detailed mitigation measures using geomembrane and agricultural lime were included in the three remaining contracts and are being implemented as of this writing.

REFERENCES

Brady, Keith, and R.J. Hornberger, 1989. Mine Drainage Prediction and Overburden Analysis in Pennsylvania. In Proceedings of Annual West Virginia Surface Mine Drainage Task Force Symposium, April 25-26, 1989, Morgantown, W.VA., 13p.

Byerly, D.W. 1981. Evaluation of the acid drainage potential of certain Precambrian rocks in the Blue Ridge Province. In proceedings of the 32nd Annual Highway Geology Symposium, May 6-8, 1981, Gatlinburg, Tenn., p.174-185.

Byerly, D.W. 1990. Guidelines for handling excavated acid-producing materials. Federal Highway Administration Special Document D.O.T. FHWA - DF-89-0011, 81 p.

Eskenasy, Diane M.A. and Charles A. Dunnagan., 1985. Toxic rock syndrome. In abstracts and program, AEG 28th Annual Meeting, Oct. 7-11, 1985, Winston Salem, N.C., p. 61.

Huckabee, J.W.; C.P. Goodyear, and R.D. Jones, 1975. Acid rock in the Great Smokies: Unanticipated impact on aquaticbiota of road construction in regions of sulfide mineralization: Transactions of the American Fish Society, No. 4, pp 677-684.

Skousen, J.G.; J.C. Sencindiver; and R.M. Smith., 1987. A review of procedures for surface mining and reclamation in areas with acid-producing materials: West Virginia University Energy and Water Research Center, Publication EWRC 871, 39p.

Sobek, A.A.; W.A. Schuller; J.R. Freeman; and R.M. Smith, 1978. Field and Laboratory Methods Applicable to overburden and Minesoils. E.P.A. Publication #EPA 600/2-78-054.

Winchester P.W., 1981. Some geotechnical aspects - early planning along Corridor K, Appalachian Development Highway; Section between Andrews and Almond, North Carolina. In proceedings of the 32nd Annual Highway Geology Symposium, May 6-8, 1981, Gatlinburg, Tenn., pp186-202.

S T A T E O F T E N N E S S E E

APPENDIX A

October 3, 1990
Sheet 1 of 4
Project No.
County:

SPECIAL PROVISION

REGARDING

ACID PRODUCING ROCK MATERIALS

Description. This work shall consist of locating, sampling, testing and disposing of acid producing rock materials in accordance with the Standard Specifications except as modified herein or as directed by the Engineer.

Acid producing rock materials are those rock and rock like materials (including all rock types, minerals, ore deposits, coal, etc.) that contain sufficient amounts of certain minerals to produce acid levels of leachate when exposed to atmospheric conditions and weathering processes.

In the event acid producing materials exist in the excavation, proper monitoring, testing and disposition of the material shall be required.

The following procedure describes the process of montioring acid producing materials in highway construction projects. The intent is to locate acid producing rock within a given cut interval before excavation of the rock material proceeds. When acid producing rock is encountered, it shall be disposed of under the direction of the Engineer.

Acid Producing Rock Monitoring Procedure. Prior to excavating any potential acid producing rock materials as determined by the Engineer, samples of cuttings shall be obtained by the Engineer during the drilling operation and then tested using the acid/base accounting method. A minimum of three holes for each blast are to be sampled (additional holes may be sampled when deemed necessary by the Engineer). Holes to be sampled will be selected to obtain optimum coverage of the rock material.

A sample shall consist of approximataly one cup of powdered rock material taken from the drill cuttings. Samples are to be taken at five foot intervals for the full depth of the drill hole.

Upon completion of the sampling procedure on three drill holes or more, the samples will be analyzed by the Engineer for potential acid production using the acid/base accounting method. Each sample will be analyzed for potential acid, potential alkalinity, per cent pyritic sulphur, net acid/base potential and paste pH. Test results will be available in approximately 48 hours.

No potential acid producing rock material is to be excavated prior to completion of the testing procedure. Any delays, idle equipment, etc. caused by the testing program will not form a basis for a claim for additional working time and/or compensation. Once the test results are known, the Engineer will make the final decision on the disposition of any rock material found to have acid drainage potential.

In addition, sampling of the rock material during excavation is required. One sample for every 1,000 cubic yards of excavation will be taken and analyzed by the Engineer. Each sample will be taken from the rock material as it is being hauled from the excavation. Field conditions may warrant a change in the number of samples taken from the excavation.

Treatment and Disposition of Acid Producing Rock Materials. Acid producing rock materials shall be treated and disposed of in accordance with the folowing:

General. Disposal sites on sloping terrain shall be benched to provide stability for the embankment at no additional compensation.

1. Net Acid Base Account of zero (0) and higher: No treatment is required.

2. Net Acid Base Account Less than zero (0) and more positive than minus five (-5): This material shall be placed in the roadway embankment if sufficient area exists or in Contractor acquired waste site(s) if sufficient roadway embankment area does not exist. Placement in roadway embankments shall consist of lifts having a thickness of two feet or less. Each two foot lift of rock material shall be treated with agricultural lime or limestone screenings at an application rate of 500 pounds per 1,000 square feet of surface area (3.38 tons of lime or screenings per 1,000 cubic yards of rock material). Placement in waste areas will be in strict accordance with any applicable permits as well as recognized construction operations with treatment by agricultural lime or limestone screenings at a rate of 3.38 tons/1000 cubic yards.

   The outer two feet of the embankment (except the subgrade area under the roadway pavement and shoulders) or waste site shall be sealed with A-6 or A-7-5 clay type soil to reduce the infiltration of surface water. Topsoil, seeding and mulching shall immediately follow.

3. Net Acid Base Account of minus five (-5) or below (more negative): This material shall be placed in approved disposal areas either within roadway embankments or waste sites, as applicable.

   A five foot thick layer of limestone (Borrow Excavation (Graded Solid Rock)) shall be placed in the bottom of the disposal area and covered with a layer of geotextile fabric. Then a six foot thick layer of compacted A-6 or A-7-5 clay type soil shall be placed on the filter cloth. A layer of agricultural lime one foot thick shall then be placed on the clay soil.

The acid producing rock material shall be placed on this prepared foundation. In roadway embankment areas, the material shall be placed in lifts two feet thick and compacted in accordance with Section 205 of the Standard Specifications. Each lift shall be treated with agricultural lime spread at a rate of 500 pounds per 1,000 square feet of lift surface.

After placement of the last lift of acid producing rock material, the entire exposed surface of the disposal area shall be covered with a layer of agricultural lime one foot thick and then capped with a six foot thick layer of compacted A-6 or A-7-5 clay type soil to complete the encapsulation process. Topsoil, seeding and mulching shall immediately follow.

When acid producing rock material is encountered in the construction, it must be separated to the extent feasible utilizing normal excavation procedures and transported as soon as possible to the selected disposal site. At the disposal site the material shall be placed and treated as specified herein. Any questionable material identified during construction will be temporarily covered with ten (10) mil polyethlyene sheeting until laboratory results determine its quality and manner of disposition. Cost of polyethylene sheeting will be included in the cost of other items of construction.

Materials. Borrow Excavation (Graded Solid Rock) shall consist of sound, non-degradable limestone with a maximum size of three feet. At least 50 percent by weight of the rock shall be uniformly distributed between one foot and three feet in diameter, and no greater than 10 percent by weight shall be less than two inches in diameter. The material shall be roughly equi-dimensional. Thin, slabby material will not be accepted.

Agricultural Limestone shall meet the requirements of Subsection 918.17.

Limestone Screenings shall consist of not less than eighty-five percent of calcium carbonate and magnesium carbonate combined and shall be Size No. 10 as specified in Subsection 903.22.

The Filter Cloth shall meet the requirements of Subsection 918.27 of the Standard Specifications.

COMPENSATION

Method of Measurement. The items of Borrow Excavation (Graded Solid Rock), Agricultural Limestone and Limestone screenings will each be measured by the ton.

Road and Drainage Excavation (Unclassified) and Borrow Excavation (Select Material) will be measured by the cubic yard in its original position by cross-sectioning the area excavated.

The filter cloth will be measured by the square yard.

Basis of Payment. The accepted quantities of materials will be paid for at the contract unit price per ton for Borrow Excavation (Graded Solid Rock), per ton for Agricultural Limestone, per ton for Limestone Screenings and per square yard for Filter Cloth, complete in place.

Clay like soil (A-6 or A-7-5) obtained from the roadway excavation and used for encapsulation of acid producing rock materials will be paid for by the cubic yard under Item 203-01 Road and Drainage Excavation (Unclassified). If this material is not available from the roadway excavation, it will be paid for by the cubic yard under Item 203-03.01 Borrow Excavation (Select Material).

In the event that a contract unit price is not available for any item needed for treatment and disposal of acid producing rock materials, the work shall be performed in accordance with Subsection 104.03.

The contract unit prices thus paid shall be full compensation for all excavation, hauling, site preparation, placement, encapsulation, treatment and incidentals associated with proper disposal of acid producing rock materials.

Payment will be made under:

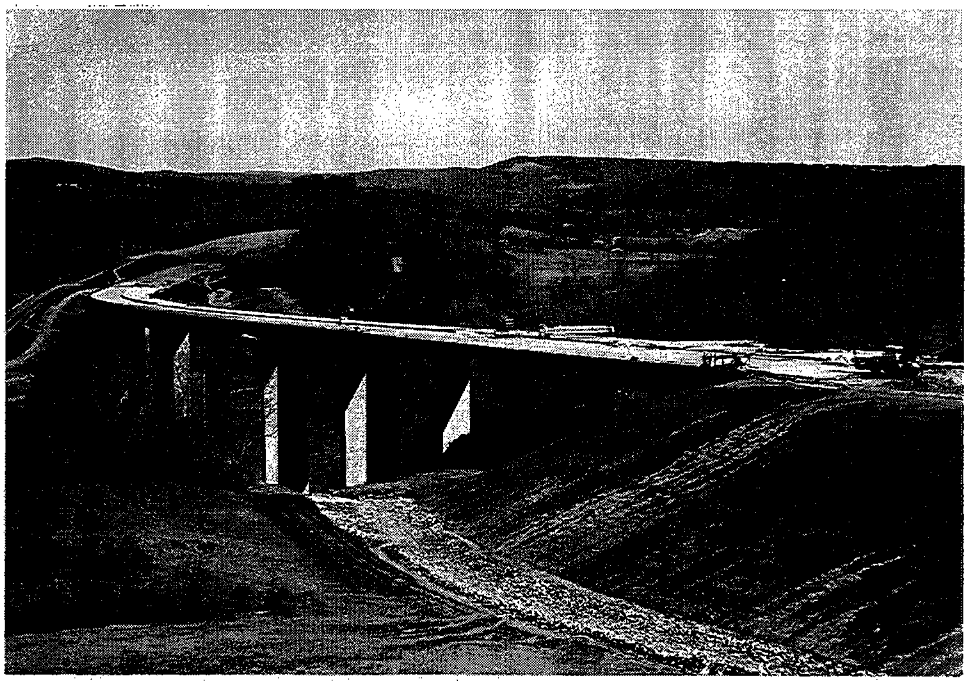
Item No.	Pay Item	Pay Unit
203-01	Rd.& Dr.Excav. (Uncl)	Cubic Yard
203-02.01	Borrow Excav. (Graded Solid Rock)	Ton
203-03.01	Borow Excavation (Select Material)	Cubic Yard
709-13.01	Geotextile Fabric	Square Yard
710-13.03	Filter Cloth	Square Yard
801	Agricultural Limestone	Ton
801-10	Limestone Screenings	Ton

A guidebook to the

HIGHWAY GEOLOGY AT SELECTED SITES IN THE BOSTON MOUNTAINS AND ARKANSAS VALLEY, NORTHWEST ARKANSAS

By

David W. Lumbert and Charles G. Stone



Prepared for the 43rd Highway Geology Symposium

Dr. Sam I. Thornton, Chairman
Fayetteville, Arkansas
August 19-21, 1992

A guidebook to the

HIGHWAY GEOLOGY AT SELECTED SITES IN THE BOSTON MOUNTAINS

AND

ARKANSAS VALLEY, NORTHWEST ARKANSAS

By

David W. Lumbert and Charles G. Stone

Prepared for the 43rd Highway Geology Symposium

Dr. Sam I. Thornton, Chairman
Fayetteville, Arkansas
August 19-21, 1992

INTRODUCTION

This guidebook was prepared for the 43rd Highway Geology Symposium's one-day field trip in northwest Arkansas, August 20, 1992. The primary purpose of the trip is to illustrate the geotechnical conditions encountered at selected sites along the newly "relocated" U.S. Highway 71. To illustrate the wide variety of geotechnical problems encountered in highway construction in this region (including those caused by deep weathering, terrace deposits, alluvium, soils, springs and other features), two additional stops on nearby highways are included.

Rapid population growth and development in northwest Arkansas during the 1960's created excessive traffic demand on the arterial highway system. Congestion along U.S. Highway 71 caused undue delays in north-south traffic, particularly in the Fayetteville-Springdale and Rogers-Bentonville areas.

Construction of the U.S. Highway 71 Bypass at Fayetteville afforded some relief in the congestion; however, diverted through traffic combined with additional traffic generated by rapidly expanding urban development caused the congestion to increase again. To alleviate the congestion, the bypass was upgraded to a fully controlled access four-lane facility with interchanges and grade separations.

This project was followed by construction of a connecting four-lane route for U.S. Highway 71 between Fayetteville and Bentonville.

The Arkansas Highway Commission recognized the need for further improved road facilities in northwest Arkansas and authorized two studies which established a north-south freeway or expressway project for the U.S. Highway 71 relocation between the vicinity of Alma and Fayetteville.

The Relocation Project is a four-lane controlled access facility extending approximately 43 miles from I-40 in Crawford County northward to U.S. Highway 71 Bypass at Fayetteville in Washington County. The project will consist of two twelve-foot lanes in each direction separated by a variable-width median. The right-of-way width will average 300-400 feet. Access will be fully controlled with interchanges and grade separations (overpasses and underpasses) at selected locations.

The design speed for the project is 65 mph. Roadway grades will be limited to 5 percent and maximum horizontal curves will be 5^{o} 30'. The construction on the relocated U.S. Highway 71 started near Alma in January of 1987. Some sections are scheduled for use in early 1995, and the anticipated completion date is the year 2000.

After travelling from Fayetteville to Fort Smith, the field trip proper begins at Fort Smith in the mildly deformed shales and sandstones that underlie the gentle rolling landforms in most of the Arkansas Valley (the Arkoma basin of the petroleum industry). The tour then proceeds north from Alma to Fayetteville, crossing the rugged, highly dissected Boston Mountains (southern Ozarks), which are underlain by nearly horizontally bedded massive sandstones and less competent shales, with some fossiliferous limestones.

We gratefully acknowledge the contributions of Jim Gee and Jake Clements of the Arkansas State Highway and Transportation Department, Division of Materials; Norman F. Williams, George W. Colton, and Norma Lynn Kover of the Arkansas Geological Commission; and Dr. Sam I. Thornton of the Department of Civil Engineering, University of Arkansas. Their assistance in selecting field trip stops and in the preparation of the guidebook were truly invaluable.

GENERAL GEOLOGIC SETTING

(Note: The information included in this section has for the most part been abstracted or adapted from many publications on the area, notably the reports by Zachry and Harris, Brewster and Williams, Hendricks and Parks, Sutherland and Manger, Haley and Hendricks, and Croneis. These publications are listed in the accompanying bibliography.)

The field trip starts in the western part of Arkansas Valley section of the Ouachita province and continues northward across the Boston Mountains section of the Ozark Plateaus province (Figure 1).

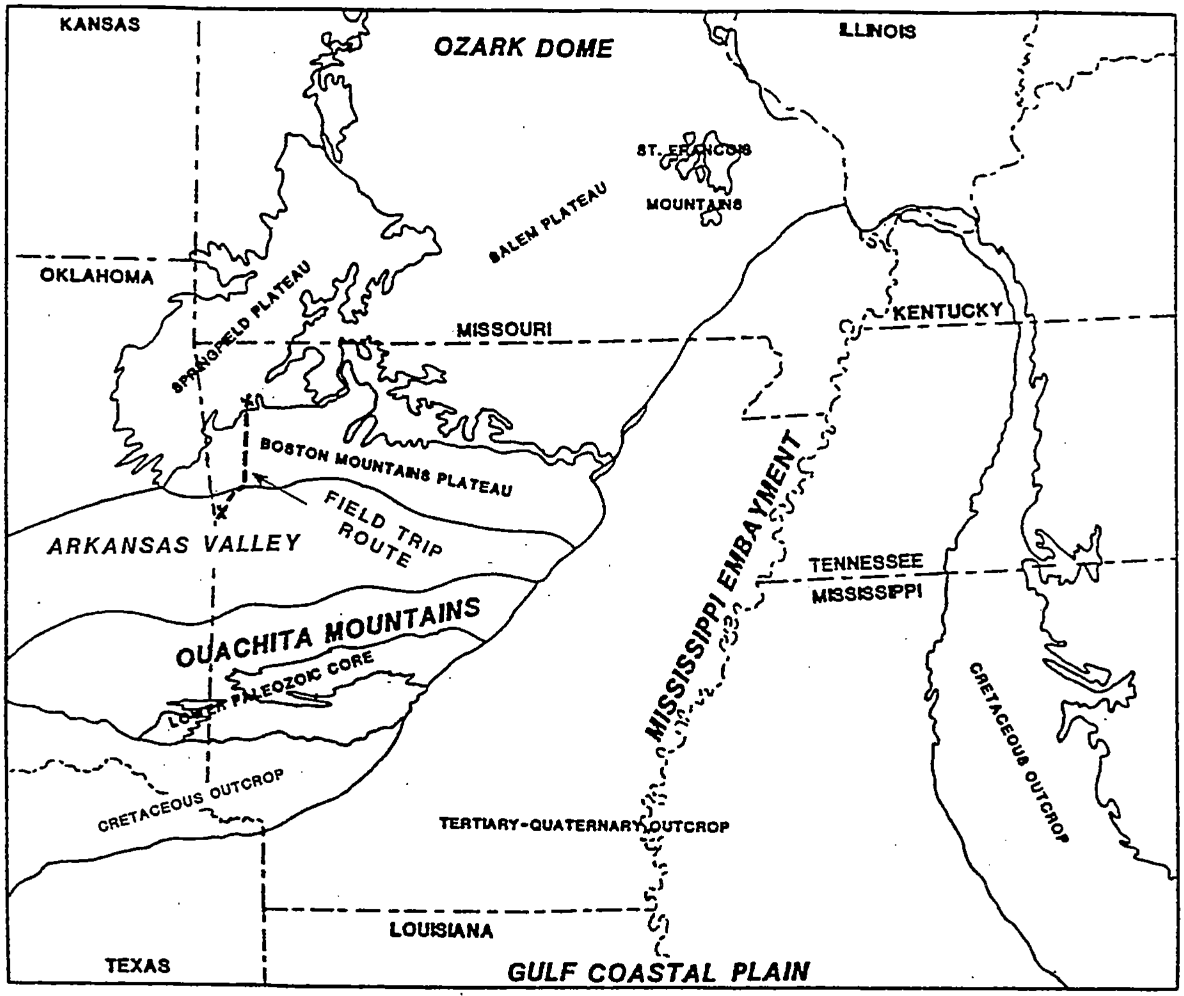


Figure 1. -- *Geologic provinces of Arkansas and nearby areas.*

The exposed stratified rocks in the western part of the Arkansas Valley belong to the Atoka, Hartshorne, and McAlester formations of Early and Middle Pennsylvanian age (Figure 2). They consist mainly of alternating beds of shale and sandstone, but include some coal beds. In this area they increase in thickness from about 7000 feet in the north to about 11,000 feet in the south.

SYSTEM	SERIES	FORMATION	COLUMNAR SECTION	THICKNESS IN FEET	CHARACTER OF ROCKS
PENNSYLVANIAN	DES MOINES	Savanna formation	COAL; CHARLESTON COAL; UNCONFORMITY	450-850	Light-gray to light brownish gray, micaceous, very fine grained to fine-grained sandstone (cross-bedded and ripple-marked; contains plant fossils as well as marine invertebrates) interbedded with medium dark gray shale and siltstone.
		McAlester formation	COAL; COAL; LOWER HARTSHORNE COAL	500-1000	Light-gray to light brownish gray, micaceous, very fine grained to fine-grained sandstone (cross-bedded and ripple-marked; contains plant fossils as well as marine invertebrates) interbedded with medium dark gray shale and siltstone.
		Hartshorne sandstone	COAL; UNCONFORMITY	15-100	Thin-bedded to massive, very light gray, micaceous, very fine to fine-grained sandstone.
	ATOKA	Atoka formation	COAL; COAL; COAL; UNCONFORMITY	2,300-10,000	Sandstones and dark to black sandy, micaceous shales. Sandstones usually brown, thin to medium bedded, compact and ripple marked. Massive, cross-bedded sandstone with white quartz pebbles near base.
	MORROW	Bloyd shale: Kessler limestone member		(0-20)	Compact, gray to brown, fossiliferous limestone, locally conglomeratic.
		Bloyd shale: Woolsey member	BALDWIN COAL	(0-45)	Terrestrial sediments (including the Baldwin coal bed up to 18 inches thick). Conglomerates at top and base locally.
		Bloyd shale		0-350	
		Bloyd shale: Brentwood limestone member		(0-100)	One, two or three limestone beds, each 3 to 10 feet thick with interbedded shales. Limestones impure, fossiliferous, and locally cross-bedded.
		Hale formation: Prairie Grove member		(60-200)	Gray, limy, fine to medium-grained sandstone, fossiliferous oolitic limestone, and, locally, a basal conglomerate.
		Hale formation		60-250	
		Hale formation: Cane Hill member	UNCONFORMITY	(0-65)	Shale, siltstone, and fine-grained sandstone. Locally limy. Basal conglomerate.
MISSISSIPPIAN	UPPER	Pitkin limestone	UNCONFORMITY?	0-200	Massive, gray, fossiliferous limestone.
		Fayetteville shale: Wedington sandstone member		(0-55)	Gray to brown sandstone, in part calcareous; and massive and cross bedded in places.
		Fayetteville shale		10-350	Black, carbonaceous, pyritic, fissile shale with clay ironstone concretions. Thin, dark limestones locally.
		Batesville sandstone		0-75?	Gray to brown, calcareous sandstone, weathering to brown or buff, porous sandstone.
		Batesville sandstone: Hindsville limestone member		(0-50?)	Gray, fossiliferous, bituminous limestone, in part oolitic with thin sandstone beds.
		Ruddell shale		?	(Not definitely known to be present in western Arkansas) Gray, fissile clay shale in north central Arkansas.
		Moorefield formation	UNCONFORMITY	0-30?	Outcrops tentatively assigned to this formation are gray, glauconitic, fine to medium crystalline limestone.
	LOWER	Boone formation		50-400	Massive, gray, crystalline, fossiliferous limestone with much nodular and bedded chert.
		Boone formation: St. Joe limestone member	UNCONFORMITY?	(10-100)	Thin-bedded, non-cherty, gray to reddish-brown, crinoidal limestone.

Figure 2. -- *Columnar section of rocks exposed in northwestern Arkansas.*

Gravel-and silty sand-capped terraces were formed by the Arkansas River and some of its tributaries during Pleistocene time at several levels above the present river. Alluvium was deposited in wide areas along the Arkansas River and its major tributaries, and in narrower areas along the smaller streams.

Normal faults are abundant in the northern part of the Arkansas Valley – and in the adjacent southern Boston Mountains --, and vaguely defined folds with flank dips of from two to five degrees are present. More intensely deformed folds, commonly with associated thrust faults, appear successively to the south toward the Ouachita orogenic belt.

The northern part of the Arkansas Valley area developed as a structural feature in middle Atoka (Pennsylvanian) time as an extensional stress field broke the shelf into generally east-trending normal faults with extensive down-to-the-south and lesser down-to-the north displacement (Figure 3). Subsidence caused by the faults accommodated over 25,000 feet of Atoka strata in the southern part of the basin adjacent to the Ouachita Mountains. Approximately 2700 feet of lower Atoka rocks overlie the Morrowan (Pennsylvanian) section immediately south of Frog Bayou (Stop 3), and up to 1500 feet are present in the Boston Mountains to the north.

The Boston Mountains section is a dissected highland that extends from north-central Arkansas to northeastern Oklahoma (Figure 1). Elevations in Arkansas range from 800 to 1000 feet in the valleys up to around 2400 feet on the mountains. The hilltops are flat, and the valleys are generally steep-sided and have relatively narrow floodplains. Carboniferous strata in the Boston Mountains are essentially flat lying and deformation is mainly confined to widely spaced fault zones. Regional dip is to the south at approximately one-third of a degree. The boundary between the mountains and the Arkansas Valley section to the south is the Mulberry fault north of Alma, across which down-to-the-south displacement ranges from 2000 to 2500 feet.

The Boston Mountains are bounded to the north by the successively lower Springfield and Salem plateaus and to the south by the Arkansas Valley section, a peripheral foreland basin associated with the Ouachita orogenic belt in west-central Arkansas and east-central Oklahoma (Figure 1). Paleozoic sedimentary rocks of Mississippian age crop out in the floor and lower valley walls of deeply incised streams. The hilltops and most of the valley walls are underlain by strata of Pennsylvanian age (Figure 2). Carboniferous rocks of the Boston Mountains overlie a Paleozoic sequence that ranges in age from Cambrian to Devonian and has a thickness of 2000 to 2500 feet. Rocks ranging from Lower Ordovician to Devonian are exposed, along with some Carboniferous rocks, on the Springfield and Salem plateaus to the north.

A brief description of the Carboniferous formations and Quaternary deposits exposed along the route of this field trip follows (Figure 2).

MISSISSIPPIAN SYSTEM

Fayetteville Shale (10-350')

Most of this formation is a black carbonaceous fissile clay shale in which clay ironstone concretions are numerous. Locally a thin highly fossiliferous limestone occurs near the base in northwestern Arkansas.

In the upper part of the Fayetteville Shale is a sandstone member, the Wedington, which reaches a thickness of 200 feet, but commonly is less than 50 feet thick. The rock is dense, hard, light gray to brown, fine-grained and generally thin-bedded. Locally, however, it may be massive, especially where the sandstone is flaggy.

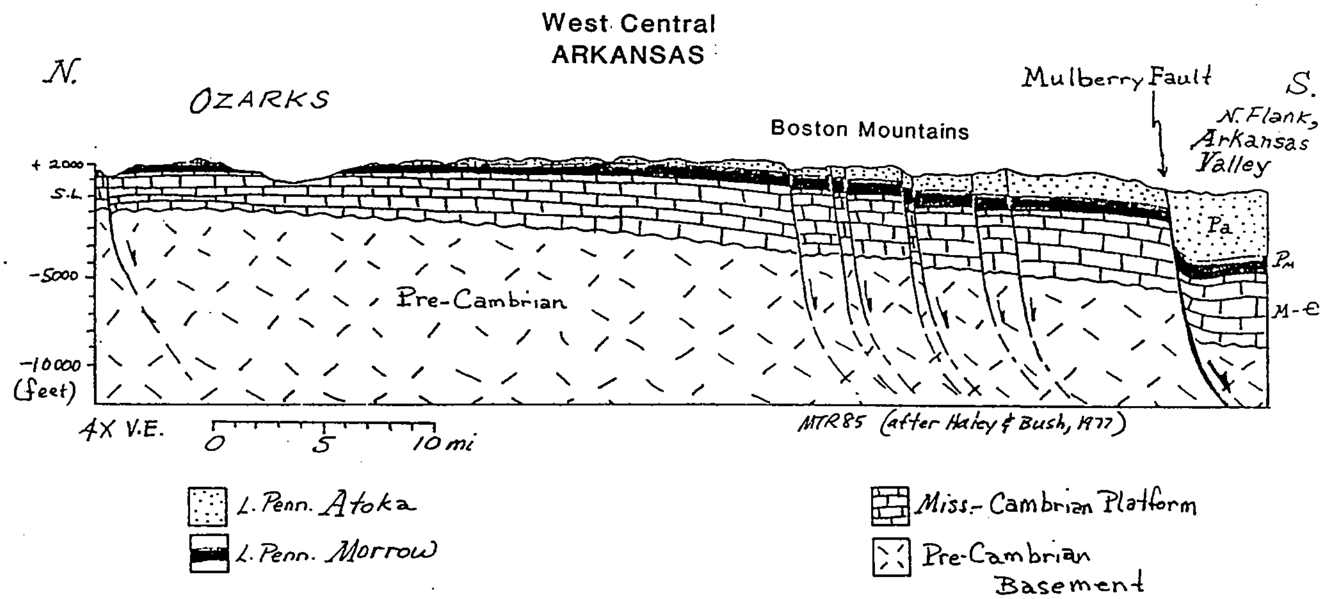


Figure 3. -- *Generalized geologic cross-section across northwest Arkansas. (Adapted from M.T. Roberts).*

Where the Batesville Sandstone is present, it is conformably overlain by the Fayetteville Shale. Elsewhere the Fayetteville lies upon the Boone, except for a few localities where it rests on the Hindsville Limestone Member of the Batesville. The Fayetteville is overlain by the Pitkin Limestone, probably disconformably. Where the Pitkin was removed by pre-Pennsylvanian erosion, the Fayetteville is overlain by some part of the Morrow Group.

Pitkin Limestone (0-200')

The Pitkin Limestone is the uppermost formation of Mississippian age in northwestern Arkansas. It outcrops from near Batesville in Arkansas on the east to near Muskogee in northeastern Oklahoma. Typically the Pitkin consists of massive beds of compact, bluish-gray limestone which in places is ferruginous and porous. Locally it may be sandy enough to show indistinct cross-bedding on weathered surfaces. Its outcrop is usually a vertical cliff with large blocks of the undermined limestone lying on the slope below. Most of the rock is fossiliferous, bryozoans, corals, crinoids, and brachiopods being especially numerous.

The Pitkin rests disconformably on the Fayetteville Shale and is overlain by the Hale Formation with disconformity, as shown by the irregular upper surface of the Pitkin and a conglomerate at the base of the Hale.

PENNSYLVANIAN SYSTEM

Hale Formation (60-250')

The Hale Formation is the lowermost of the Pennsylvanian formations of northern Arkansas. It extends from eastern Oklahoma along the Boston Mountain escarpment to the vicinity of Batesville. Its thickness in northwestern Arkansas probably averages about 150 feet. The formation consists of a shale-rich member below and a sandstone-dominated member above.

The lowermost part of the formation is a basal conglomerate composed of a ferruginous limestone matrix containing pebbles of limestone, sandstone, and cement.

Above the basal conglomerate is a black clay shale with thin ripple-marked sandstones. This shale is normally followed by a sandstone section including thin and massive beds. Above these sandstones is a black fissile clay shale which in places contains much sandstone and sandy shale. The upper part of the formation is characteristically a series of massive, brown, cross-bedded, calcareous sandstones which develop a cavernous surface on weathering. Lenses and beds of limestone occur throughout the sandstone and in places are as much as 50 feet thick.

The Hale is unconformable on the Pitkin, and north of the edge of the Pitkin, the unconformity truncates successively older beds, finally bringing the Hale to rest upon the lower part of the Fayetteville Shale. Where the overlying Bloyd is present, its contact with the Hale is conformable. In some areas the Bloyd was removed by erosion and the Hale is overlain by the Atoka Formation.

Bloyd Shale (0-350')

The Bloyd Shale is best developed in the western part of the Boston Mountains in Arkansas where it is 200 feet thick. It thins to the north, east, and west but extends westward to the vicinity of Muskogee, Oklahoma and eastward into Independence County, Arkansas.

The formation consists of shale and two limestone members. The lower part is a black fissile clay shale 5 to 20 feet thick. Above the shale is the Brentwood Limestone Member, composed of one, two, or three limestone beds, each 3 to 10 feet thick, separated by shales. The limestones are impure, very fossiliferous, vary from fine-grained to crystalline, and are locally cross-bedded.

Above the Brentwood is a black carbonaceous shale in which a thin coal bed occurs. Above the coal-bearing shale is the Kessler Limestone Member. It resembles the Brentwood in general appearance, but is thinner, darker in color, locally conglomeratic and weathers into thin shaly plates. The shale above the Kessler resembles the Fayetteville Shale but is somewhat more sandy.

The Bloyd is conformable with the underlying Hale Formation and is overlain by the Atoka Formation with a slight angular unconformity.

Atoka Formation (2300-10,000')

Rocks of Atoka age are exposed in Arkansas from the Boston Mountains escarpment southward across the Arkansas Valley to the Ouachita Mountains. They extend from the Coastal Plain westward into Oklahoma.

The Atoka Formation is the most widespread formation in Arkansas. It was named for the town of Atoka, Oklahoma, near which it is well exposed. The formation ranges in thickness from about 2300 feet in the Boston Mountains to about 10,000 feet in the central Arkansas Valley section.

The Atoka consists predominantly of shale with lesser amounts of interbedded sandstone. The shale is mostly black, carbonaceous, micaceous, sandy, and splintery. The sandstone beds are white or light gray and coarse grained, or massive to gray or brownish and very fine-grained. All the sandstone is micaceous. Channel sandstones are rather frequently found and there is considerable lateral gradation of sandstone to shale.

Conglomerates are prominent in exposures in the Ozark region, and coal beds are found at several stratigraphic positions in the Atoka formation, but they are usually thin and of limited extent.

The Atoka is underlain by the Bloyd Shale with a slight angular unconformity and is unconformably overlain by the Hartshorne Sandstone.

Hartshorne Sandstone (15-300')

The Hartshorne Sandstone, named for outcrops near Hartshorne, Oklahoma, is one of the important ridge-forming sandstones of the Arkansas Valley. Its area of outcrop is confined principally to the western part of the Arkansas Valley. The formation typically is less than 100 feet thick, but locally may exceed 300 feet.

The Hartshorne consists of sandstone beds that are coarse-grained, massive and cross-bedded where the formation is thick and generally fine-grained and thin-bedded where it is thin. They are typically light gray to white in color and some are ripple marked. The thicker Hartshorne sequences were probably formed as channel deposits. The formation rarely includes invertebrate remains, but plant fossils are numerous. In some localities fossil forests are found. Minor amounts of gray to black, sandy to silty shale are present in the Hartshorne.

The formation rests unconformably on Atoka rocks, and is overlain conformably by the McAlester Shale.

McAlester Formation (500-1800')

The McAlester Formation was named for exposures near the town of McAlester, Oklahoma. The formation usually has a thickness ranging from 500-1000 feet, but reaches a maximum thickness of about 1800 feet in the Poteau Mountain area south of Fort Smith.

The formation consists mainly of gray, sandy, micacous shale with some dark gray to black, clay shale and discontinous beds of sandstone. Most of the sandstone is buff, fine-grained, argillaceous, and micaceous, but some beds of white, coarse-grained, clean sandstone are found. A number of coal beds are present in the McAlester. The most important coal in Arkansas, the Lower Hartshorne coal, is the lower part of the McAlester. The McAlester is overlain unconformably by the Savanna Formation.

QUATERNARY SYSTEM

Terrace Deposits (0-40')

Alluvial terrace deposits of Pleistocene age occur along the Arkansas River and to a lesser extent along other major streams in the region.

The base of the highest Arkansas River terrace is about 50-60 feet above the present river level. It is restricted to only a few places, primarily because of subsequent periodic reworking by the Arkansas River. The terrace is composed of rounded gravels of sandstone, siltstone, chert, quartz, and minor traces of igneous rock, with interstitial clay and sand.

The lower terrace deposits are widespread along the Arkansas River and generally occur about 20 to 30 feet above the present river level. They are composed of red silty clay with small secondary, brownish-white limestone nodules. Generally, the terrace is very sandy. Pebbles reworked from the high terrace deposits commonly occur near the base.

Alluvium (0-260')

Clay, silt, sand and gravel compose the alluvial deposits of the Arkansas River floodplain. Near Fort Smith, the alluvium averages 70 feet, but may locally exceed 200 feet in thickness.

Soils and Colluvium (0-10')

Throughout much of the Arkansas River Valley and Boston Mountains, soils and colluvial slope-wash debris, usually varying from a few inches to about 10 feet in thickness, cover the Paleozoic formations. These deposits are best developed on the flat-lying valley floors and gentle slopes. On the steeper slopes and hills the deposits contain many locally derived angular to subrounded pebbles and cobbles of siltstone, sandstone, and some limestone. Irregular deep weathering often occurs in the Paleozoic strata beneath the soil and colluvial deposits, and a few cavernous features occur in the Paleozoic limestone units. Some springs are present either at the base of the soil and colluvium or within some of the Paleozoic units. Large-scale mass movement of rock has occurred in several areas. It is most notably developed at Devils Den State Park west of Winslow, where jumbled masses of sandstone covering several acres in size have become detached and slid downslope from the bedrock exposures.

ECONOMIC GEOLOGY

Some of the most prolific gas fields in the state are in the Arkansas Valley area between stops 1 and 2 of the field trip. Most of the gas comes from sandstones in the Atoka Formation, but in several fields Morrowan rocks are important producers. In the Boston Mountains at West Fork and near Brentwood, several small natural gas fields produce from older sandstones. In the Arkansas Valley, significant quantities of high-rank bituminous to semi-anthracite coal have been mined, especially from the Lower Hartshorne coal. Rock aggregate is produced commercially from the Hartshorne Sandstone and the Pitkin Limestone and other formations in the region.

SELECTED BIBLIOGRAPHY

Adams, G.I., and Ulrich, E.O., 1905, Description of the Fayetteville quadrangle, Arkansas-Missouri: U.S. Geological Survey Atlas, Folio 119.

Bishop, W.H., 1961, The geology of the Brentwood-Sulphur City area, Washington County, Arkansas: University of Arkansas, unpub. M.S. Thesis.

Brewster, E.B., and Williams, N.F., 1951, Guidebook to the Paleozoic rocks of northwest Arkansas: Arkansas Geological and Conservation Commission Guidebook, GB 51-1.

Bush, W.V., and others, 1977, A guidebook to the geology of the Arkansas Paleozoic area: Arkansas Geological Commission Guidebook, GB 77-1.

Caplan, W.M., 1957, Subsurface geology of northwestern Arkansas: Arkansas Geological and Conservation Commission Information Circular, IC-19.

Haley, B.R., and others, 1976, Geologic map of Arkansas: U.S. Geological Survey map, scale 1:500,000.

Haley, B.R., and Hendricks, T.A., 1971, Geology of the Van Buren and Lavaca quadrangles, Arkansas and Oklahoma: U.S. Geological Survey Professional Paper 657-A.

Croneis, Carey, 1930, Geology of the Arkansas Paleozoic area: Arkansas Geological Commission Bulletin 3.

Easton, W.H., 1942, Pitkin Limestone of northern Arkansas: Arkansas Geological Survey Bulletin 8.

Hendricks, T.A., and Parks, Bryan, 1950, Geology of the Fort Smith district, Arkansas: U.S. Geological Survey Professional Paper 221-E.

Purdue, A.H., 1907, Description of the Winslow quadrangle, Arkansas-Indian Territory: U.S. Geological Survey Atlas, Folio 154.

Roberts, M.T., 1985, Carboniferous shelf and basin facies of eastern Oklahoma: A.W.G. field trip, Tulsa, Oklahoma.

Sutherland, P.K., and Manger, W.L., 1979, Mississippian-Pennsylvanian shelf to basin transition, Ozark and Ouachita regions, Oklahoma and Arkansas: Oklahoma Geological Survey Guidebook 19.

Zachry, D.L., 1977, Stratigraphy of middle and upper Bloyd strata (Pennsylvanian, Morrowan), northwestern Arkansas: Oklahoma Geological Survey Guidebook 18.

Zachry, D.L., and Harris, J.P., 1991, Morrowan (Pennsylvanian) reservoir units, southern Boston Mountains of western Arkansas: Midcontinent Section, Society for Sedimentary Geology (SEPM) and Department of Geology, University of Arkansas publication.

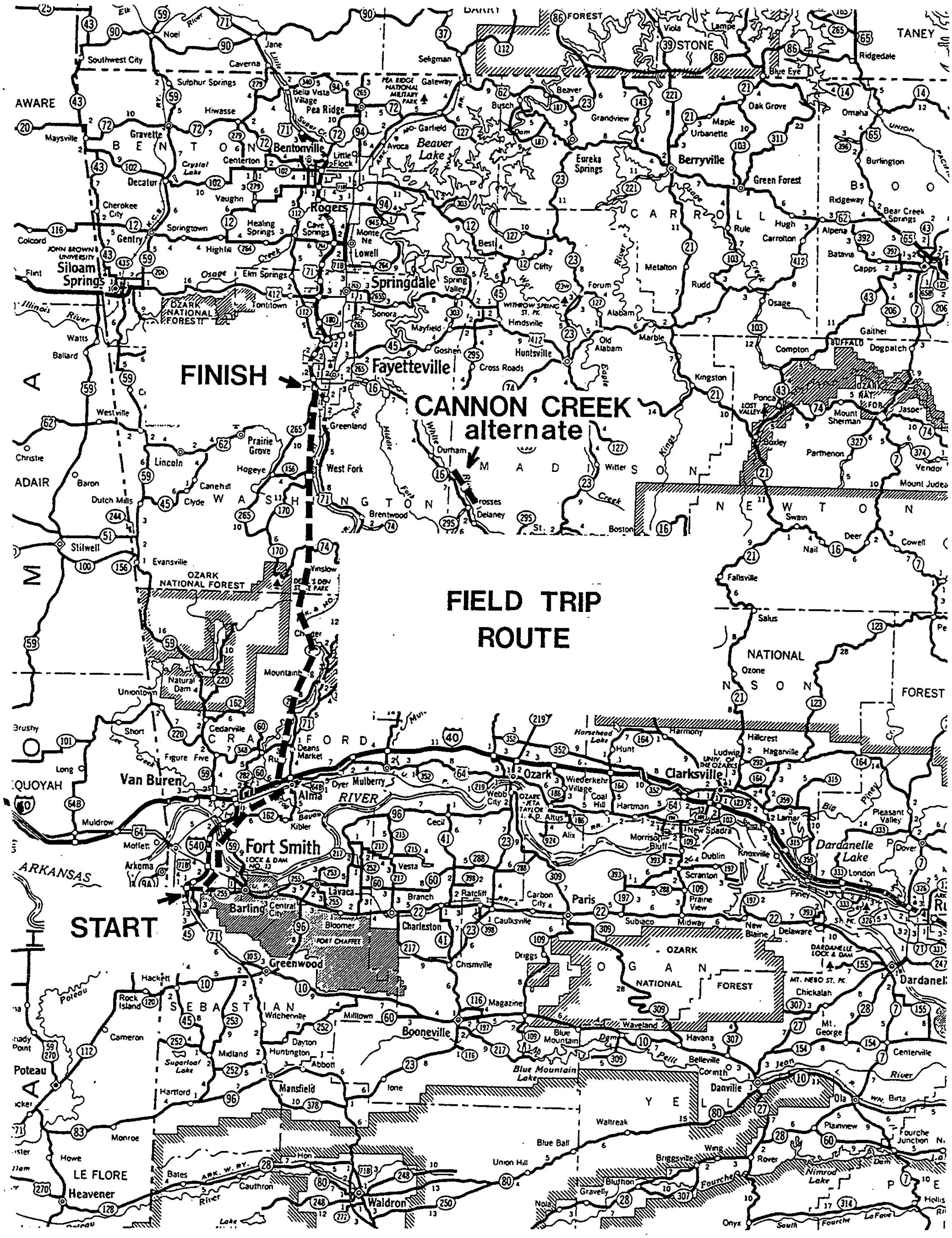


Figure 4. -- *Index map showing field trip route.*

ROAD LOG ITINERARY IN THE BOSTON MOUNTAINS AND ARKANSAS VALLEY NORTHWEST ARKANSAS

By

David W. Lumbert[1] and Charles G. Stone[2]

Depart from the Fayetteville Hilton and proceed south to our first stop in Fort Smith, Arkansas via U.S. Hwys. 71 B, 71, I-40, and I-540 (Figure 4). Enjoy the scenery of the Arkansas Valley and Boston Mountains for about the next 1 1/2 hours. A brief description along the route to STOP 1 follows.

GENERALIZED ROAD LOG FROM FAYETTEVILLE TO FORT SMITH

Cumulative *Mileage*	
0.0	Hilton at Fayetteville, Arkansas. Proceed south on U.S. Hwy. 71B. For approximately the next ten miles you are riding over Fayetteville Shale.
2.8	Junction with U.S. Hwy. 71 and proceed south.
3.6	City limits of Greenland, Arkansas. A former Governor of Arkansas once received a traffic ticket for running a red light here.
8.8	A.H.T.D. maintenance headquarters to right on the north side of West Fork, Arkansas.
9.8	Juntion of State Hwy. 170 at West Fork, Arkansas. Proceed south on U.S. Hwy. 71.
11.0	Abandoned quarry in Pitkin Limestone on left. View of West Fork of White River on right.
11.8	View to west of construction on the new relocation of U.S. Hwy. 71. This will be Stop 4 of our field trip later this afternoon.
12.0	Contact on left of Cane Hill and Prairie Grove Members of the Hale Formation.
12.8	Brentwood Limestone and Woolsey Members of the Bloyd Shale on left.
16.3	Rest area on right; continue south on U.S. Hwy. 71.
21.3	City limits of Winslow, Arkansas. We travel across shales and sandstones of the Atoka Formation to Alma, Arkansas.
24.8	Leave Washington and enter Crawford County.
25.6	Mount Gaylor, Elevation 2,090' above m.s.l.

[1]Geologist, Arkansas Highway and Transporation Department, P. O. Box 2261, Little Rock, Arkansas 72203

[2]Geological Supervisor, Arkansas Geological Commission, 3815 West Roosevelt Road, Little Rock, Arkansas 72204

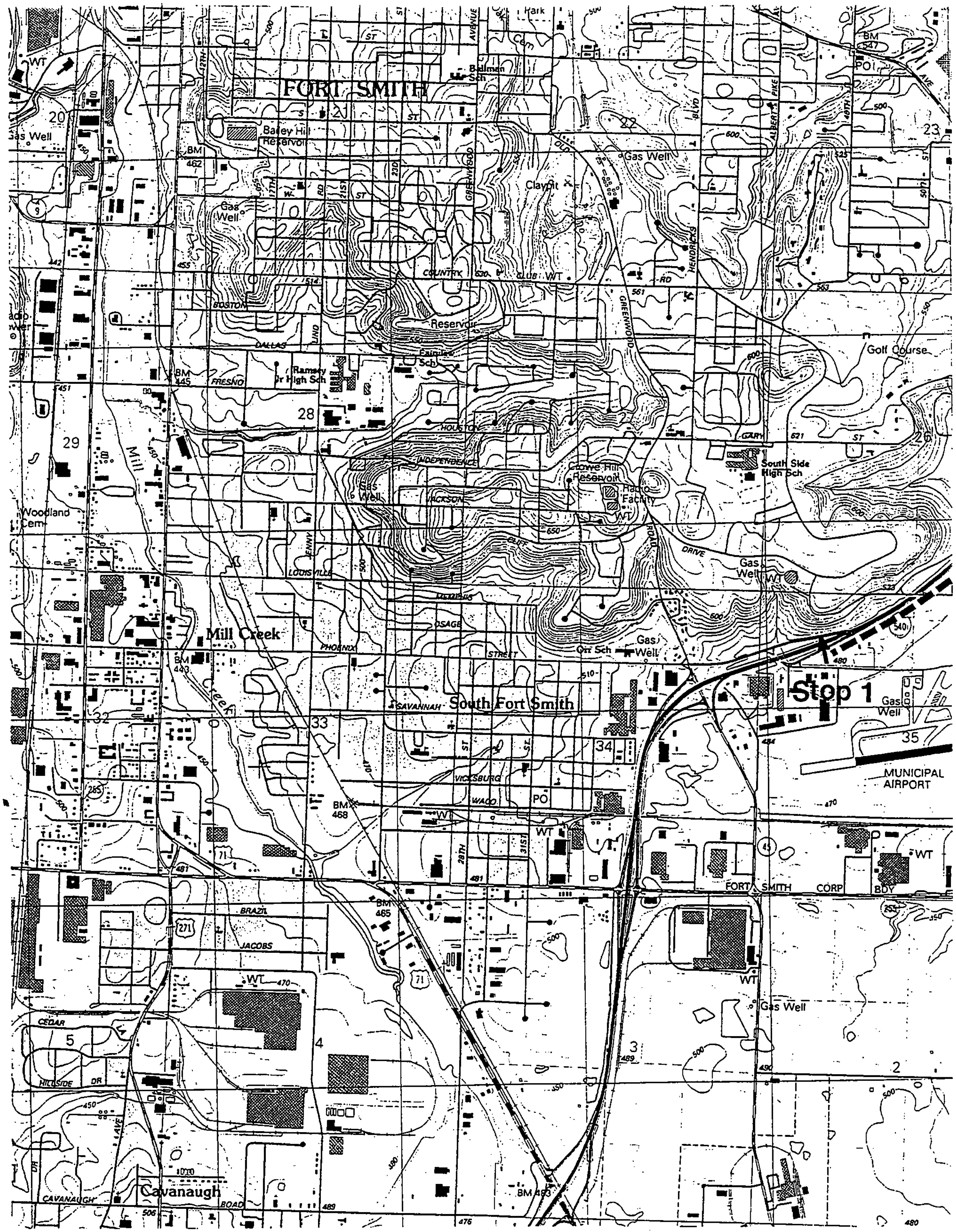


Figure 5. -- *Map showing location of Gabion wall complexes in Fort Smith.*

28.3 Artist Point rest area on right; later this will be our LUNCH STOP.

33.6 Junction of State Hwy. 282 to Chester, Arkansas. Proceed south on U.S. Hwy. 71.

36.4 Various schools on right at Mountainburg, Arkansas. Note the dinosaurs in the playground.

40.3 A small remaining section of the "original" U.S. Hwy. 71 on left.

46.1 City limits of Alma, Arkansas.

47.2 Junction of U.S. Hwy. 71 with I-40. Proceed right (west) on I-40.

48.0 Interchange for the newly relocated U.S. Hwy. 71. Continue on I-40 to southern Fort Smith.

52.7 Take exit from I-40 and proceed south on I-540.

55.9 Bridge over Arkansas River. Leave Crawford and enter Sebastian County.

60.8 Rogers Avenue and State Hwy. 22 exit. Continue south on I-540.

62.8 Take exit 10 on I-540. Stop on left side of the exit! This is the first stop of the field trip.

ROAD LOG FROM FORT SMITH TO FAYETTEVILLE, ARKANSAS

Cumulative
Mileage

0.0 **STOP 1. GABION WALL COMPLEXES IN FORT SMITH**

Gabion walls were constructed at two sites along a 0.4 miles stretch on the west side of I-540 and exit 10 (Figure 5). We plan to examine the larger, more westerly gabion wall across from the parking area at Exit 10 (Figure 6 A and B). The soils, thin colluvium, and the weathered to unaltered McAlester Shale failed here primarily because the cut slope was very steep, some weathered shales previously had slipped and rotated downslope into the cut, and the unaltered gray-black shales (while dipping slightly to the northwest) contain joints that are inclined steeply into the cut.

The Gabion walls are constructed of interconnected rectangular 3-foot wire-mesh baskets filled with coarse nondegradable sandstone. The wire for the gabion baskets is 11 gauge with a tensile strength in the range of 60,000 to 80,000 psi. The lacing wire meets the same specifications and is 13 1/2 gauge. Openings of the mesh are approximately 4 1/2 by 3 1/4 inches. These gabions are of single-unit construction with the base, ends, and sides connected to the base section of the gabion in such a manner that strength and flexibility at the point of connection are at least equal to that of the mesh.

0.2 Proceed east on Airport Road beneath I-540 and turn left (north) on I-540. Numerous natural gas wells produce in the area along the north flank of the Massard Prairie anticline.

0.6 View of smaller Gabion wall at overpass to left.

2.0 Exit 8A to Rogers Avenue and State Hwy. 22. Proceed on I-540.

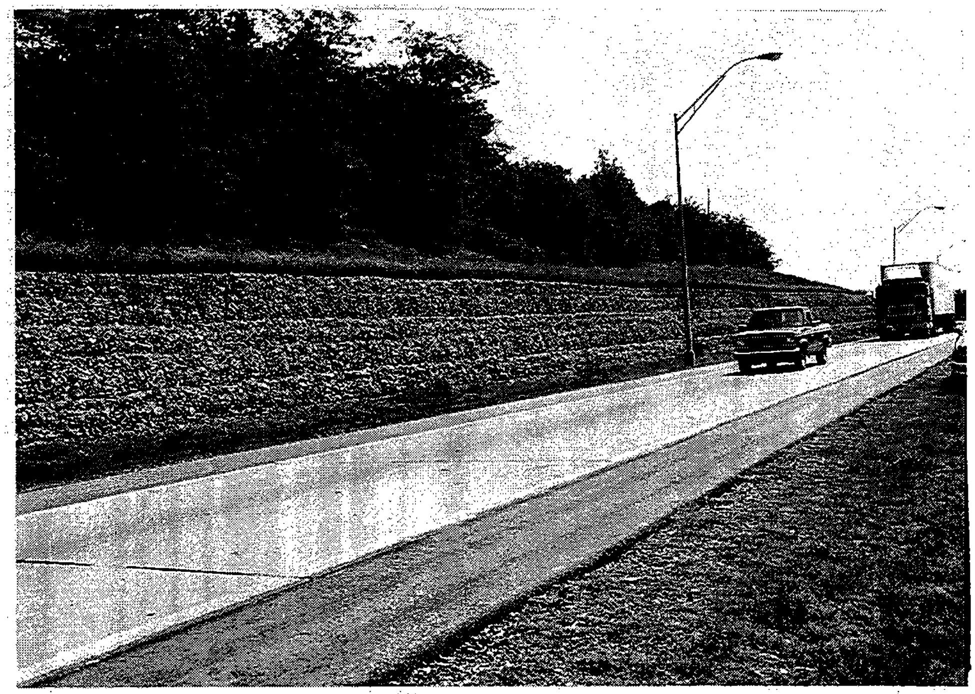


Figure 6 A. -- *Gabion wall at exit 10 on I-540 in Fort Smith.*

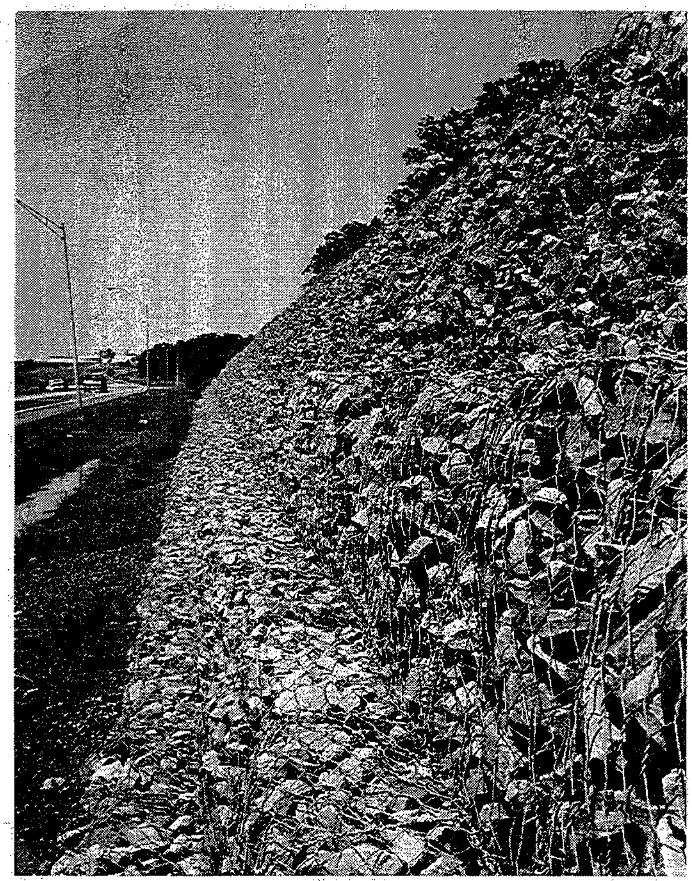


Figure 6 B. -- *Closeup showing details of individual rectangular 3-foot meshed wire baskets filled with stacks of rock forming the Gabion wall.*

Figure 6. -- *STOP 1.*

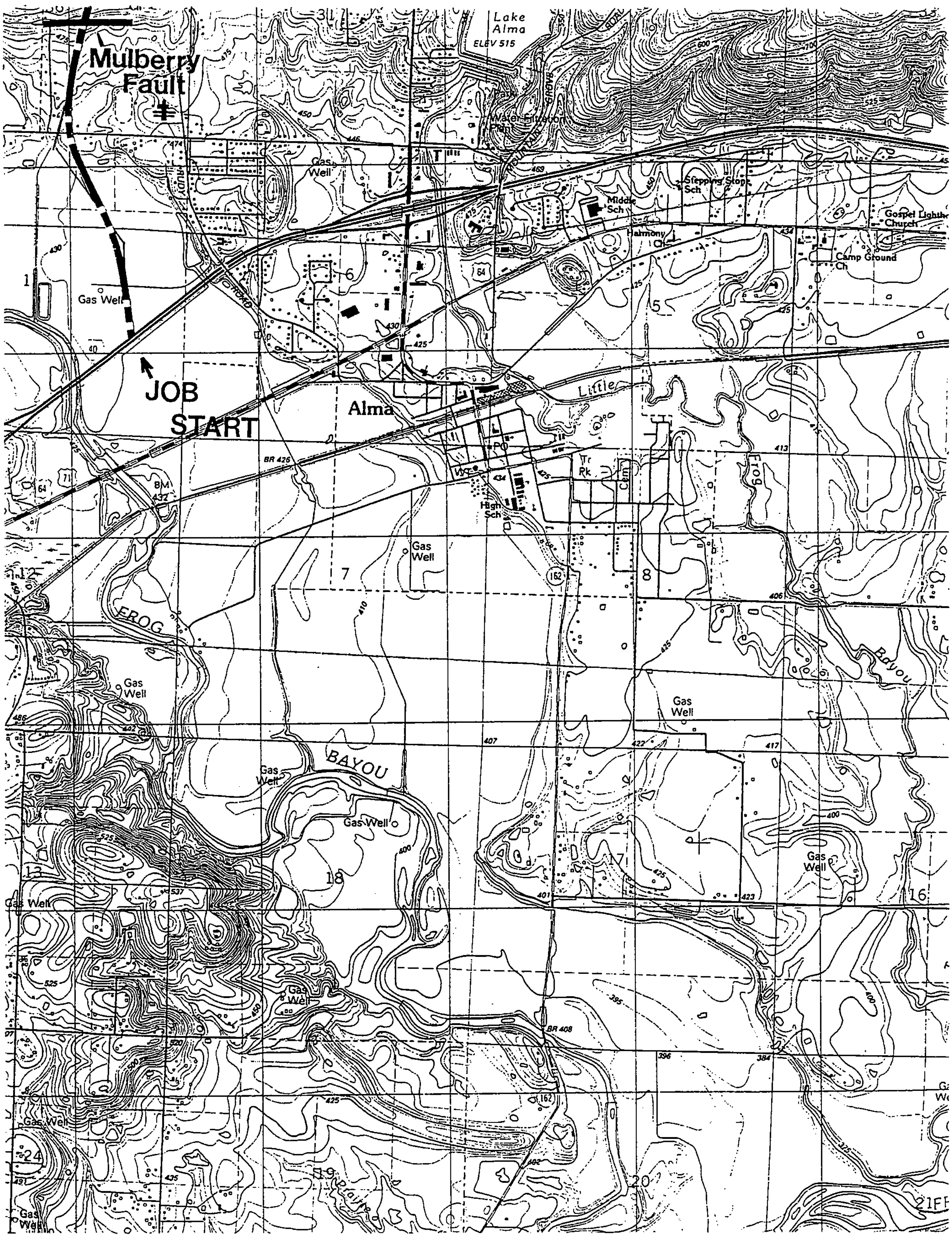


Figure 7. -- *Map showing relocation route for U.S. Highway 71 near Alma, Arkansas.*

6.9 South end of Arkansas River bridge. Leave Sebastian and enter Crawford County. Extensive alluvial flood plains have been developed along the Arkansas River and along the lower courses of many of its tributaries in the area. In times of flood, all these alluvial plains are submerged except where protected by levees. The visible alluvium is mostly silt, but sand and gravel usually occur at depth. Extensive terrace deposits of gravel, sand, silt, and clay are present mostly about 50-60 feet above the Arkansas River. Normal faults have partially determined the present course of the Arkansas River for considerable distances and also the courses of several smaller streams.

7.6 Exit 3 and State Hwy. 59 to Van Buren, Arkansas. Continue on I-540.

9.6 Exit 2B to U.S. Hwy. 64 to Van Buren, Arkansas. Continue on I-540.

10.2 Proceed on exit to right and continue east on I-40.

11.9 A.H.T.D. weight station on I-40.

12.6 Preston quarries in Hartshorne Sandstone to left. Aggregate and riprap used for A.H.T.D. are processed here by Arkhola Quarries, Inc.

14.9 Take exit 13 on the west side of Alma, Arkansas and proceed north on the newly relocated U.S. Hwy. 71 (Figure 7). The Alma gas field is located on the south edge of town.

16.1 Overpass for Collum Lane road.

16.6 Overpass for Maple Shade road. The relocated Hwy. 71 roadbed begins ascent up dipslope of highly jointed sandstones of the middle Atoka Formation. We have passed across the Mulberry fault, just south of this exposure, although it is not exposed. It is a normal fault that is downthrown to the south with some 2,000 to 2,500 feet of displacement. It approximates the break between the Boston Mountains and the Arkansas Valley (Figure 3).

17.1 Road underpass.

17.5 This is a fine exposure of alternating shales, siltstones, and sandstones of the middle Atoka Formation. Several southward-prograding deltaic sequences are present.

18.4 Road overpass.

19.2 On right, small quarry site for aggregate used on this road.

19.8 Overpass for State Hwy. 282.

20.0 Cuesta and dipslope to south formed by massive weathered sandstones of the middle Atoka Formation.

20.6 Slightly older southward-dipping black shales and brown sandstones of the middle Atoka Formation.

21.7 **STOP 2. LOWER ATOKA FORMATION AND LANDSLIDE.** Begin at Gregory Chapel overpass and proceed north 0.2 miles on relocated U.S. Hwy. 71.

This is a good opportunity to examine the highly variable rock types, small faults, and numerous joints that typify many of formations encountered in road excavations throughout this area. These alternating massive to thin beds of brown sandstone, very thin beds to laminae of light gray siltstone,

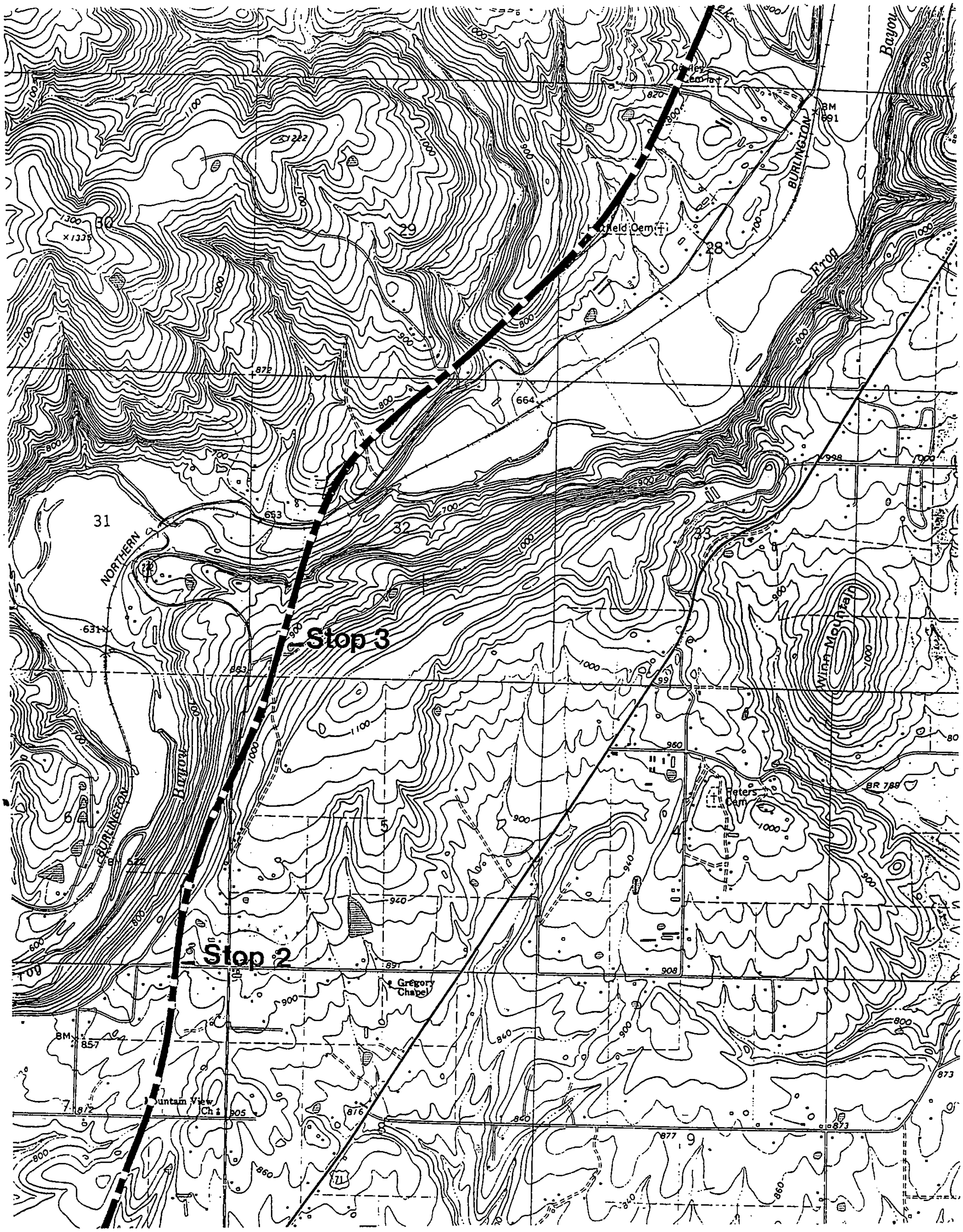


Figure 8. -- *Map showing STOPS 2 and 3 on U.S. Hwy. 71 relocation route at Frog Bayou and vicinity.*

and fissile to clayey gray-black shale represent several cycles of southward prograding deltaic deposits in the lower Atoka Formation (Figure 9A). A thin coaly seam occurs in the shaly clay interval at the top of a channel sequence. Ripple marks, cross-bedding, load features, scour channels, shale and sandstone clasts, and coalified plant fragments are abundant in some sandstones. Bioturbations (worm trails, burrows, etc.) and some invertebrate fossils are present in a few of the siltstones and silty shales. Small amounts of pyrite and its white oxidation product (iron sulfate) and white to yellow sulphur are present in the coaly interval and in some channel sandstones.

Several small normal faults, usually downthrown to the south, offset the gentle southward dipping strata and are marked by an increase in the density of joints. Small slickensides and some brecciated rock often occur in the fault zones. The small faults are likely splays from the larger fault(s) immediately to the north at STOP 3.

At this road cut, 89,000 cubic yards of material were removed. Some relatively small landslides have formed along the northeast side of the excavation, mostly in the soils and partially weathered shales (Figure 9B). They are presently being evaluated for removal and/or remedial procedures. Most of the landslides occur in the same stratigraphic interval that has caused significant landslide accumulations at STOP 3 – some 0.7 miles to the northeast. The problem is mostly related to the high degree of the slopes and the high surficial water content of the weathered shales. Thin bentonite-rich shales are reported in the lower Atoka and upper Bloyd and they probably occur in these strata. Especially upon weathering, these intervals could further aid and abet the local instability of the bedrock and soils.

22.4 **STOP 3. FROG BAYOU BRIDGE AND LANDSLIDES.** Begin at overpass for State Hwy. 282 and proceed on foot some 0.6 miles to south edge of Frog Bayou bridge (Figure 8).

Here we will discuss problems of landsliding in sandstone-shale complexes in the lower Atoka Formation, and some possible solutions. The engineering properties of these interlayered sedimentary rocks and their residual soils can be complex. The materials are anisotropic, reflecting their original structure and also subsequent changes in the local character, including the man-made changes caused by construction. Most landslides in the Pennsylvanian sandstones and shales are directly related to the original rock structure and their total environment.

A series of rather small northeast-trending normal faults, mostly downthrown to the north, cut across the area between the slide and Frog Bayou Bridge. The lithology on the downthrown side of the fault plane complex is for the most part wet, notably unstable colluvium. In the process of excavation within the area between the slide and the bridge, several large boulders were removed. These boulders are further indication of the presence of the faults. A large downthrown-to-the-south normal fault also occurs near the center of Frog Bayou Bridge thus the area between the two fault zones is a graben (Boyd R. Haley, personal commun., 1992).

Initially, the roadway was excavated with nearly vertical slopes, removing over 91,000 cubic yards of material. During the excavation period, cracks were discovered in the colluvium near the top of the slope. At this time, the contractor began excavating the colluvium on a gentler 3:1 slope to prevent possible failure. However, after a heavy local rainfall, large amounts of surface water runoff and increased subsurface seepage caused two slope failures to occur in the immediate area (Figure 10 A).

In reconstructing the failed slope, all soft unstable material was excavated on a 2:1 slope from near the existing right of way to the durable shale (Figure 11). A rock buttress was constructed on the shale with the toe 120 feet right of centerline. The maximum height is 25 feet as measured from the toe. Constructed on a 4:1 slope, the backfill material consists of a soil and rock mixture and/or Type II material chiefly composed of shale with a Slake Durability Index (SDI) of 50-95, or any other rock-like

Figure 9 A. -- *Small fault separating massive channel sandstone with abundant joint sets on left from thin siltstones and shales on right in the lower Atoka Formation. The downthrown block is on the left (south).*

Figure 9 B. -- *Minor landslides with a series of crown scarps and small pressure ridges in soils and weathered shales at the northeast part of STOP 2 roadcut.*

Figure 9 -- *STOP 2*

Figure 10 A. -- *A small recent continuation of a formerly larger landslide at the south part of STOP 3.*

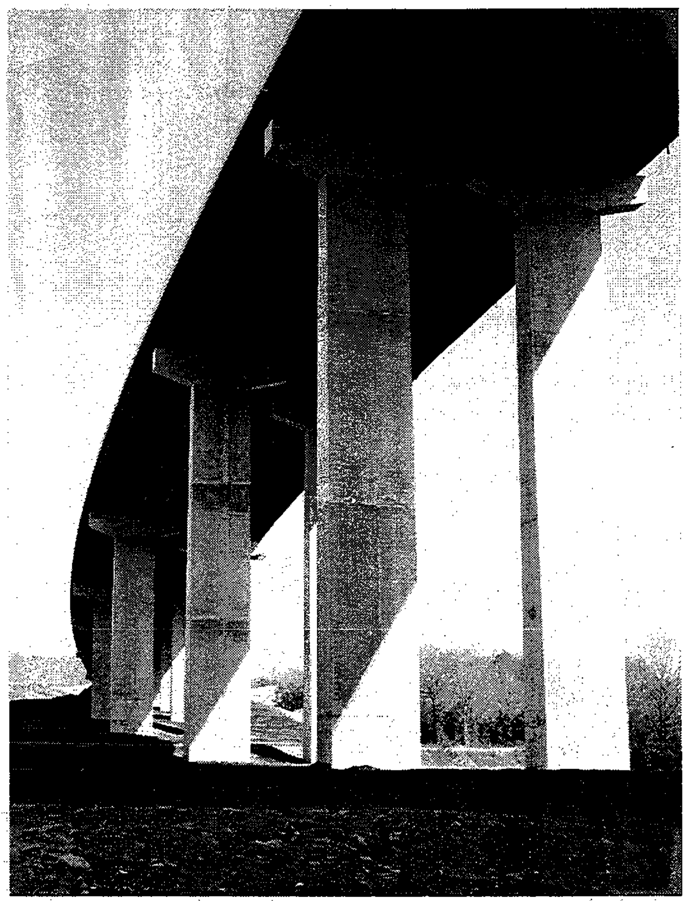


Figure 10 B. -- *View of columns and span on 198-foot high and 2287-foot long Frog Bayou bridge.*

Figure 10 -- *STOP 3*

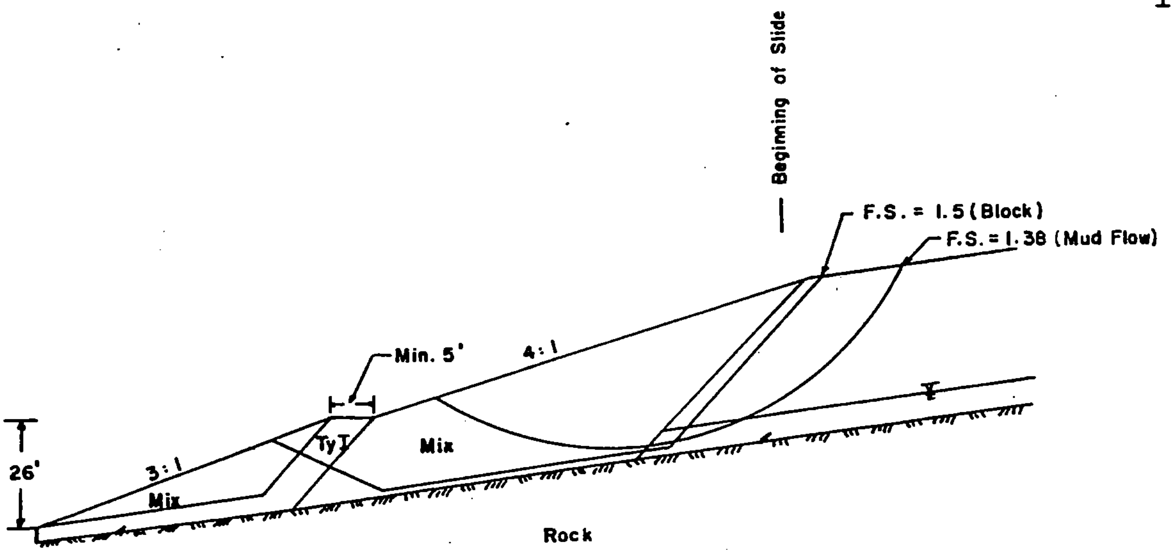


Figure 11. -- *This typical section from the construction plans illustrates the design criteria taken to correct the Slope Failure at STOP 3.*

material. The material used for the rock buttress is composed of Type I granular materials with boulders no greater than 18" or shale with a SDI greater than 95.

This particular slide repair also involved constructing a special 3' x 2' rock drain composed of Class B concrete aggregate and a filter fabric in a ditch on the slope to intercept water and direct it to the roadway ditch.

In addition to discussion of the slides, there will be a summary of the construction at the Frog Bayou Bridge (Figure 10 B). The Frog Bayou Bridge is one of the highest in this part of the country. Its highest point, measured from the bridge deck to the bottom of the pier, is 198 feet. The bridge is 2287 feet in length. Proceed north across the bridge.

23.6 Continue beneath Frog Bayou bridge and turn left (northeast) on State Hwy. 282 to Mountainburg, Arkansas.

24.5 Small concrete bridge on State Hwy. 282.

25.9 Small concrete bridge with minor retaining wall on State Hwy. 282.

26.7 **OPTIONAL SCENIC STOP. SILVER BRIDGE ON STATE HWY. 282 OVER FROG BAYOU.**

There are several normal faults dissecting the lower Atoka and they are indicated by the lithologic "pinchouts" of massive sandstones in the overlying bluff. The stream alluvium is composed mostly of sandstone pebbles and cobbles and locally exceeds 10 feet in thickness.

27.6 Crossing Missouri-Pacific Railroad tracks.

27.9 Town of Mountainburg, Arkansas. Proceed left (north) on U.S. Hwy 71.

28.1 Downtown Mountainburg, Arkansas.

29.7 Lake Fort Smith State Park to right.

30.0 Bridge over Frog Bayou.

31.8 Several small normal faults exposed in lower Atoka Formation on right.

36.8 **LUNCH STOP. ARTIST POINT REST AREA** (on west side of Hwy. 71). This is an opportunity to enjoy views of the rugged scenery of the Boston Mountains, with Lake Shepherd Springs in the valley to the east (Figure 12 A). Ozark handicrafts and other items are available at the nearby shops.

There is a general accordance of the Boston Mountain summits with occasional monadnocks such as White Rock Mountain in the distance to the east. To most early workers these summits represented the oldest "peneplain" surface developed in the Ozark region. Later investigations have added some new concepts about the surface, notably that it was formed by semi-arid pediplain erosion during a late Tertiary or early Pleistocene interglacial cycle. Mr. Jake Clements, a member of A.H.T.D., did graduate work on the origin of these high surfaces and has kindly consented to discuss them during the lunch stop.

39.2 Massive thick channel sandstones in the lower Atoka Formation are exposed in the roadcuts to left.

39.5 Mount Gaylor tower and shops. The elevation is 2,090 feet.

40.5 Leave Crawford County and enter Washington County.

43.1 Entering Winslow, Arkansas.

44.9 Bridge on upper West Fork of White River.

47.5 Junction of State Hwy. 74 (east); continue north on U.S. Hwy. 71.

48.2 Entering Brentwood, Arkansas. This is the type section of the Brentwood Limestone Member of the Bloyd Shale.

49.0 Rest Area on left.

52.5 Turn left on Washington County road 35 at Woolsey, Arkansas. The type section of the Woolsey Member of the Bloyd Shale (Lower Pennsylvanian) occurs in exposures adjoining this site.

52.6 Woolsey bridge over West Fork of White River.

52.8 Crossing Missouri-Pacific Railroad tracks.

53.0 Juntion of Washington County roads 35 and 228, at Pitkin Corner. Proceed right (north). The Pitkin Limestone (Upper Mississippian) was named for the old post office at this site.

53.8 Entering West Fork, Arkansas.

54.7 Turn left on dirt road at Karnes Cemetary sign.

55.2 Junction with several work roads used in construction on relocated U.S. Hwy. 71. Proceed left (south).

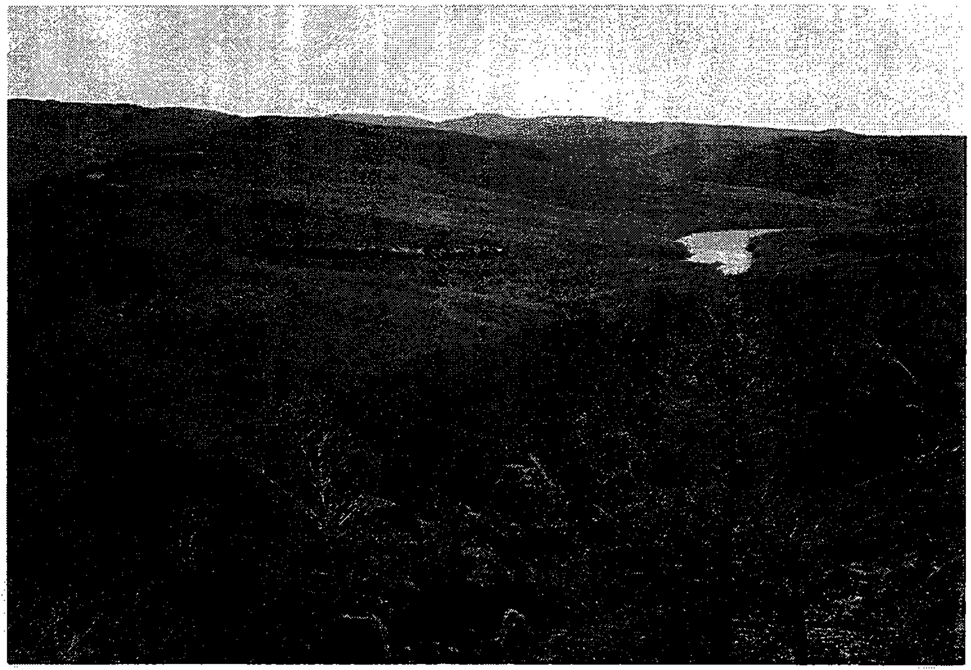


Figure 12 A. -- *Scenic view of the Boston Mountains from Artist Point rest area with Lake Shepherd Springs and large bluff formed by massive sandstones of the lower Atoka Formation in the valley below.*

Figure 12 B. -- *Excavation, bridge, and buttress walls south of West Fork, Arkansas in lower Atoka and Bloyd strata at STOP 4.*

Figure 12 - *LUNCH STOP and STOP 4*

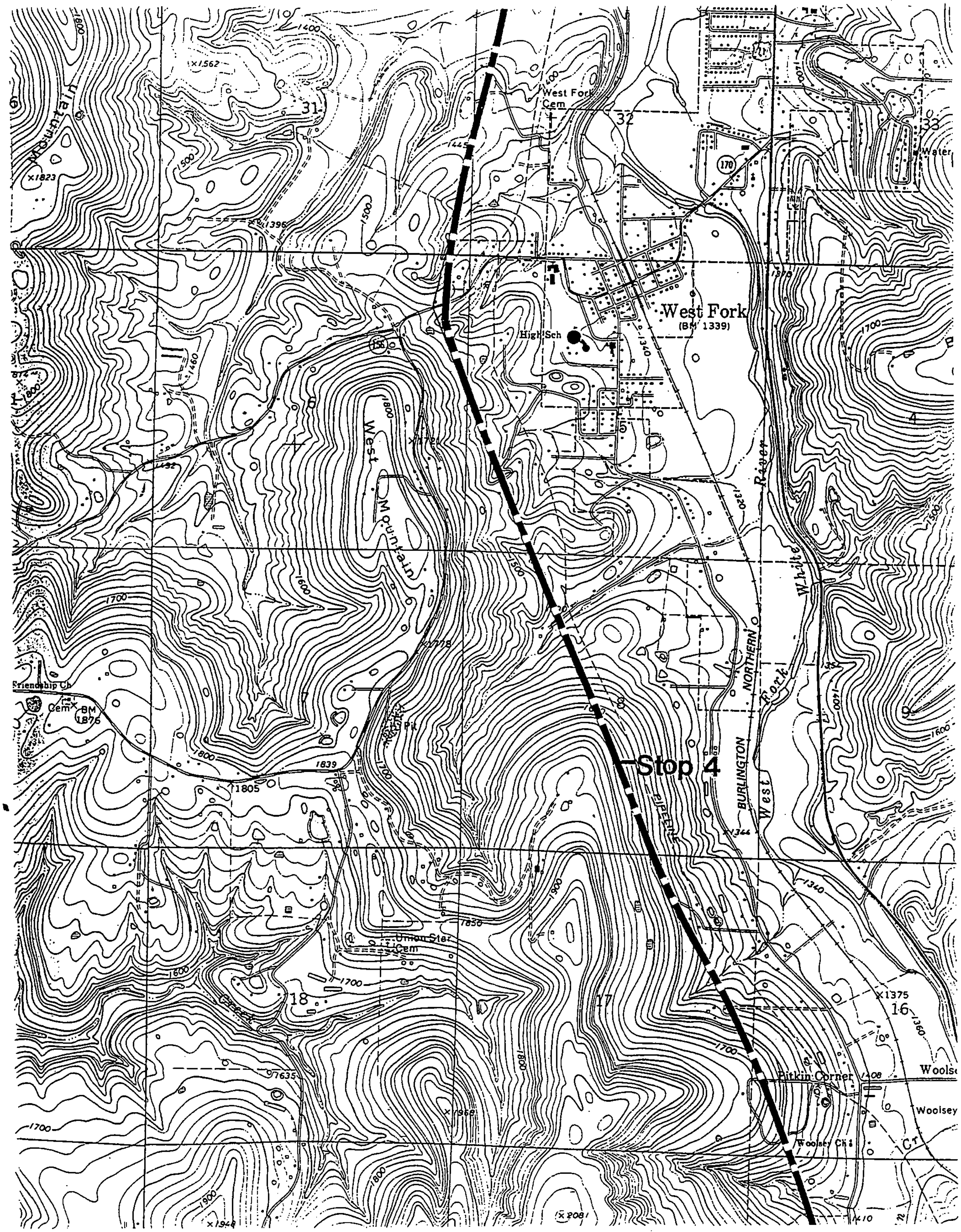


Figure 13. -- *Map showing STOP 4 on relocation route for U.S. Highway 71 in southern West Fork, Arkansas.*

55.5 STOP 4. SOUTHERN WEST FORK EXCAVATIONS, BRIDGE, AND BUTTRESS WALLS

At this stop there will be an opportunity to discuss and evaluate the numerous geotechnical problems of extensive side hill excavations, bridge construction across a narrow valley, and buttress walls over 70 feet high that are undercut as much as 20 feet to achieve foundation stability (Figures 12B,

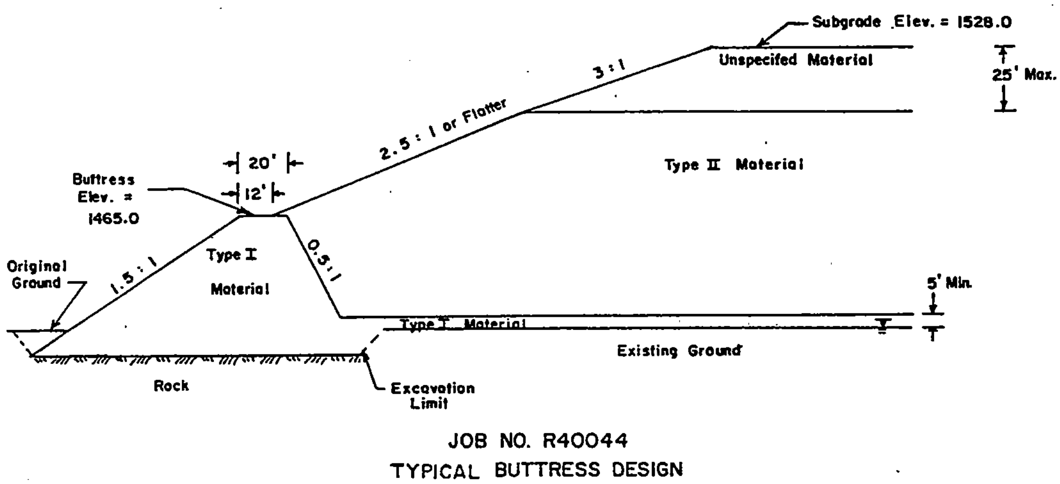


Figure 14. -- *Typical section from the construction plans illustrating the design criteria adapted for the buttress at STOP 4.*

13, and 14). The problems were accentuated by the structural unpredictability of the interbedded Atoka and Bloyd sandstones, shales, and minor limestones and conglomerates. Construction projects in several adjoining states with similar geotechnical conditions have experienced costly failures caused by long-term degradation of shale and interlayered strata in embankments. These include large slope failures and, as here, the potential of detrimental foundation settlement and creep.

Owing to the preponderance of shales along the Highway 71 relocation project, the Slake Durability Index (SDI) test procedure was adapted and used by A.H.T.D. whenever applicable. Based on discussions with highway engineers in other states, and the Federal Highway Administration, and based on our geotechnical investigation, soil and rock parameters were assigned to three types of embankment materials to be used in this project for design purposes: Unspecified, Type II and Type I. The parameters are as follows:

Unspecified Material

c = 500 psf ϕ = 70 degrees δ = 120 pcf	Includes soil and soil-like shale w/SDI 0-50

Type II Material

c = 200 psf ϕ = 26 degrees δ = 130 pcf	Includes intermediate shale with SDI 50-94, and/or non-durable sandstone and weathered sandstone

Type I Material

c = psf
ϕ = 35 degrees
δ= 135 pfc

Includes rock-like shale with SDI 95+, and /or limestone & durable sandstone

The material specifications were written primarily for thick sequences of shale and did not address sequences composed primarily of sandstone; however, it did address interbedded sandstones with interbedded shale. A solution to the materials problem for the stability of this particular buttress, and to maintain the above design parameters, was to allow Type II materials to contain mostly hard and durable rock and soil-like degradable material.

Due to a shortage of material, one alternative discussed for this job was to redefine the SDI requirements for various specified materials. Obviously, if changes are made in these specifications, design material parameters and embankment designs would require re-evaluation to verify their stability.

Within the immediate area, over 3,336,000 cubic yards of material was removed to meet the specific grade design.

This is also a fine opportunity to examine thick-to thin-bedded alternating rock types along a long hill and valley excavation and the minor effects of a normal fault.

There are many exposures of upper Bloyd and lower Atoka strata at the excavations south of the buttress and bridge. A calcareous conglomerate contains some goniatites and other invertebrate fossils of probable late Morrowan age. These strata contain bioturbations (worm trails, burrows, etc.), ripple marks, cross-bedding and other features indicative of shallow marine and deltaic deposition.

Recently a "small" landslide took place 0.6 miles south of this site during excavation in the same general lithologic sequence. Again it is uncertain whether this landslide is completely due to the slope angle, wet weather conditions, contrasts in rock types or combinations thereof, or, in part, to abnormally high plasticities of some thin bentonite-rich shales.

55.8 Return north to work road junctions and continue east to Washington County roads 35 and 228.

56.3 Turn left on Washington County roads 35 and 228.

57.6 Junction of McKnight and Main Streets in West Fork, Arkansas. Proceed right (east). The shallow West Fork natural gas field is located in this area.

57.7 Turn left on Campbell Street and Washington County road 63.

58.4 Large blocks of slumped Pitkin Limestone on left. The West Fork of White River on right.

58.9 McClinton-Anchor quarry in Pitkin Limestone to (west) and view of relocated U.S. Hwy. 71 which cuts across the former quarry. Crushed rock and riprap is used on local construction projects, as well as on portions of the relocated U.S. Hwy. 71. A monadnock of Pitkin Limestone in the east end of the former quarry now lies east of relocated U.S. Hwy. 71 (Figure 15A). It is underlain by the Fayetteville Shale.

60.5 Junction of Washington County roads 63 and 65. Proceed left on County road 65.

Figure 15 A. -- *A monadnock of Pitkin Limestone underlain by Fayetteville Shale on the east side of relocated U.S. Hwy. 71 and formerly a part of the McClinton-Anchor quarry at mileage 58.9.*

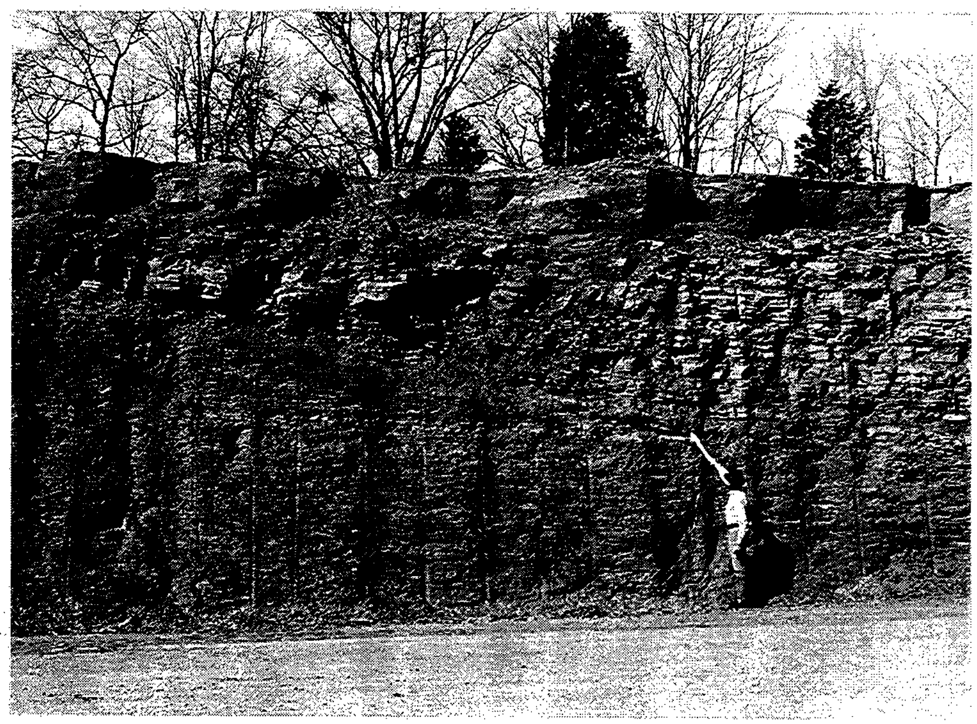


Figure 15 B. -- *Thin-bedded sandstones and shales of the Cane Hill Member overlain by a channel sequence of massive to thin sandstones of the Prairie Grove Member of the Hale Formation at mileage 64.3.*

Figure 15. -- *Mileages 58.9 and 64.3.*

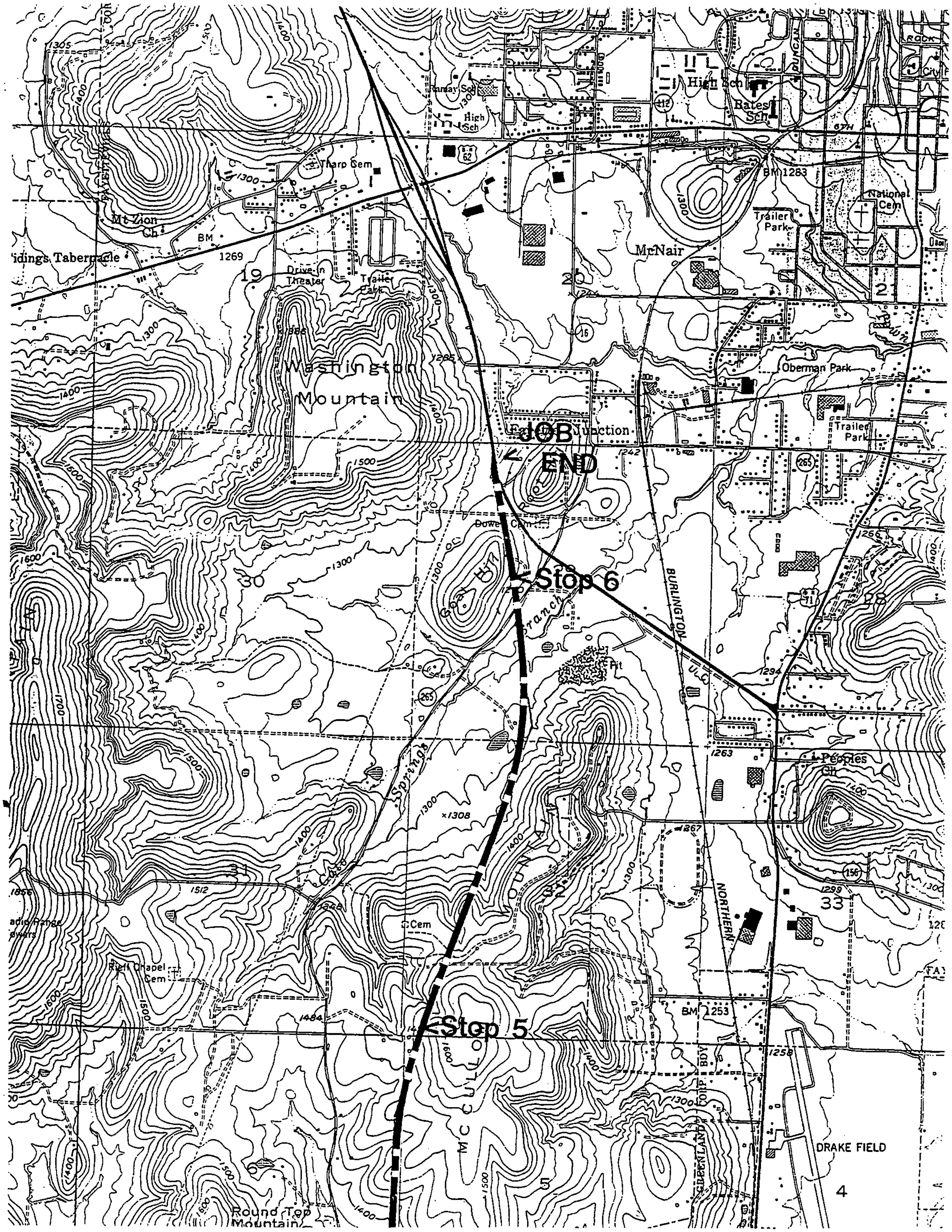


Figure 16. -- *Map showing Stops 5 and 6 on relocation route for U.S. Highway 71 in south Fayetteville, Arkansas.*

60.7 Overpass of Washington County road 65 and relocated U.S. Hwy. 71. Proceed on County road 65. The exposed massive brown sandstones are in the Hale Formation of Early Pennsylvanian age.

61.5 Pavement begins!

62.4 Washington County road 65 with temporary crossover on relocated U.S. Hwy. 71.

63.6 Junction of Washington County road from Greenland, Arkansas. Proceed west partially across relocated U.S. Hwy. 71.

63.7 Turn north on access lane for relocated U.S. Hwy. 71. Minor exposures of the Fayetteville Shale are present here.

64.3 Exposures of gentle south-dipping thin-bedded, brown sandstones and gray shales (Cane Hill Member) overlain by thin to massive brown, channel sandstones (Prairie Grove Member) of the Hale Formation (Figure 15 B).

64.9 **STOP 5. BRENTWOOD LIMESTONE MEMBER -- WESTERN McCOLLUM MOUNTAIN.**

The Brentwood Limestone Member of the Bloyd Shale is exposed in this large excavation (Figures 16 and 17A). The Brentwood consists of several limestone beds seperated by shales. The lowermost limestone with minor shale layers varies from about 3-15 feet in thickness. It is usually rather massive, bluish gray in color, highly cross-bedded, and very fossiliferous. The fossils include brachiopods, corals, bryozoans, crinoids, gastropods, and blastoids. Above this unit is a dark fissile clay shale that ranges from 5 to 12 feet in thickness. Another highly lenticular limestone of the upper part of the Brentwood Member overlies this unit and varies from 2 to over 12 feet in thickness. The total thickness of the Brentwood Member at this site is about 40 feet. About 25 feet of black to brownish gray clay shale with some thin brown silty sandstone beds of the Woolsey Member of the Bloyd Shale overlies the Brentwood Member (Doy L. Zachry, personal commun., 1992). The rocks dip gently to the south and are in the upthrown southeast block of the Price Mountain (Fayetteville) fault system about 0.8 miles to the north at STOP 6.

This locality provides an excellent example of the lateral and vertical variability of the Brentwood Member. Rapid thinning and thickening of the various lithologic types occur in the short distance of less than 1/4 mile. The interval is extensively channeled, and limestone units are often truncated by conglomerate, siltstone or shale.

To meet design requirements, some 1,022,000 cubic yards of material were removed in the excavations at STOP 5.

65.2 Proceed east over Washington County road at temporary "crossover", then north on right lane of relocated U.S. Hwy. 71. There are numerous exposures of brown sandstones and gray shales of the Hale Formation. At the north end of the roadcut, the Hale rests unconformably upon the Fayetteville Shale, the Pitkin Limestone being locally absent by erosion. Round to subangular cobbles and pebbles, mostly of Pitkin Limestone, are present in a reddish brown 4-foot-thick conglomeratic sandstone at the base of the Hale. Plant fossils are notably present in this interval.

65:5 Junction with State Hwy 265 to Cato Springs. Proceed across this road to north on relocated U.S. Hwy. 71 to the juntion with U.S. Hwy. 71 in south Fayetteville, Arkansas.

65.7 **STOP 6. FAYETTEVILLE SHALE AND PRICE MOUNTAIN GRABEN.**

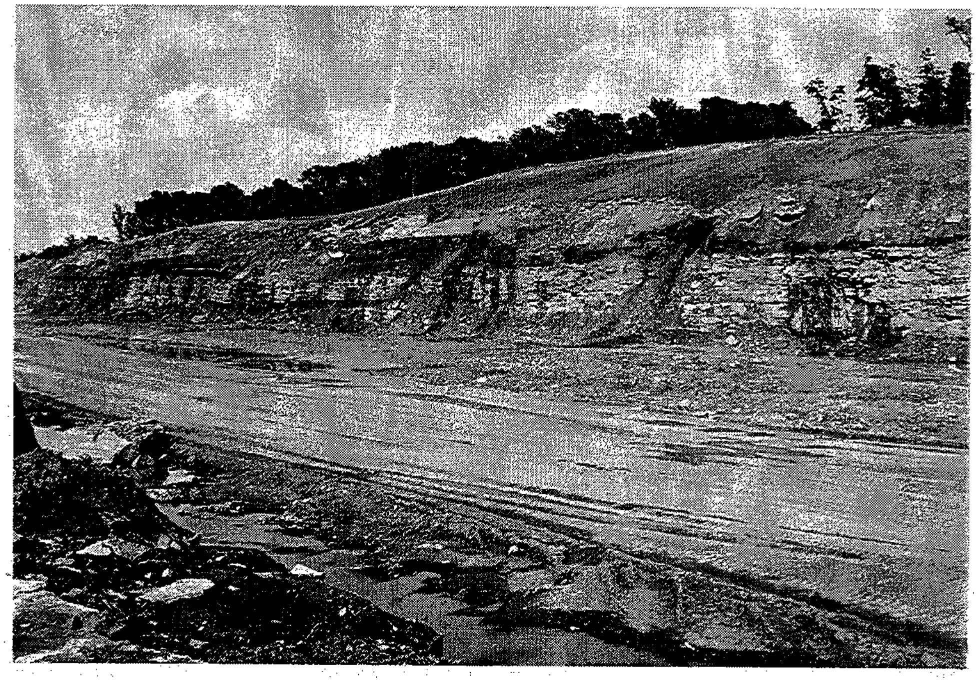


Figure 17 A. -- *Interbedded light gray fossiliferous limestones and dark shales of the Brentwood Member overlain by dark shales and minor brown sandstones of the Woolsey Member at STOP 5.*

Figure 17 B. -- *White veinlets and encrustations of calcite and halloysite(?) on faulted and sheared surfaces of the steeply dipping Fayetteville Shale at STOP 6.*

Figure 17. -- *STOP 5 and STOP 6.*

A most interesting feature at this Stop (Figure 16) is a large graben (E.E. Glick, personal commun., 1992) formed by the downthrown strata between two large northeast-southwest-trending normal faults that form the Price Mountain (Fayetteville) fault system. The centerline cuts through the easternmost fault and the westernmost fault is about 1500 feet to the west. The bedrock is mostly black, fissile, organic rich shales of the lower Fayetteville Shale (Mississippian), and they occasionally contain invertebrate fossils, some being diagnostic goniatites (Doy L. Zachry, personal commun., 1992). Septarian ironstone concretions occur within the shales and have minerals encased in them, including calcite, gypsum, barite, pyrite, and halloysite; a few are oil saturated. Typically the strata dip from 20-35 degrees to the northwest. Thin sheared intervals with rock gouge and shiny slickenside surfaces are also present in the shale. Veins and encrustations of calcite and halloysite(?), up to 6 inches wide locally occur within this shale (Figure 17 B). Interpretation of the many cores from the borings further indicate numerous secondary faults within the graben. There is little lithologic continuity in the sheared and faulted cores. The two large faults forming this graben probably extend to the Precambrian granitic basement about 1500-1800 feet below.

Notice the absence of landslides and the presence of a hearty crop of recently seeded grass on the steep slope to the west where the Fayetteville Shale and the overlying Wedington Sandstone Member were excavated. Incidentally, some 684,000 cubic yards of material were removed from the entire construction site to meet design requirements.

IF TIME AND CONDITIONS PERMIT THE FIELD TRIP WILL CONTINUE TO THE CANNON CREEK ALTERNATE STOP about 22 miles east-southeast of STOP 6. The Cannon Creek locality is located on State Hwy. 16 about 1/2 mile southeast of the Washington-Madison County line.

ALTERNATE STOP. REINFORCED EMBANKMENT AT CANNON CREEK.

This site affords a brief examination of a 76-foot-high reinforced embankment where State Hwy. 16 adjoins Cannon Creek.

Arkansas State Highway 16 is a two-lane secondary road over the Boston Mountains linking Brashears and Fayetteville. In 1985, a realignment project involved replacing a hairpin curve and one-lane bridge over Cannon Creek, with a straightened roadway section, an embankment with a side slope height of 76 feet, and a four-barrel concrete box culvert (Figure 4).

In 1986, slope failures in the upper Bloyd Shale (Doy L. Zachry, personal commun., 1992) during the early stages of construction of the embankment eventually led to the cancellation of the project. In order to attain a greater safety factor against slope failures, several alternatives were considered. In 1987, construction resumed under a new contract which included the use of geogrid material as the primary embankment reinforcement. The new contract also included instrumentation and monitoring plans to evaluate the performance of the reinforced embankment.

While under construction, monitoring instruments were installed at different locations throughout the downstream side of the embankment. Instrumentation consisted of multipoint extensiometers, inclinometers, strain gages, settlement stakes, tensiometers, pneumatic piezometers, soil matrix potential sensors, and moisture-temperature indicators. Instrumentation has provided useful information on the effectiveness of the reinforcement in terms of embankment stability, cost, and long-term maintenance projections.

The material used for the embankment fill was a highly plastic clay (AASHTO[1] type A-7-6). Attenberg limit tests performed on 9 samples recovered from borings in the borrow areas gave a high average plastic index of 35. The clay was believed to have a high shrink-swell ratio. Grain-size analysis indicated that all samples had at least 70% by weight of material finer than the #200 sieve. The

[1] American Association of State Highway and Transportation Officials

unconfined-undrained triaxial tests on compacted samples of the clay indicated an undrained shear strength of about 900 psf.

The design by the geogrid supplier utilized three primary reinforcement grades, designated as types 1, 2, and 3. They are assumed to have allowable 120-year design strengths in one material direction of 1,000, 2,000 and 3,000 lb/ft. respectively. Specifically, total geogrid strain at the allowable design load was limited to 10% over 120 years in order to limit long-term embankment deformations.

The design specified that continous horizontal layers of intermediate reinforcement consisting of lightweight geogrid extending 4.5 feet into the slope, be placed on 1-foot vertical intervals over the entire slope face. Intermediate reinforcement is believed to be a necessity when dealing with expansive clay soils.

The fill was spread in nominal 8-inch lifts. Sheep-foot rollers were used to compact the clay fill to the specified minimum 95% relative compaction according to the AASHTO-T-99 standards. Specified moisture contents ranged from optimum to 4% dry of optimum for the lower 30 feet of embankment and from +2% to -2% of optimum in the upper embankment section.

The design and stability analysis of the embankment was implemented using a factor of safety of 1.3, foil phi angle of 20 degrees, and soil cohesion of 50 psf.

END OF FIELD TRIP -- PROCEED TO THE FAYETTEVILLE HILTON.

1992 UPDATE: STEEL WIRE ROPE SAFETY NET SYSTEMS APPLICATION TO ROCKFALL MITIGATION IN THE U.S.

Charles N. Yarnell
Sales & Marketing Manager
Brugg Cable Products, Inc.
R.R. 16 Box 197E
Santa Fe, New Mexico 87505
(505) 438-6161 Fax (505) 438-6166

ABSTRACT

Rockfall and debris flow are most effectively controlled by the application of steel wire rope net systems. Nearly a half century ago, Brugg Cable Products, Inc. recognized the advantages of wire rope netting for preventive structures and designed and installed the first wire rope safety net system for prevention of snow avalanches. Field testing of this net indicated that the avalanche prevention net can be alternately used for protection against rockfall and debris flow. The main objective of continued development was to make full use of the flexibility of wire rope netting, it's chief advantage over rigid type systems. Over the years, numerous tests have been performed in the factory and field and a dampening device was developed and added to our rockfall protection systems. Today, there are a wide variety of sophisticated protective netting systems available for a multitude of applications, including rockfall mitigation. Until recently, these systems have been used extensively outside North America with our Company having over 1200 systems installed worldwide. Their introduction into the North American market has been progressing steadily from coast-to-coast. Continued growth is anticipated as familiarity with design parameters and experience with installations increases confidence in their use.

A review of the approval, testing, design considerations, locations and performance of U.S. installations of wire rope safety net systems completed is presented with the hope that the information may contribute to the safety of our transportation system against the dangers of rockfall and other hazards.

INTRODUCTION

In 1991, a paper reviewing the basic application of steel wire rope systems and the various installations in the United States was presented at the annual AEG meeting. Three slope protection systems covering a combined total of almost 65,000 square feet of wire rope safety nets and eight permanent rockfall protection barriers of over 12,650 linear feet were discussed. Yarnell, 1991.

Since 1991, extensive testing has been conducted to improve system performance and installation, as well as, add to the available designs. Two slope protection systems and five rockfall protection barriers have been installed in six different states, all but one having never used similar systems.

SLOPE PROTECTION

The steel wire rope safety net, due to its inherent strength and flexibility, offers the option of draping large unstable rock faces, diverting rock slides, and controlling larger amounts of debris than previously used chain-link or gabion mesh materials.

Of the applications of wire rope safety nets used or planned for slope protection to date, some replace or are used in combination with chain-link or gabion materials. This is because the static, and sometimes dynamic loads, have severely damaged or destroyed these aforementioned materials when used alone.

The applications listed include a project in Washington state for which the nets are presently being manufactured are will be installed this summer.

Applications

Orchard Ridge Associates

Located directly behind a commercial building in Brewster (N.Y.) Business Park, six ten-by-twenty foot wire rope nets, factory-assembled to form one thirty-by-forty foot net, were installed by the developer using large trees for anchorage. The one foot square openings in the net allow smaller rubble and debris to slide, but are controlling the large rock formations from rolling down and damaging the newly constructed building.

Tennessee Department of Transportation - Interstate 40

The second application, completed in the spring of 1992, involved draping 14,400 square feet of combined wire rope netting and gabion wire mesh to a rock chute that was previously draped with gabion wire mesh alone. The nets were 10' x 20' each and woven in a rectangular pattern to form 12" squares. Rock bolts were installed for attachment of a segmented top horizontal and several vertical net support ropes. The gabion wire mesh was installed using hog rings to the wire rope nets on the ground to form rows. A crane then lifted the bundled row to the top support rope and attachment with a seaming wire rope. Subsequent rows were added and seamed to the top and, after being unrolled, to each adjoining row.

The additional strength of the wire rope net will withstand the dynamic forces of small rock rolling from above under a space remaining beneath the top support rope. The segmented top support rope, which replaced a single long span, is more redundant and in combination with the vertical support ropes increases the overall strength.

Washington Department of Transportation

At the time this paper is being written, 12' x 25' wire rope nets (rectangular weave, 8" square pattern) are being manufactured for installation on a rock slope as part of rockfall mitigation measures in the Cabinet Creek area of upper northwest Washington. The area to be covered is over 60,600 square feet with the combination wire rope net and gabion wire mesh. Previously, gabion wire mesh was twice placed on the rock slope only to fail from excessive loading. Extensive planning and ground work are being done to insure smooth helicopter installation of the netting 300 feet up on the almost inaccessible slope.

ROCKFALL PROTECTION BARRIER

Since the introduction of wire rope net rockfall protection barriers over 40 years ago, continual research and development have been conducted to improve the net designs for catching and containing rockfall. Field testing of actual systems followed in 1989. Late in 1991, additional large-scale testing modeled after the CALTRANS tests of 1989 was conducted. The first systems manufactured and shipped in 1992 incorporated design improvements as a result of these tests.

In early June of 1992, a new low-impact wire rope net rockfall fence design was field tested. Although only preliminary data is available, at this time, it shows substantial improvement in capability over existing non-wire rope net designs.

A brief discussion of the field testing is presented because it directly contributes to the designs installed in 1992. Three states have added 2210 linear feet of wire rope net rockfall barriers to their methodology of rockfall mitigation since the start of 1992. One retro-fit of an existing chain-link and cable design was also completed.

Lastly, several rockfall impacts have occurred on systems recently installed in two different states. Even though there are difficulties analyzing what actually occurred, certain assumptions can be made that indicate the effectiveness of the systems.

Applications

Field Testing

In 1989, the California Department of Transportation (CALTRANS) tested four wire rope nets provided by Brugg Cable Products, Inc. and L'Entreprise Industrielle, Inc. In response to the CALTRANS tests, additional tests were conducted in 1991 and 1992 by John D. Duffy, an engineering geologist and consultant on leave from CALTRANS.

The Switzerland based research project tested 20 different designs and numerous individual net components. The result of these research projects has been the intensive testing and evaluation of more than 25 flexible rockfall designs using over 160 rock rolls with impact energies ranging from 5 ft-Tons to 369 ft-Tons.

A detailed report of the above tests are presently being prepared and should be available soon.

Pennsylvania Department of Transportation - S.R. 924

Two sections of an existing chain-link and cable fence damaged by a large rockfall in late 1990 was retro-fitted with steel wire rope safety nets. One section is a total of 160 linear feet, while the other is approximately 59 linear feet. A modified net support rope with braking devices is incorporated in the design so the existing posts could be used.

Tennessee Department of Transportation - Interstate 40

Normally the catchment areas below the slopes would be sufficient to handle the small rocks falling from the top. However, the solid rock surface affords no velocity or energy dampening. The berm at the intersection of the catchment and roadway forms a jumping ramp for the small fast rocks, propelling them to the middle of the interstate and sometimes beyond. Due to the elevation changes, site distance is also shortened.

Two sections of 10 foot high rockfall protection barrier of 100 feet and the other 440 feet in length were placed approximately 3 feet inside the Jersey barriers located at the edge of the roadway. This spacing allows for deflection of the barrier under impact conditions. The system is rated for 70 ft-Tons impact force.

California Department of Transportation - Santa Cruz

A 70 foot long, 50 ft-Ton rockfall catchment barrier was installed in Santa Cruz County, CA in the environmentally sensitive area known as The "Redwood Forest". Even though a "special" angular net was required to match the contour of the slope, the design, fabrication and installation were completed in less than 30 days. To visually blend the barrier into the background, black paint was applied to the supporting columns and black chain-link material was used on the slope side of the nets.

Ohio Department of Transportation - Steubenville

Twelve feet high wire rope nets were supplied for dual use as a temporary construction and later as a permanent rockfall protection barrier above the tie-back wall. The steel columns, supplied by the customer, are 12" x 53# H-piles with field welded cable guides. For the temporary installation, the H-piles were driven 5 feet into sand-filled 14 inch diameter holes. Fourteen hundred feet was erected to protect two recently constructed lanes of S.R. 7 from damage due to construction and blasting operations. Afterward, the barrier will be permanently installed inside the tie-back wall at the southern end of the project. Additional barriers are also planned for the upper slope and in the chutes to control potential rockfall.

A smaller 8 foot high, 200 feet long, 50 foot-Ton rated rockfall barrier will be erected at the far south end of the project. This barrier protects against rockfall in the transition area from a catchment ditch to the original slope geometry.

Rockfalls at Existing Sites

Pennsylvania Department of Transportation - S. R. 248

The 9'-6" high, 70 foot-Ton rated rockfall barrier installation was completed in January 1992 in the Lehigh Gap just north of Allentown. Sometime this spring, a 24 cubic foot rock with an approximate weight of 2 Tons was found resting in the nets. Post incident analysis using basic physics could lead to the conclusion that an impact as large as 80 ft-Tons and resulted in successful operation of the system without damage.

New Mexico Highway & Transportation - Embudo Canyon

There has finally been some positive press resulting from the rockfall catchment barriers erected in the environmentally sensitive Embudo Canyon area north of Santa Fe. A rock, that would have reached the roadway, impacted one of the supporting columns of the rockfall catchment barrier erected in the fall of 1991 and was subsequently stopped in the net.

Reconstruction of the incident, again involve some assumptions. It is assumed that the rock struck the column, split and impacted the net. Most of the energy was therefore dissipated at the column as evidenced by the damage to the column, activation of the braking element in the column retaining rope, and pull-out of one foundation anchor. None of the damage is considered major and the system performed satisfactorily.

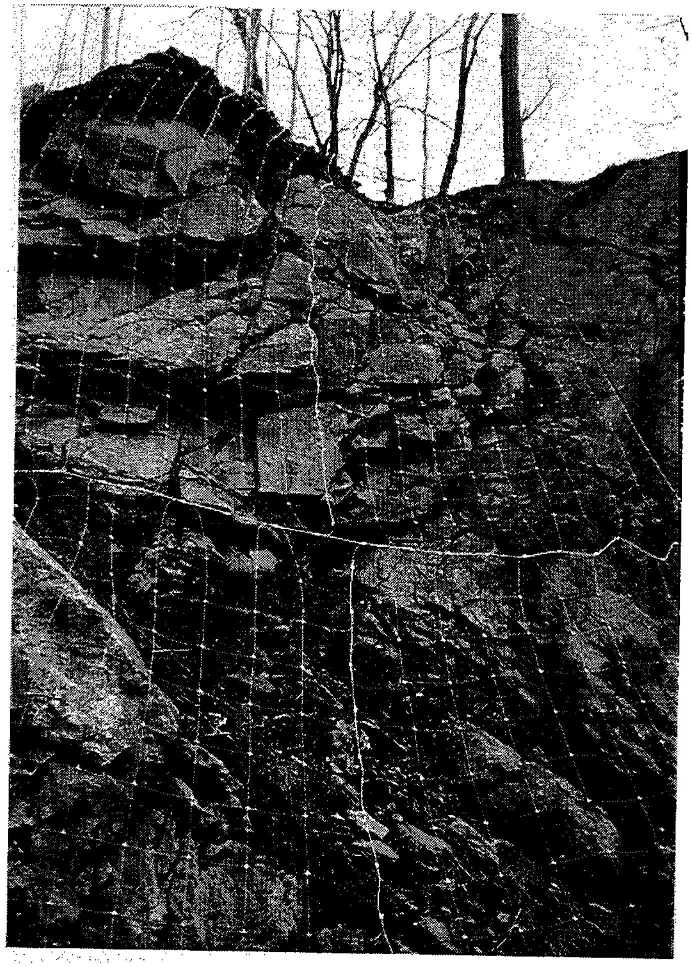
If the weight of the rock is assumed to be 3 Tons, falling from a height of 30 feet, the maximum kinetic energy would be 90 ft-Tons. These are assumptions since it is impossible to determine the actual height from where the rock fell.

CONCLUSION

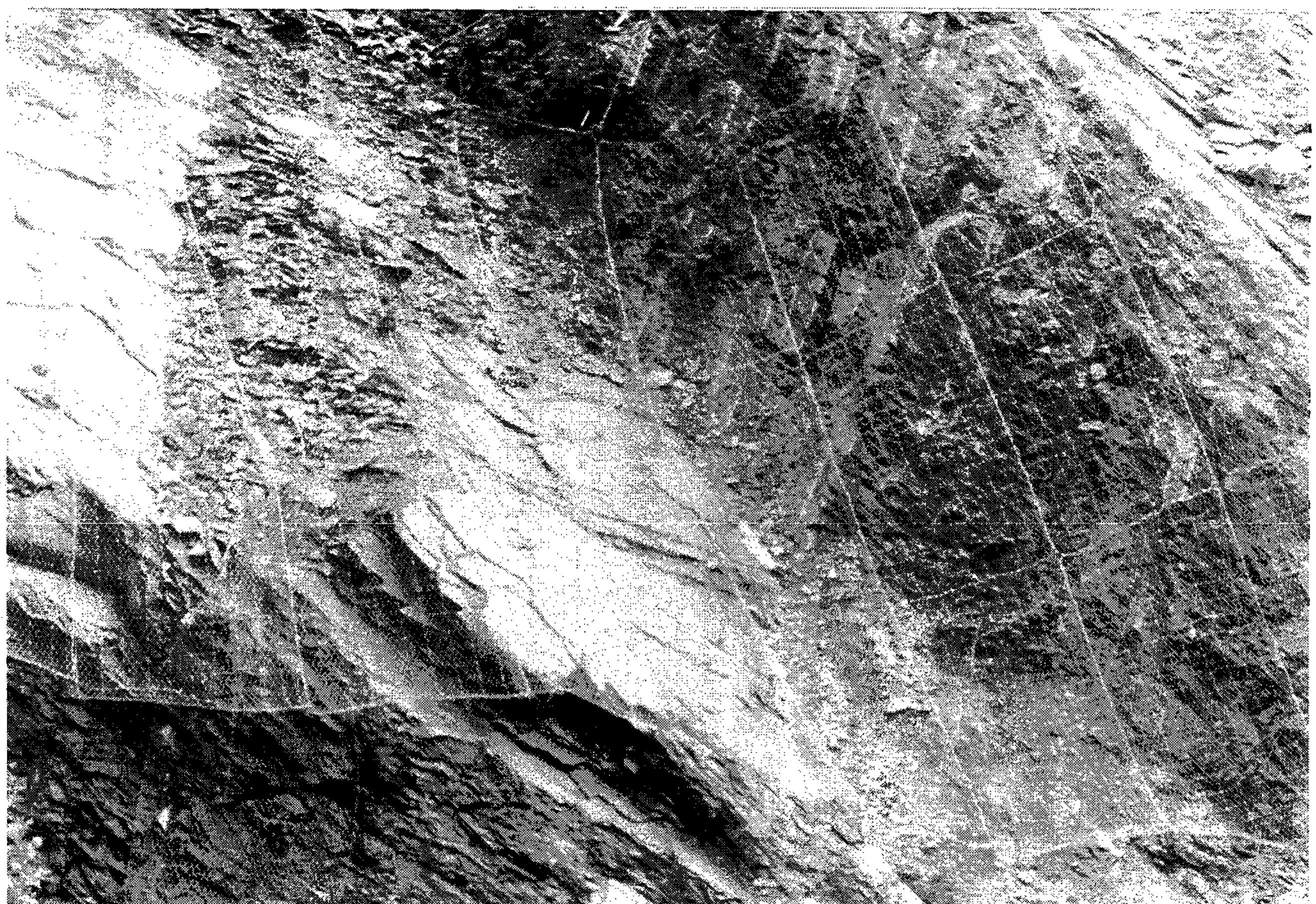
Continued testing in the field and with the installation of over 141,200 square feet of slope protection and 14,860 linear feet of rockfall protection barrier nets in the United States, there will be additional data on the effectiveness of wire rope safety nets used for rockfall mitigation. With continuing successes, this information can then be used to evaluate and design more effective systems for use in situations where alternative methods of mitigation are too costly or that would unduly affect the environment.

References

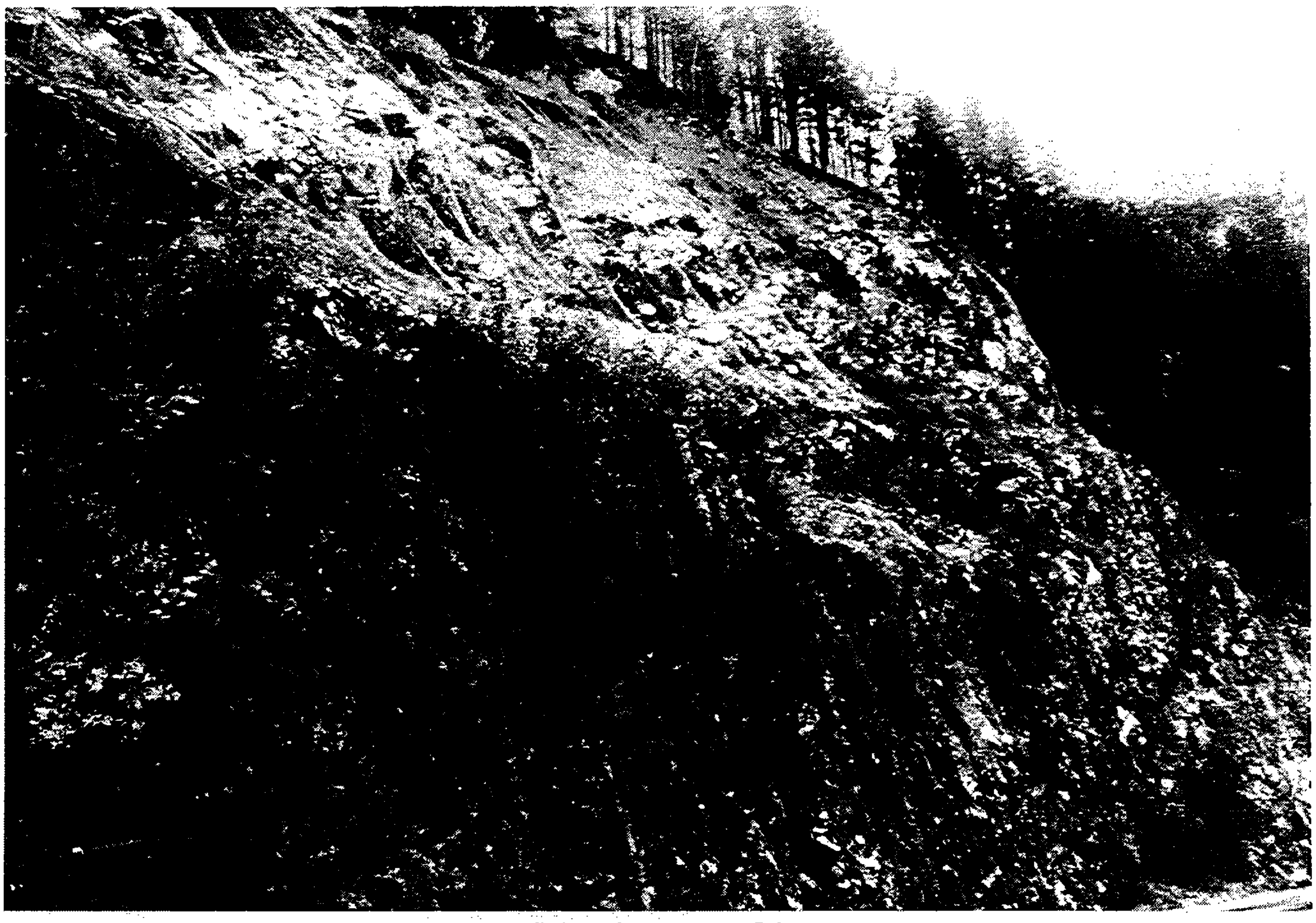
Yarnell, Charles N. (1991), "Steel Wire Rope Safety Net Systems: A Brief History of Application To Rockfall Mitigation In The U. S.", Proceedings of the National Symposium on Highway and Railroad Slope Maintenance, 34th Annual Meeting of the Association of Engineering Geologists, October, 1991, Chicago, IL



Orchard Ridge Associates
Slope Protection



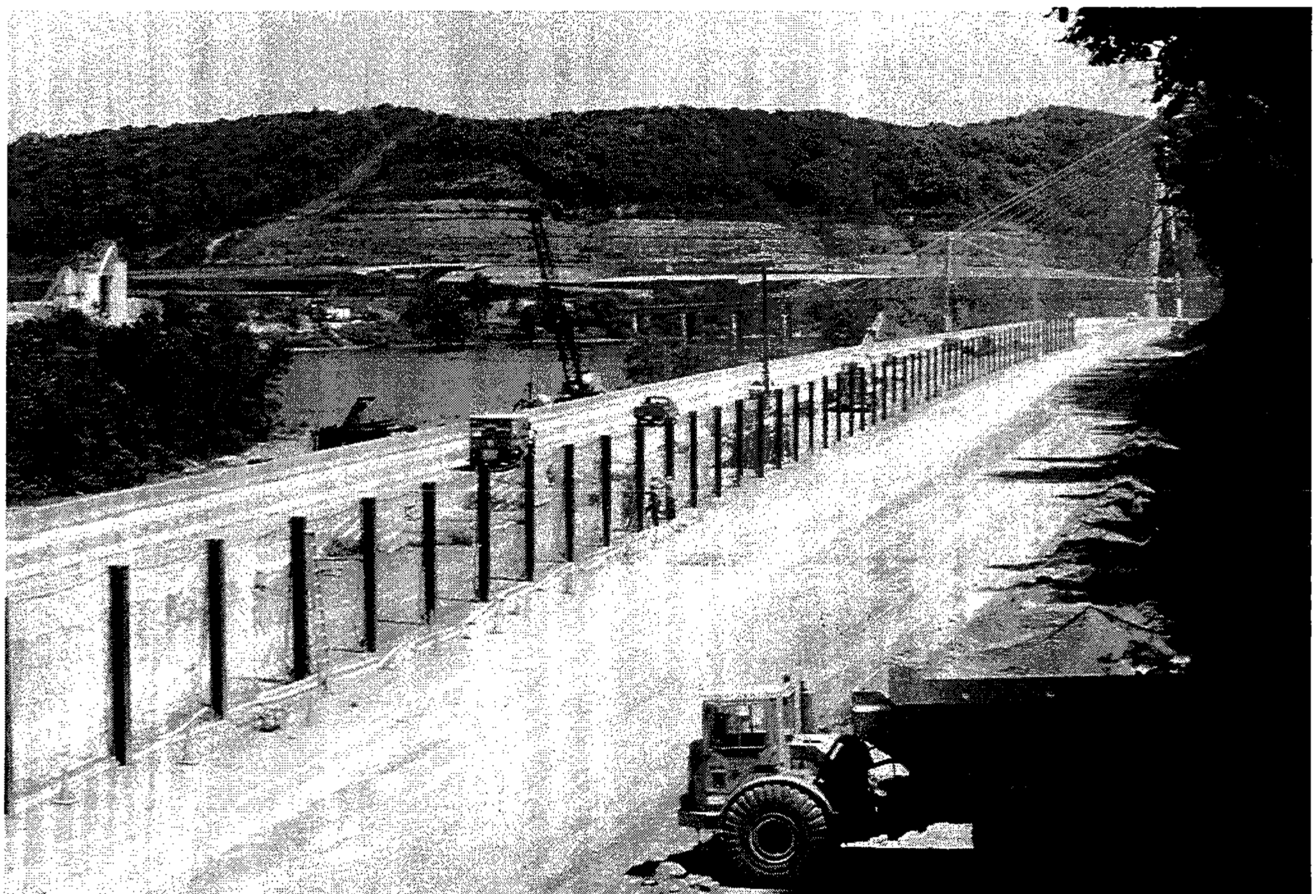
Tennessee DOT - I-40
Slope Protection



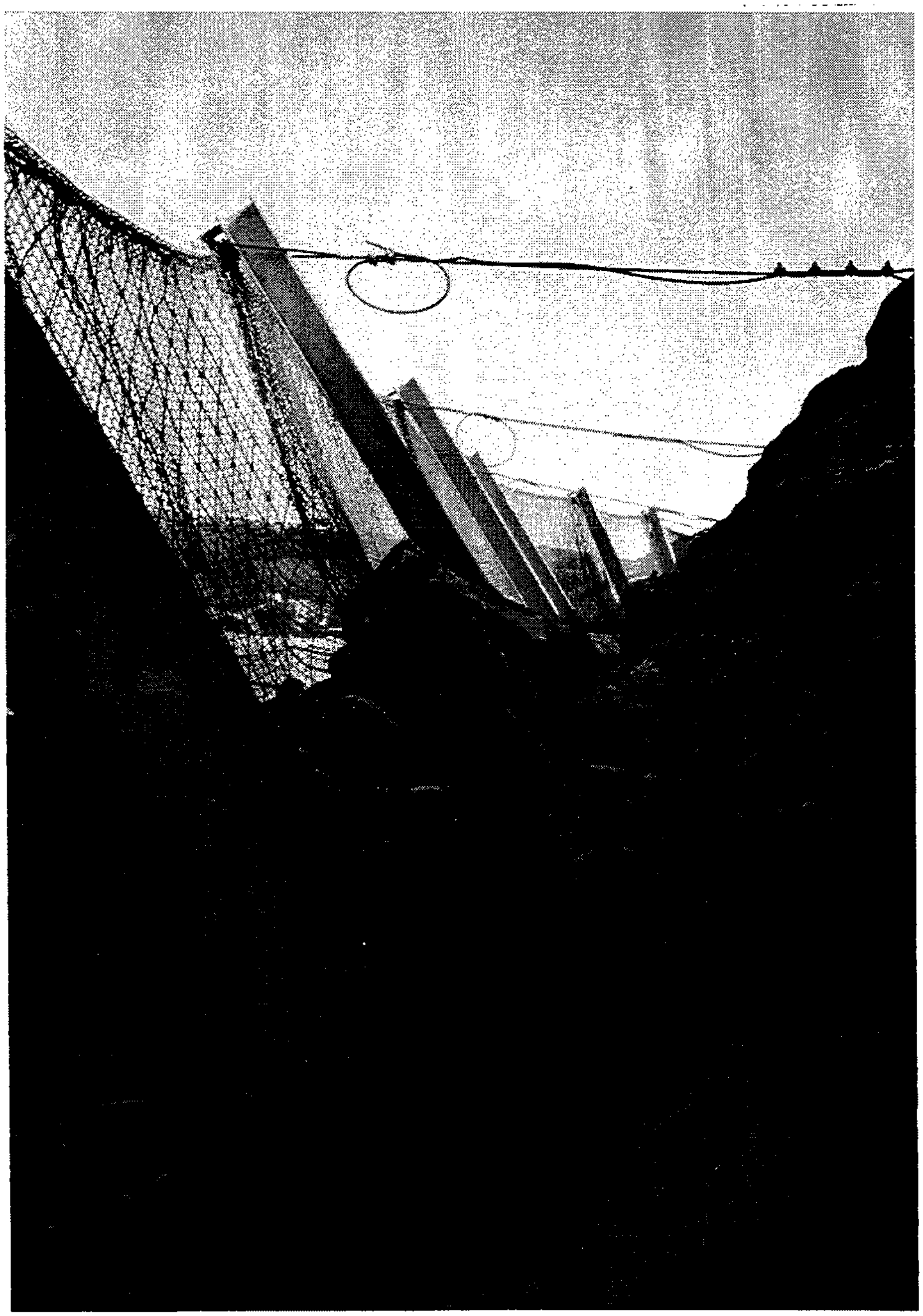
Washington DOT
Proposed Installation Site

Pennsylvania DOT - S. R. 924
Retro-fitted Nets

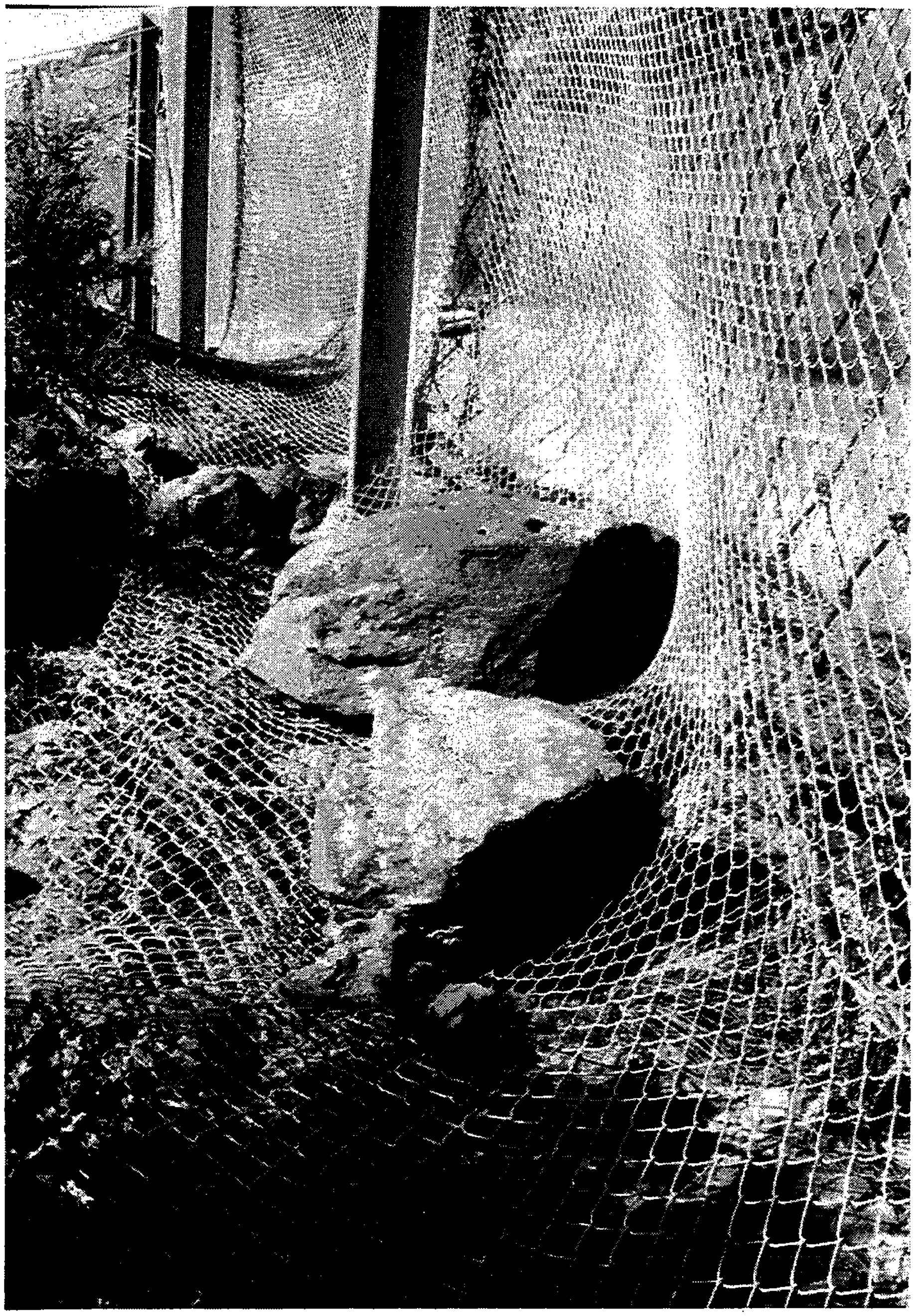
Tennessee DOT - I-40
Rockfall Catchment Barrier



Ohio DOT - Steubenville
Temporary Rock Catchment Barrier



Pennsylvania DOT - SR 248
Rockfall



New Mexico H & TD - Embudo Canyon Rockfall

VALUE ENGINEERING PROPOSAL
USING HILFIKER WELDED WIREWALL
FOR
SODA SPRINGS HIGHWAY
NEAR AUBURN, CALIFORNIA

BY

O.R. MACINTOSH, P.E.
PRESIDENT

MACINTOSH ENGINEERING & DEVELOPMENT CO., INC.
P.O. BOX 345 - 9244 OLD STATE HIGHWAY, SUITE 102
NEWCASTLE, CALIFORNIA 95658

TELEPHONE: 916/663-1366 FAX: 916/663-1175

VALUE ENGINEERING PROPOSAL USING HILFIKER WELDED WIREWALL FOR SODA SPRINGS HIGHWAY NEAR AUBURN, CALIFORNIA

ABSTRACT

The case history describes the construction of a 30' high wirewall constructed as the result of a value engineering proposal for the Federal Highway Administration near Auburn, California.

The Federal Highway Administration awarded a contract in the fall of 1989 to widen and realign a portion of the Soda Springs Highway known locally as The Foresthill Road. The project is in the foothills of the Sierra Nevada mountains, in very steep terrain.

A portion of the project, as designed, involved a geogrid soil reinforced steepened slope, at 1H:1V. The vertical height of the slope was approximately 70'.

A vertical reinforced soil wirewall (1:10 vertical slope) was proposed as an alternate in place of the geogrid soil reinforced slope. Because the existing slope was approximately 1.5 H:1V, the reinforced soil wirewall design provided substantial savings in wall surface area, excavation and backfill quantities, and provided a more environmentally acceptable solution, since less natural terrain and vegetation was disturbed.

The proposal was submitted and approved by F.H.W.A. under the value engineering clause in the specifications, resulting in a total cost reduction of $261,790.00, or a 42% savings in the bid items.

This paper describes the original design, the value engineering proposal, and the design and construction of the alternative reinforced soil wirewall.

INTRODUCTION

The Soda Springs Highway, known locally as the Foresthill Road, is a seventeen mile two-lane road connecting Interstate 80 of Auburn, California with the small town of Foresthill and tourist facilities in the Sierra Nevada mountains beyond.

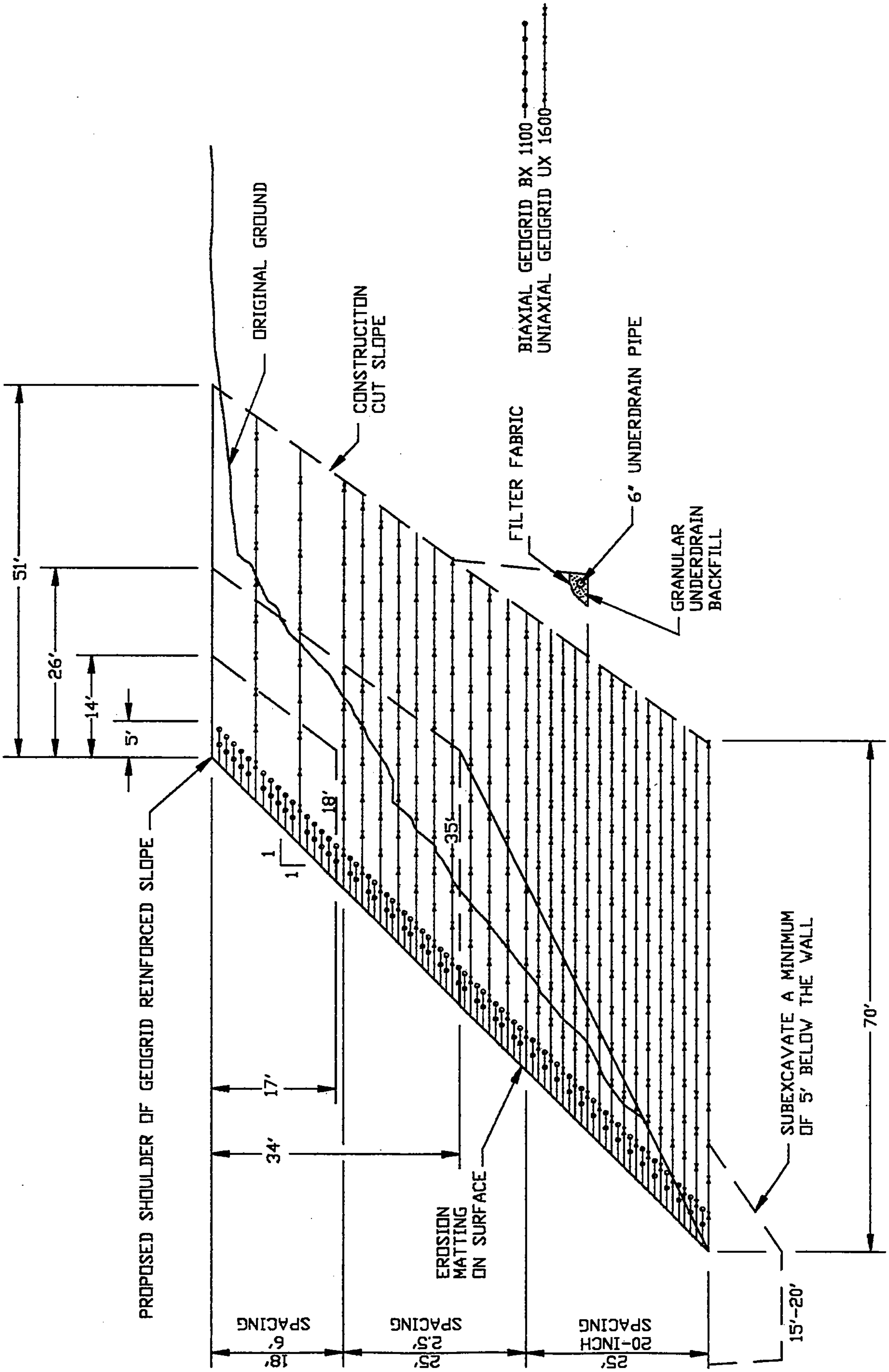
Value Engineering Proposal

The road was originally constructed as a wagon road in the late nineteenth century to carry miner's equipment and supplies during the gold rush. The road has undergone several upgrades since then, the latest in the mid-1950's. The road prior to this construction project was substandard, and the construction project, of which this paper is a part, was designed to bring a portion of the roadway into present day conformance regarding width, vertical and horizontal alignment, sight distance and safety.

The Federal Lands Division of the U.S. Department of Transportation, Federal Highway Administration advertised a reconstruction project for a 3.277 mile section of the Highway in the summer of 1989. The low bidder, in the amount of $4,104,672, was Baldwin Contracting Company of Marysville, California. The project was awarded in October 5, 1989, and work began the following spring.

This roadway is in very mountainous terrain, with many of the natural side slopes at 1V:1.5H, or steeper.

In the area between stations 215 + 50 and 217 + 40, road widening was designed using a 1V:1H geogrid soil reinforced slope, having a maximum vertical height of 68 ft. (Figure 1). The soil reinforcement was shown as Tensar BX1100 (or equal) for the biaxial geogrid at the face of the slope and Tensar UX1600 (or equal) for the uniaxial main soil reinforcement geogrid. The geogrid lengths varied from 18 ft. to a maximum of 70 ft. Borings in the area, taken by the Federal Highway Administration and Anderson Geotechnical Consultants, indicated that the slope in the area was underlain by 14 to 18 feet of stiff to hard clayey silt and silty clay, underlain by cemented silty sand and sandy silt with weathered meta-sedimentary and meta-volcanic rock extending to 35 feet. A thin layer (1 foot or less) of soft wet clayey soil was encountered at a depth of 14 feet below the existing road grade.



TYPICAL SECTION

1"=20'

VALUE ENGINEERING PROPOSAL

MacIntosh Engineering proposed to the contractor a nearly vertical , 10V:1H Hilfiker Welded Wirewall, as an alternate to the geogrid soil reinforced slope shown on the plans. Soil reinforced steepened slopes have been utilized successfully on many projects and can offer significant savings in space and cost. On this project, where the existing ground is at 1.5H:1V, the 1H:1V steepened slope comprised a greater surface area, requiring more disturbance of the natural terrain, more excavation, and more backfill material than would a more nearly vertical earth retaining structure. (Figure 2)

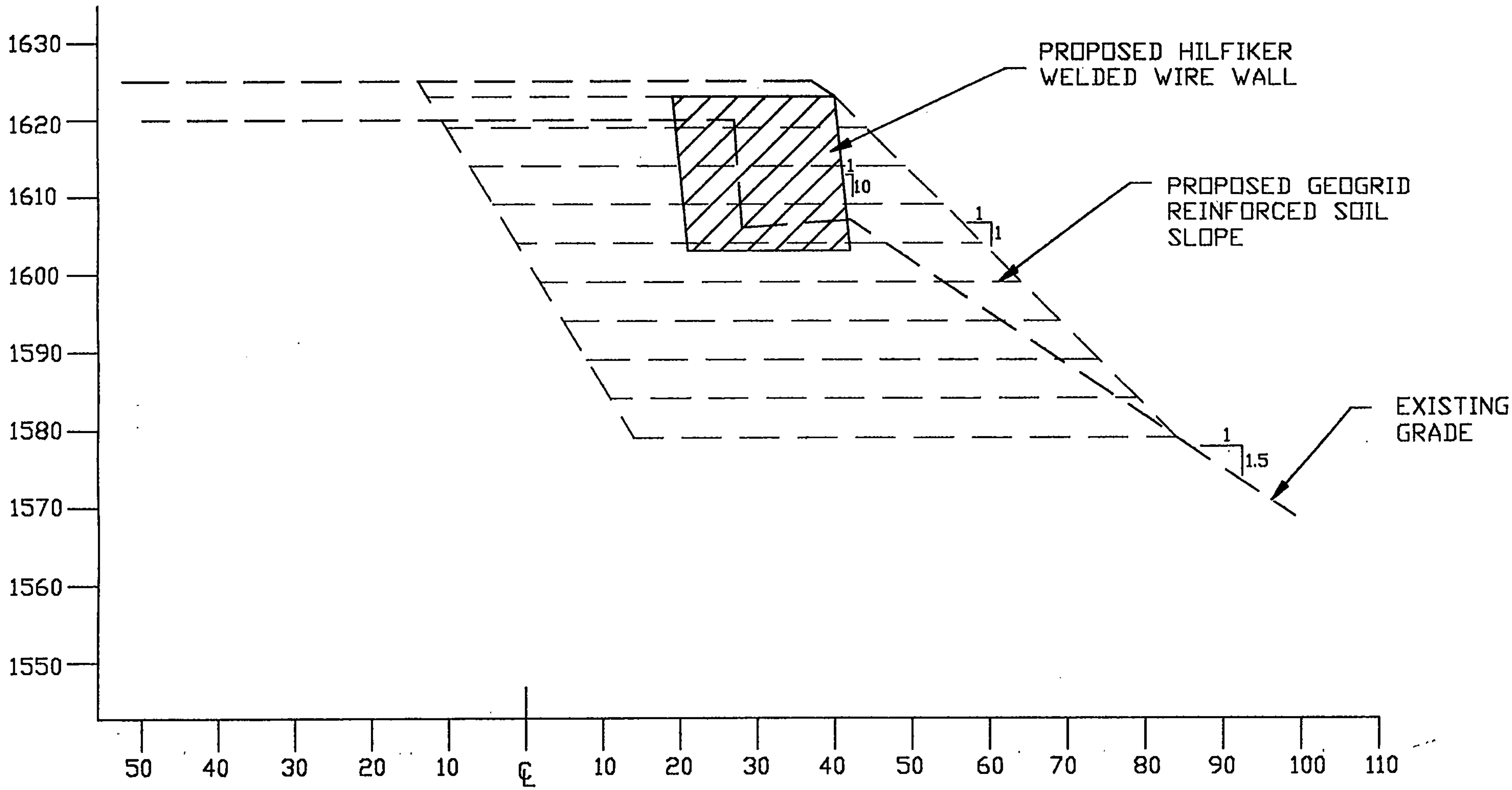


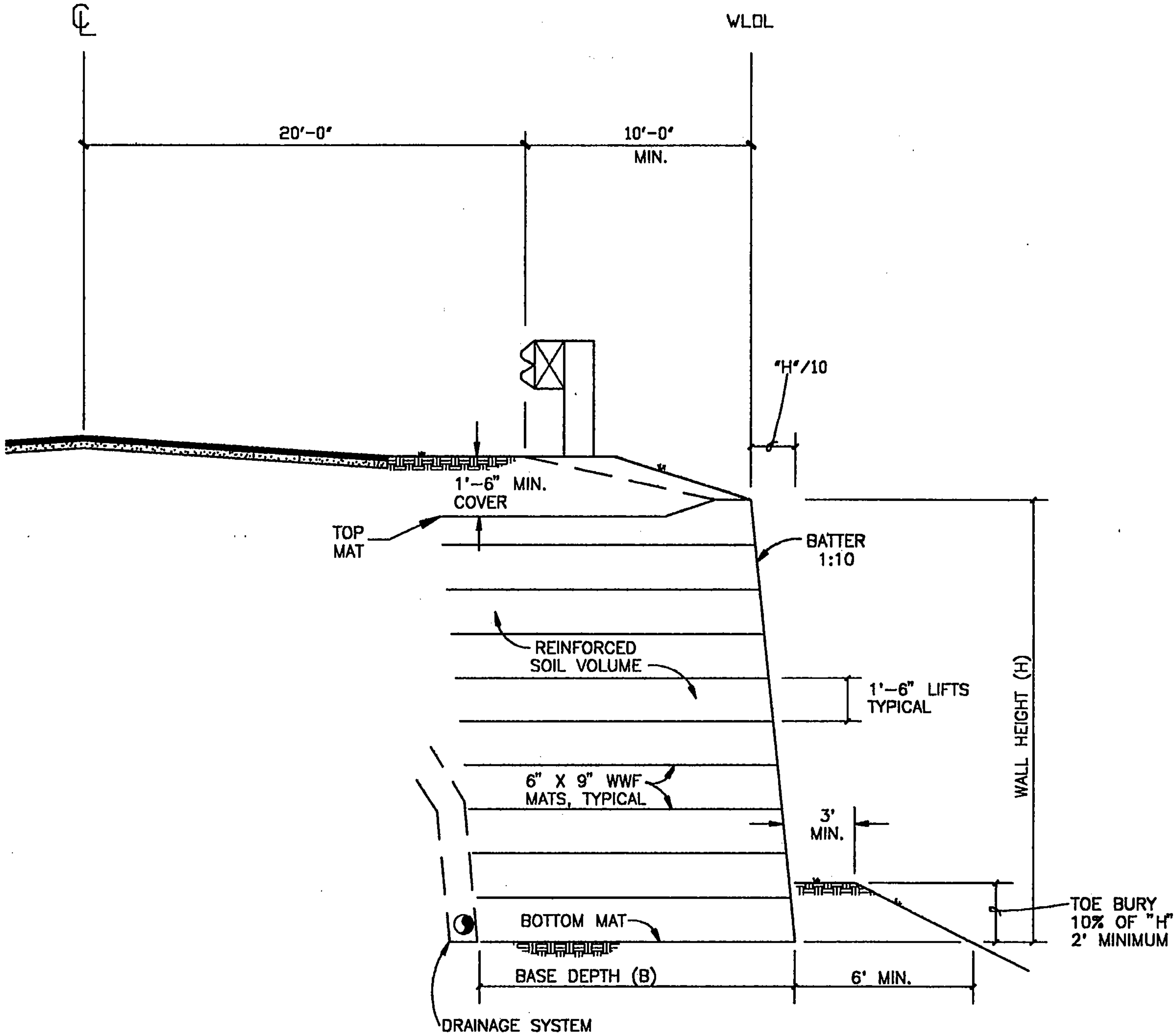
FIGURE - 2

SCALE: 1"=20'

Value Engineering Proposal

The alternate was submitted as a Value Engineering Construction Proposal (VECP), in accordance with Section 52.248.30 of the F.H.W.A. Standard Specifications. It was the opinion of the author that a vertical structure would reduce the surface area of the structure, would reduce the quantity of excavation required and provide an overall cost savings.

The Hilfiker welded wirewall, shown in Figure 3 below, is comprised of 6"x9" galvanized wire mesh, ranging in size from W4.5xW3.5 to W24xW9, depending on height, loading and foundation conditions. The wire is shipped to the job site with a 90° bend 18" high which forms the facing. The mats are 8 ft. wide. A backing mat and screen are placed behind the facing to preclude the escapement of fine materials. The backfill is placed in lifts and compacted to 90% in conformance with ASTM D-698. The backfill is normally 6" minus material with less than 25% passing the № 200 sieve, having an internal friction angle of 30°. Once backfill is brought to the top of the facing element, another facing element is attached, and the procedure is repeated to the top of the wall.



SECTION

FIGURE - 3

SCALE: NONE

Value Engineering Proposal

The Value Engineering Construction Proposal was submitted by the contractor to the Federal Highway Administration. The cost of the Hilfiker wirewall, including design, wirewall materials, excavation, wall erection, drainage system, and backfill was $368,310.00. Items saved were:

Reduce	603(1B)	2" Pipe Culvert (128 LF)	$ 7,680.00
Eliminate	613(6A)	Biaxial Geogrid	52,800.00
Reduce	613(6B)	Uniaxial Geogrid	476,616.00
Reduce	625(2)	Seeding Hydro Method (Approx.)	200.00
Eliminate	625(7)	Matting	25,100.00
Reduce		Backfill	67,704.00
		TOTAL REDUCTIONS:	$630,100.00

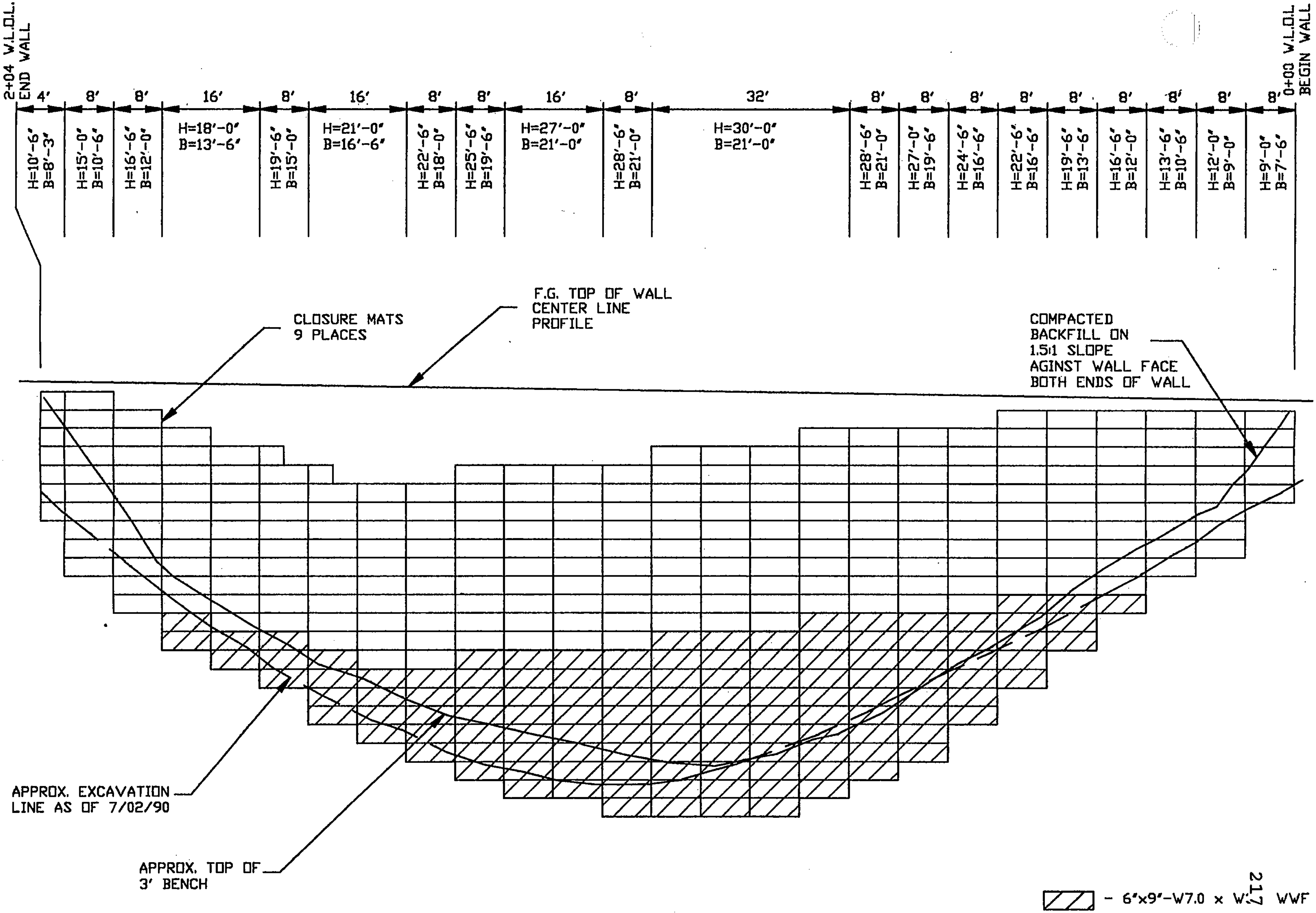
TOTAL COST SAVINGS: $261,790.00
(or 41.5% of the original bid items.)

Design:

External stability of the Hilfiker wirewall was provided by Anderson Geotechnical Consultants, Inc., of Roseville, California. The stability of the slope supporting the Hilfiker wirewall was analyzed using the computer program STABL, developed at Purdue University. Two cases were analyzed. The first case considered was the wirewall as a 5000 lb/s.f. load applied to the slope beneath it. The second case considers the area within the wall to possess a high shear strength, thus forcing the failure plane beneath it. In both cases, the wall had a factor of safety of ≥1.0 with a bedrock seismic acceleration of 0.3g. The bearing capacity was based on portions of the wall bearing in the upper clayey cohesive soil using a shear strength of 2000 lbs/s.f. with phi = 0 degrees. Using a safety factor of 3.0, the allowable bearing capacity was 4500 lbs/s.f. Consolidation tests indicated that from 1 to 1.5 inches of settlement could be expected beneath the wall, taking place over a period of one month.

The external stability of the wirewall regarding overturning and sliding as well as internal stability of the Hilfiker welded wirewall were provided by MacIntosh Engineering. Figure 4 shows the geometry of the wirewall. The structure was determined to have a safety factor against overturning of 3.47 and a safety factor against sliding of 2.04. The maximum bearing pressure was calculated to be 4954 p.s.f.

These savings did not include a substantial reduction in excavation, which was not an original pay item but considered as a subsidiary obligation of the contractor. The vertical structure also eliminated most of the potential for erosion, and lessened the impact on existing vegetation and habitat. It required less inspection time and greatly reduced the construction schedule.



217

FIGURE - 4

SCALE: HORZ. 1"=20'
VERT. 1"=10'

CONSTRUCTION

Construction of the wirewall was started September 17, 1990. The welded wire mats were a maximum of 19' 6" in length, at the maximum height of 28' 6", tapering to 8' 3" at the ends of the wall. The welded wire mats were W4.5xW3.5 except where the height exceeded 16' 6", W7xW3.5 mats were used. Drainage was provided by a prefabricated drainage composite placed on the cut slope, connecting to 6" perforated PVC pipe. The backfill was Grade E Aggregate Base material, having the following sieve analysis:

Sieve		% Passing
1"	—	100
3/4"	—	97 - 100
3/8"	—	67 - 79
№ 4	—	47 - 59
№ 40	—	12 - 21
№ 200	—	4 - 8

Construction of the wall went very well and was completed in October 3, 1990. The final quantity of wall was 4,758 s.f. to adjust for excavation and a slight discrepancy in the topography.

CONCLUSIONS

While the use of geogrid soil reinforcement for slope steepening can be very advantageous in flat or mildly sloping terrain, the utilization of a more vertical structure, such as a reinforced soil wirewall, can provide savings in cost, time, and materials and result in less disruption to existing vegetation and terrain.

REFERENCES:

1) "Retaining Wall Geotechnical Investigation"
Anderson Geotechnical Consultants, Inc.

2) Federal Highway Administration
"Contract CA FLH 124-1(1), Soda Springs - Auburn"

3) "Hilfiker Welded Wire-Retaining Wall Stability Calculations"
MacIntosh Engineering & Development Co., Inc.

IN-SITU MOISTURE CONTENT

OF

ARKANSAS SUBGRADES

by

ALVIN L. AUSTIN

and

JONATHAN A. ANNABLE

A paper presented to the

43rd HIGHWAY GEOLOGY SYMPOSIUM

FAYETTEVILLE, ARKANSAS

AUGUST 19-21,1992

IN-SITU MOISTURE CONTENTS OF ARKANSAS SUBGRADES

ALVIN L. AUSTIN AND JONATHAN A. ANNABLE

ABSTRACT

This paper will cover the field installation and data collection of Moisture Probes and the Falling Weight Deflectometer for the purpose of determining the moisture in the subgrade of a roadway and its effect on the structural integrity of the roadway. This research is one part of an on-going project concerning the Resilient Modulus testing. The basis for materials characterization in the 1986 AASHTO Guide for Design of Pavement Structures is the elastic or Resilient Modulus. The Guide states "For roadbed materials, laboratory resilient modulus tests should be performed on representative samples in stress and moisture conditions simulating those of the primary moisture seasons." It is the purpose of the research project to determine at what moisture content testing should take place. The University of Arkansas Department of Civil Engineering has conducted research which indicates that the moisture content should be between 100 and 120 percent of optimum moisture content for the T-99 Proctor Test. Eighteen sites have been selected on the basis of soil types encountered in the state of Arkansas. These sites represent a large portion of the soils in Arkansas. Where it was practical, two sites of the same soil type in differing situations were studied. This was done in order to observed the soil condition in other than one situation.

A nuclear gauge was used to collect density and moisture content data on the subgrade. A Falling Weight Deflectometer (FWD) was used to collect deflection data that was used with the ELMOD program to calculate the Resilient Modulus of the pavement, base, and subgrade. This data, compared with the moisture/density data, provides a method to determine the proper percent saturation at which MR testing should take place. Data collection and analysis is incomplete at this time, however 12 months of data is presently available. This data shows a possible correlation between the moisture, and the resilient modulus of the subgrade.

INTRODUCTION

The 1986 AASHTO Guide for the Design of Pavement Structures Recommends that resilient modulus testing be used to characterize highway materials. It also states that " for roadbed materials, laboratory resilient modulus tests should be performed on representative samples in stress and moisture conditions simulating those of primary moisture seasons."

In an effort to implement the Guide, the AHTD sponsored a research project, TRC-94, "Resilient Properties of Arkansas Soils." This research project was performed by the University of Arkansas. Representative Arkansas soil types were sampled and laboratory tested for resilient properties. One of the findings of TRC-94 is that the most significant variable affecting resilient modulus is moisture content. Furthermore, the researchers recommended development of a reliable, practical method for prediction of subgrade moisture.

This study - The In-situ Moisture Content of Arkansas Subgrades -is an effort accomplish this recommendation and sets the following objectives:

1. Determine the moisture content and density of subgrades under selected Arkansas pavements.
2. Examine and use subgrade models which may be applicable to Arkansas conditions.
3. Develop a procedure for estimating subgrade moisture content.

SITE SELECTION

One of the major considerations of the site selection was based on the soil type. The Soil Conservation Classifications was used in order to do this. The SCS classification is more descriptive than either the AASHTO or Unified Classification Systems. These soil types consist of: Carnsaw, Leadville, Sardis, Sacul, Sawyer, Amy, Perry, Jackport, Sharkey, Foley, Callowy, Alligator, and Cherokee. These soils represent many of the soil types that are encountered in Arkansas. These soils have a low permeability, and vary from poorly drained to well drained. Some are silty loams while others are a silty clay. Table 1 shows the summary of location and soil classification of the test sites. These soils represent the cases which have contributed to problems under roadways. They also represent many of the soil types that are encountered in Arkansas.

In order to gather data that can be used in most cases, several sites are placed within the same soil type. The difference in these sites would be a fill area being compared to natural ground level and a closer proximity to streams or rivers. Figure 1 shows the location of each site.

TABLE 1

SITE	HIGHWAY	COUNTY	SOIL TYPE	AASHTO CLASS.
1	113	Pulaski	Carnasaw	A-4
2	46	Grant	Sardis	A-7-6
3	46	Grant	Sacul	A-6
4	46	Grant	Amy	A-7-6
5	70	Monroe	Jackport	A-7-6
6	482	Ashley	Calloway	A-6
7	11	Lincoln	Perry	A-7-5
8	9	Dallas	Smithville	A-2-4
9	9	Dallas	Smithville	A-2-4
10	42	Cross	Alligator	A-7-6
11	42	Cross	Alligator	A-7-6
12	17	Arkansas	Sharkey	A-7-6
13	67	Jackson	Sharkey	A-7-6
14	63	Craighead	Foley	A-7-6
15	71	Benton	Cherokee	A-
16	29	Hempstead	Sawyer	A-7-6
17	14	Mississippi	Sharkey	A-7-5
18	80	Yell	Leadville	A-6, A-4

INSTALLATION

In order to achieve the most accurate results, an aluminum access tube was used with an inside diameter that allows only a minimum clearance between the tube wall and the nuclear probe. These tubes were installed by drilling a hole of slightly less than 1.80 inches in diameter; then a soil sampler was used to ream the hole to just under 2 inches. The tube was then pushed into the ground. The tips of the tubes are made of stock aluminium and are welded onto the tube to form a closed pointed end. The tubes were placed into the ground by pushing on the tip with a length of drill steel that was placed inside the access tube. After the tube is place into the hole, the tube is anchored by pouring concrete around the top of the tube. A four inch diameter PVC pipe is then positioned in the concrete. A PVC cap is placed on top

Figure 1

of the pipe. This is used as a protective covering for the tube. A rubber stopper is used to prevent moisture and dirt from entering the tube. A waterproof membrane was used in several cases where the aluminum and concrete had reacted. In these cases a new tube was placed. The reaction between the aluminum and the concrete corrodes away the tube. Before, during and after installation, the access tubes were checked for straightness and for any type on dent that may cause the probe to become hung in the tube. Access tubes were installed on each shoulder and at the centerline of the pavement. Both asphalt and concrete pavements are being monitored. See Figure 2 for typical section of installation.

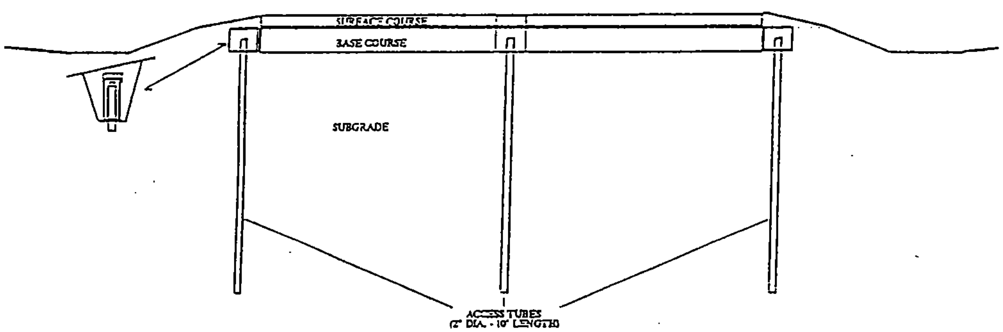


Figure 2

DATA ACQUISITION

At various times during the year, each site is visited and moisture/density readings are taken. These readings are taken at one foot intervals from a depth of 1.5 feet to 9.5 feet below the surface. The gauge is set to take a 64 second reading. This time limit was determined by weighing the time of readings against the accuracy of the reading.

At the same time as the moisture/density readings are being taken, a Falling Weight Deflectometer (FWD) is used to take deflection readings. This is done to correlate the moisture content and degree of saturation to the reaction of the pavement and subgrade to a given load. The loads that are being applied are approximately 9000 and 12,000 pounds.

The FWD data is obtained in a pattern that covers the outside wheel paths of each lane and the centerline of the road. The readings are taken every 25 feet starting 110 feet from the tubes in each direction. The reading run South to North and West to East. This was done to insure uniformity of data on each site. (See Figure 3 for FWD pattern.)

The data obtained from the nuclear density gauge shows not only the variation in moisture content but the variation in densities as each strata is measured.

There has been numerous prediction models for the movement of water through the soil. The data obtained in the project will be analyzed using at least one of these models. The infiltration of surface runoff through the pavement sections will be considered at the conclusion of the project.

FWD TEST LOCATIONS
AT
DENSITY - MOISTURE SITES

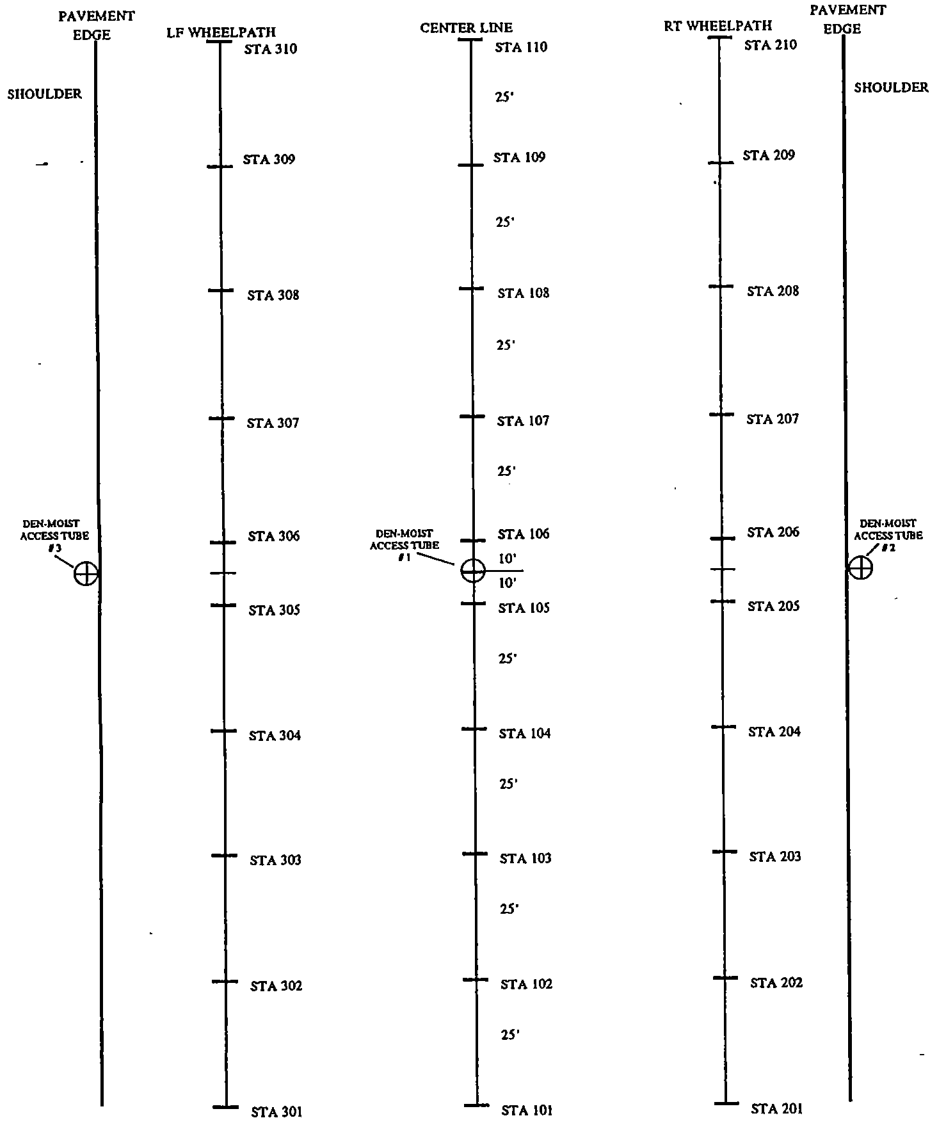


Figure 3

Shelby tubes have been pushed at almost every site. This is being done to identify the different strata in the soil and determine its classification. At the present time not all soils sampled have been tested, however the soils tested have been A-4's, A-6's, and A-7-6's. This confirms the choices of silty loams and silty clays that were considered in the site selection process. The subgrades on Sites 1 and 15 have shallow rock layers and sampling equipment for this type of sampling is not available. Soil classification will be determined, however the soil strata will not be determined. Due to the massive amount of data that has been collected only 5 moisture/density sites will be evaluated in this paper. Table 2 shows the site location and soil types that are used for this paper. All eighteen sites will be evaluated at the conclusion of this project.

TABLE 2

		SOIL TYPES		
SITE #	LOCATION	AASHTO	UNIFIED	SCS
2	Sheridan	A-4,A-7-6	ML,CH	Sardis
4	Sheridan	A-6,A-7-6	ML,CH	Amy
6	Hamburg	A- 6	CL	Calloway
7	Fresno	A-7-5	CH	Perry
16	Hope	A-7-6	CL	Sawyer

Table 3 shows the average daily traffic count and depth to water table at each site.

TABLE 3

SITE #	LOCATION	ADT	DEPTH TO WATER TABLE
2	Sheridan	650	40
4	Sheridan	650	30
6	Hamburg	3820	63
7	Fresno	340	60
16	Hope	N/A	30

The rainfall data that is being used in this project has been gathered by the National Weather Service at various stations throughout Arkansas. The rainfall data that is recorded has been obtained from the closest reporting station. While the rainfall in Arkansas can vary as much as 3 inches within a twenty mile area, the data represents a reasonable approximation of the rainfall in a given area. The rainfall data in Table 4 shows the monthly totals for each site location. By comparing the rainfall totals with the moisture in the top 4 feet of the subgrade, the data shows a time lag of 6 to 8 weeks. Because of the delay in receiving the data from the National Weather Service, the rainfall data is only complete up to December 1991.

TABLE 4

SITE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2,4	5.85	3.85	2.37	13.57	4.10	2.20	5.92	3.21	3.00	5.35	5.01	5.54
6	3.89	13.64	3.75	20.46	9.37	2.25	2.84	7.41	2.86	9.48	3.09	7.49
7	4.36	4.07	4.20	16.35	4.09	4.37	3.44	3.69	1.82	6.11	3.69	8.71
16	6.82	1.30	3.72	11.38	6.56	1.27	3.48	2.50	5.33	6.68	5.09	6.48

The moisture gradients in the soil strata seems to indicate the time lag is not only due to the water seepage into the soil, but is also a function of amount of rainfall and the type of soil that is encountered in each strata (see graphs 1 - 5 for rainfall versus moisture). The complete data on each tube shows the moisture to vary with the depth of readings. Each month's reading shows the unique strata that is encountered at each site. The readings also help to indicate a perched water table. The Shelby tube samples that were taken have confirmed the existence of such a water table at site #4.

The AASHTO "MANUAL ON SUBSURFACE INVESTIGATIONS" recommends representative in-situ samples should be obtained at every change in soil strata at an interval not to exceed 1.5 M (5 ft.) vertically. For this reason the data obtained from the moisture/density tubes will be limited to the upper 4.5 feet. This includes the upper one-half of the tube readings which will include the subgrade that is assumed to be most susceptible to being adversely effected by moisture and is the portion of the subgrade that will be tested under Resilient Modulus testing.

The data contained in the National Weather Service reports show that the moisture content in the soil is variable with regard to the amount of rainfall and the number of days during the soil has a chance to dry. Included in this are the air temperature, wind, and the number of hours per day the sun is shining. After taking all into account it is easily seen why the moisture curves do not correspond with the amount of rainfall on a monthly basis.

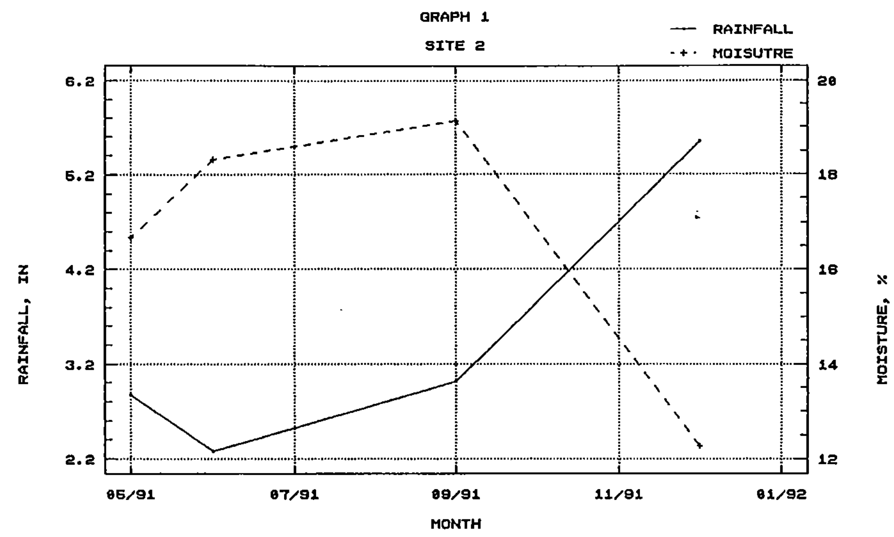
DATA ANALYSIS

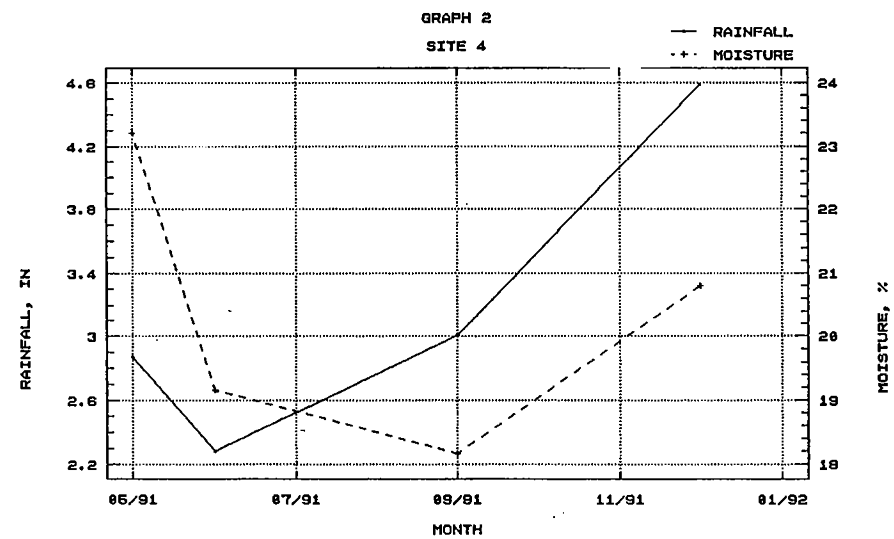
The data analysis will use the values that have been determined for the top five feet of the subgrade. This will use the top four tube readings. The readings will be from a depth of - 1.5 to - 9.5 feet. The analysis will look at the following comparisons:

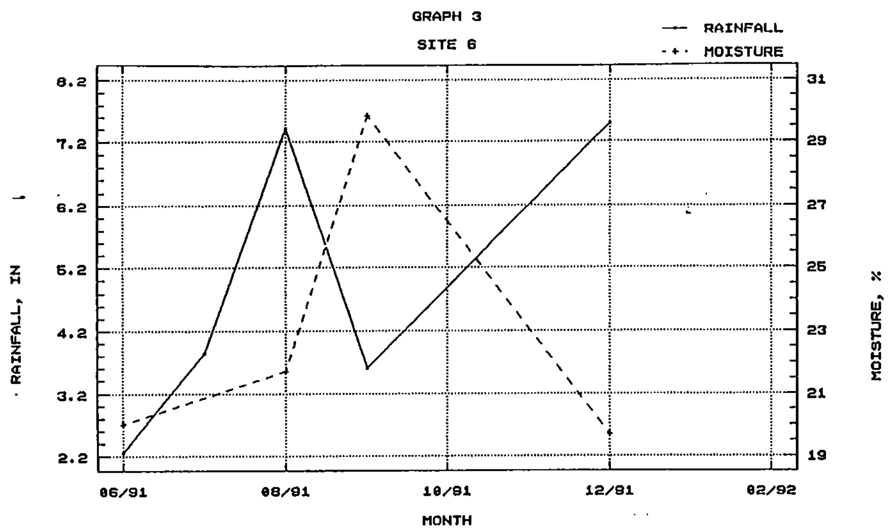
1. Rainfall v Moisture Content
2. Resilient Modulus v Moisture Content
3. Resilient Modulus (Roadhog) v Resilient Modulus
4. Roadhog Mr v Moisture Content
5. Seasonal Variation of Rainfall and Resilient Modulus
6. Variation of Moisture with Respect to Depth

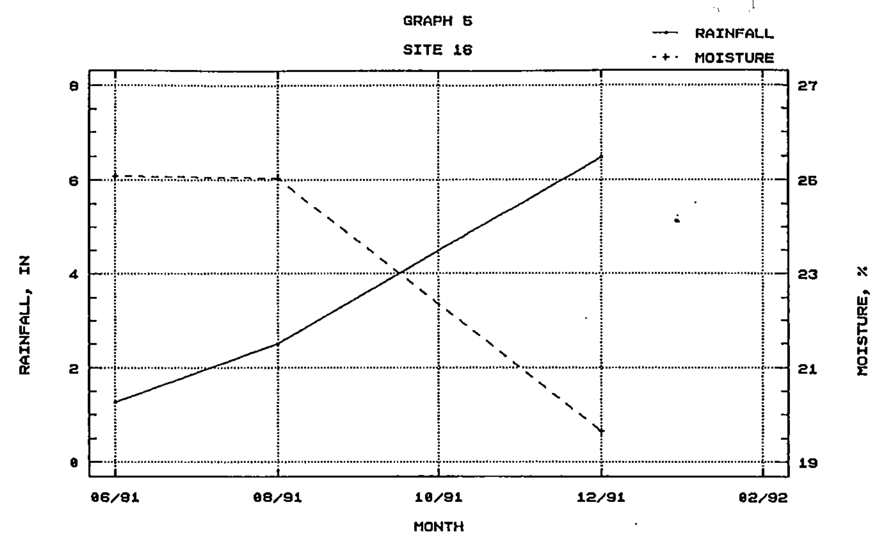
Graphs 1 - 5 show the relationship between the rainfall and moisture content. Because of data that is missing in the moisture content, the relationship does not have the continuity that might be expected. However, graphs one and two have complete data of each month of rainfall. When the rainfall data is added for the preceding months, the relationship becomes much clear than the graphs indicate.

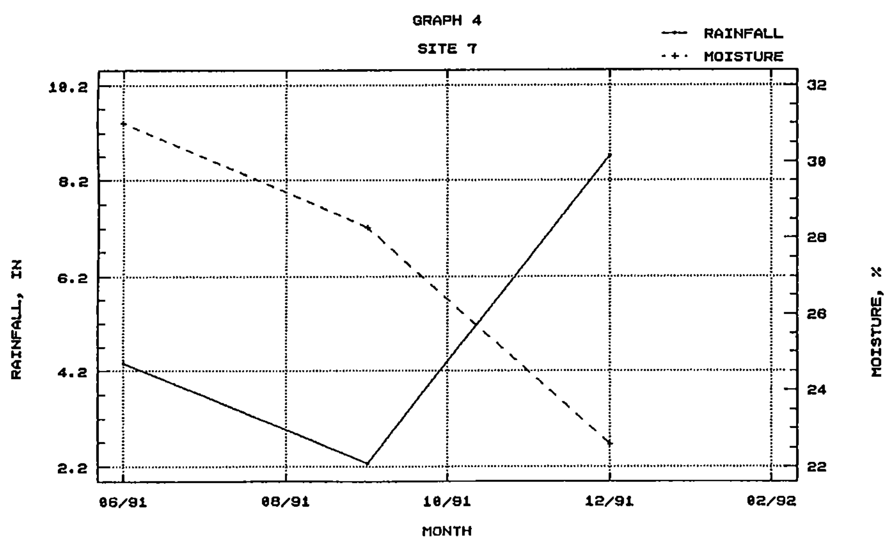
Graphs 6 - 10 show the relationship between the Resilient Modulus and the Moisture content for each site. It seems as if the moisture content does not have a major effect of the Resilient Modulus of the soils. This is because of the properties of the soil that effects the density of the soils. The density of a soil will increase as the moisture content increases to a certain point. The Procter test has

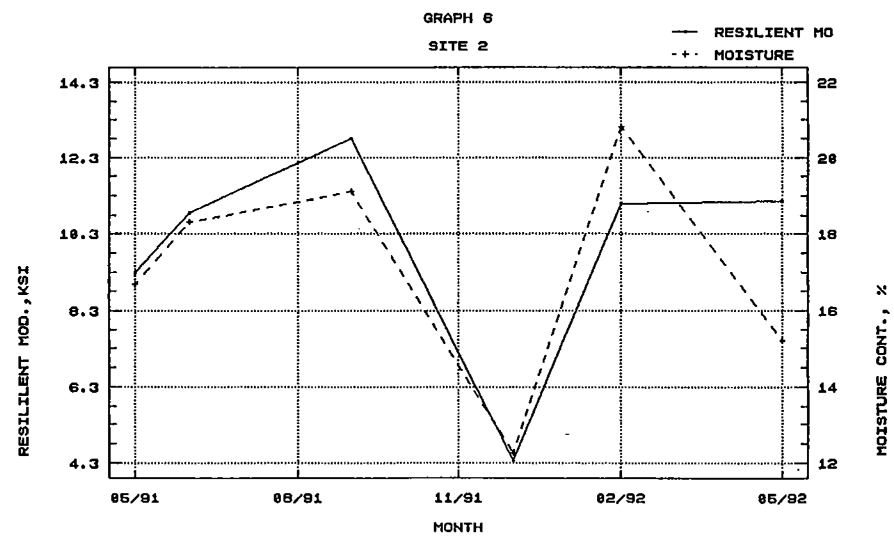


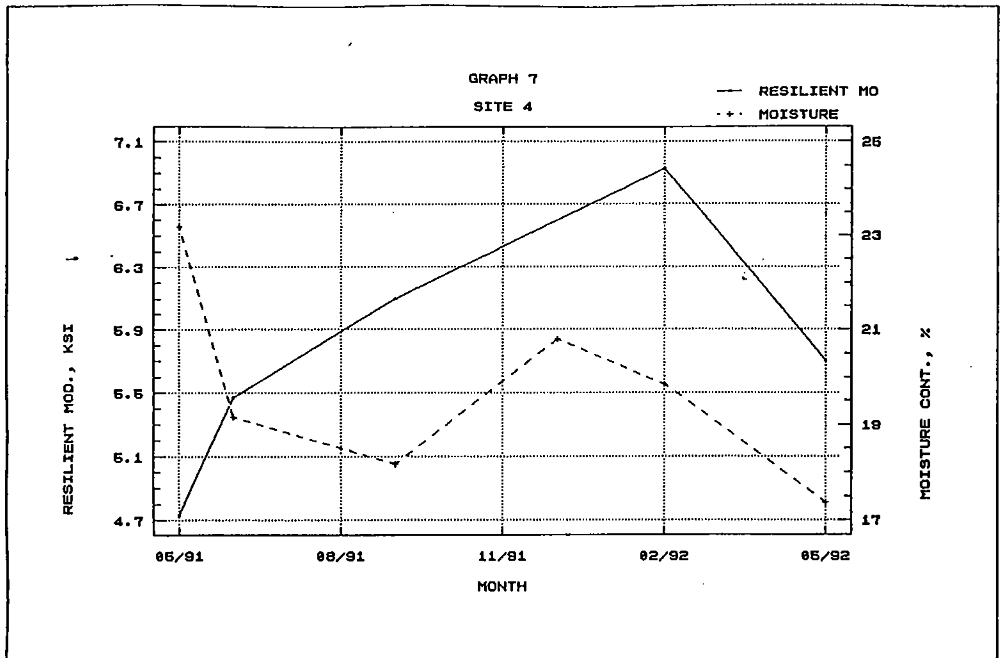


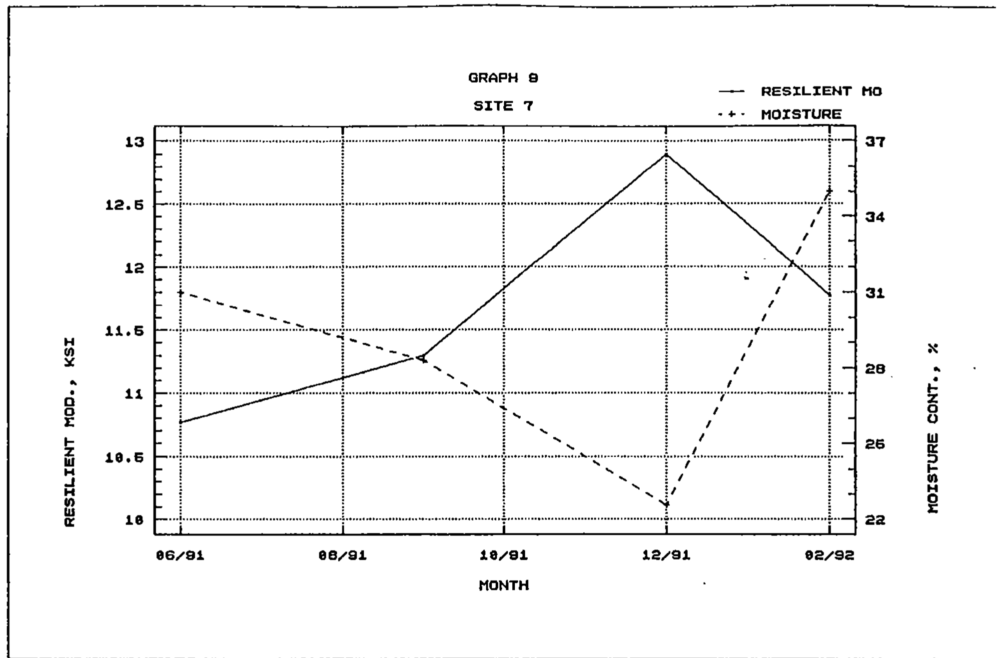

GRAPH 3
SITE 6
RAINFALL
MOISTURE
RAINFALL, IN
MOISTURE, %
MONTH
06/91
08/91
10/91
12/91
02/92

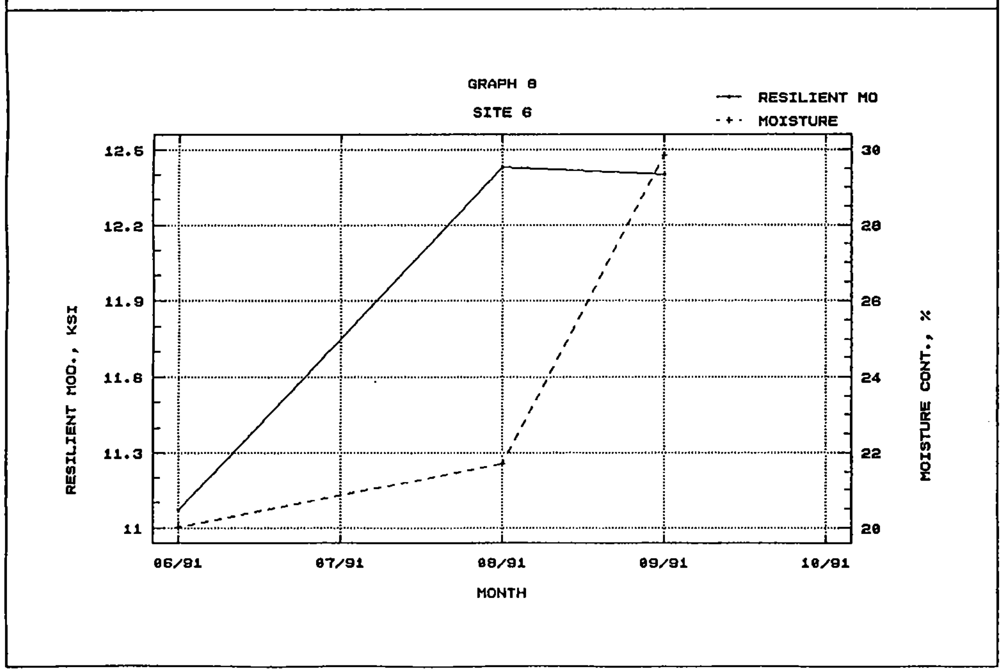

GRAPH 5
SITE 16
RAINFALL
MOISTURE
RAINFALL, IN
MOISTURE, %
MONTH
06/91
08/91
10/91
12/91
02/92

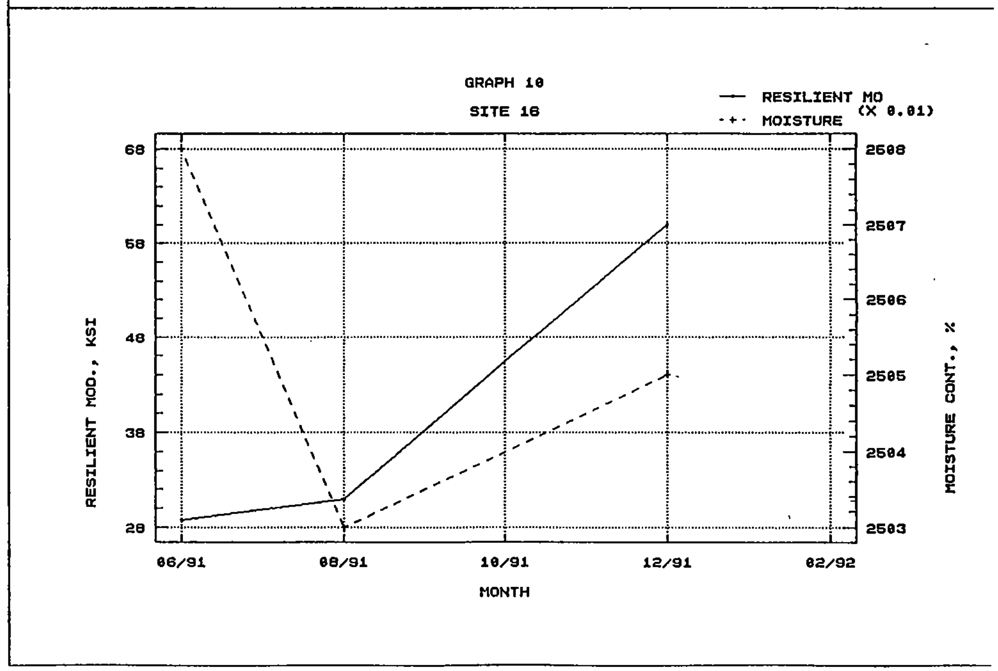

GRAPH 4
SITE 7
RAINFALL
MOISTURE
RAINFALL, IN
MOISTURE, %
MONTH
06/91
08/91
10/91
12/91
02/92


GRAPH 6
SITE 2
RESILIENT MO
MOISTURE
RESILIENT MOD.,KSI
MOISTURE CONT., %
MONTH
05/91
08/91
11/91
02/92
05/92


GRAPH 7
SITE 4
RESILIENT MO
MOISTURE
RESILIENT MOD., KSI
7.1
6.7
6.3
5.9
5.5
5.1
4.7
MOISTURE CONT., %
25
23
21
19
17
06/91
08/91
11/91
02/92
05/92
MONTH


GRAPH 9
SITE 7
RESILIENT MO
MOISTURE
RESILIENT MOD., KSI
13
12.5
12
11.5
11
10.5
10
MOISTURE CONT., %
37
34
31
28
25
22
06/91
08/91
10/91
12/91
02/92
MONTH

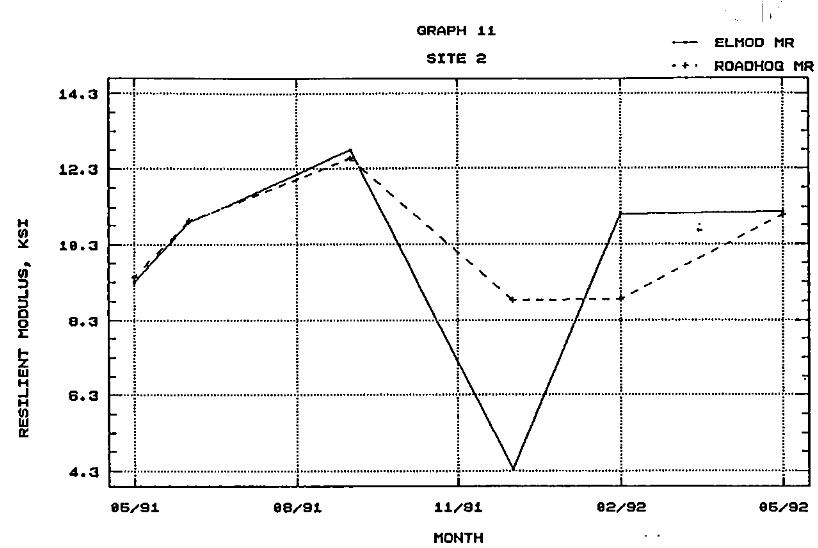

GRAPH 8
SITE 6
RESILIENT MO
MOISTURE
RESILIENT MOD., KSI
12.5
12.2
11.9
11.6
11.3
11
MOISTURE CONT., %
30
28
26
24
22
20
06/91
07/91
08/91
09/91
10/91
MONTH

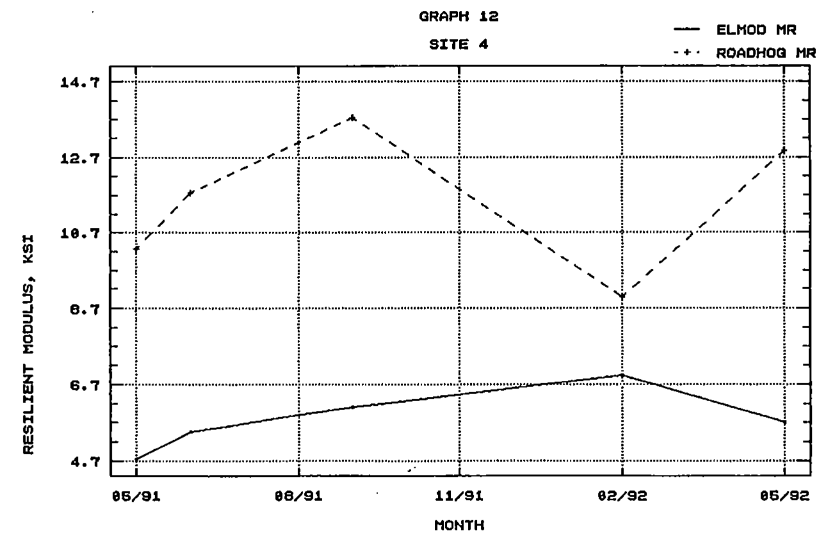

GRAPH 10
SITE 16
RESILIENT MO
MOISTURE
(X 0.01)
RESILIENT MOD., KSI
68
58
48
38
28
MOISTURE CONT., %
2508
2507
2506
2505
2504
2503
06/91
08/91
10/91
12/91
02/92
MONTH

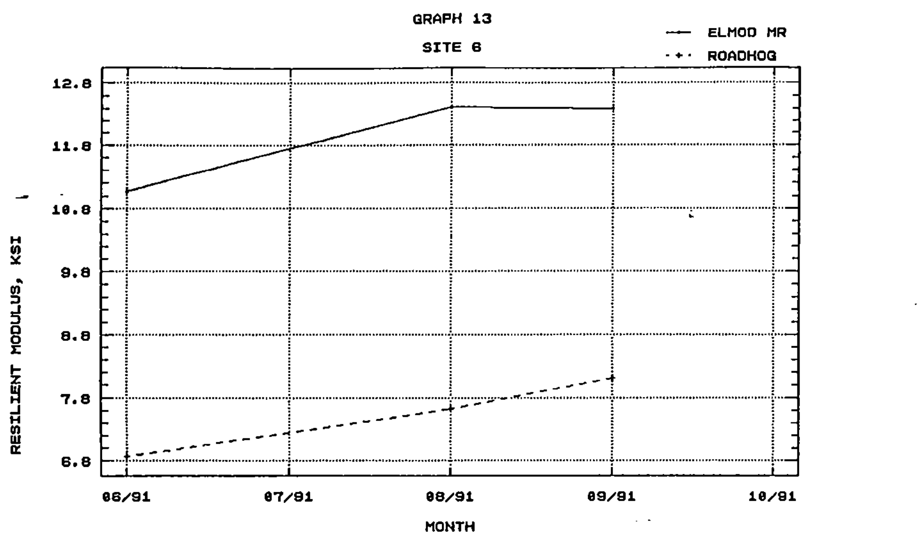
proven this for many years. Once the optimum moisture content has been reached the density starts to decrease while the moisture content increases. This would explain why the moisture increases and the Resilient Modulus increases at the same time. However, in the case of site #7, graph #9 shows a trend of the moisture decreasing while the Resilient Modulus increases. The high moisture content of the soil indicates that the optimum moisture has been reached and exceeded. This lends credence to the assumption that the optimum moisture of the soil should be exceeded during laboratory testing to determine the Resilient Modulus of the soil.

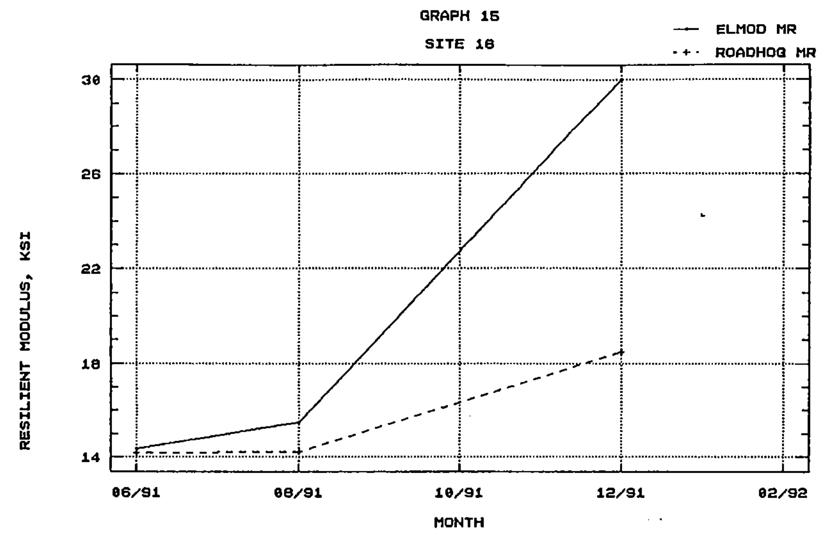
Several sites, including three of the sites chosen for this paper, have thin layers of clean sand in the soil strata. This strata of sand lies in approximately three foot intervals and is approximately 1.5 inches thick. This sand has only been found in areas that do not have a large amount of fill material and the natural subgrade has not been disturbed. As the movement of water through the soil continuously picks up and deposits clay and silt particles in this sand layer, the actual soil type of the subgrade changes and therefore, the densities of the soil will and does change.

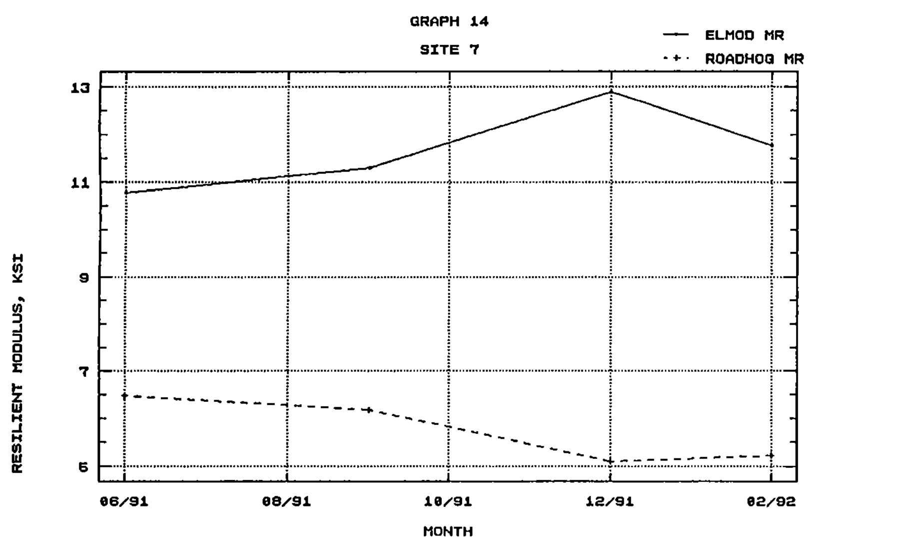
Graphs 11 - 15 show the comparison of the Resilient Modulus as computed using the ELMOD and Roadhog programs. ELMOD is the program that was developed by Dynatest for use with the their FWD. ELMOD is the Evaluation of Layer Moduli and Overlay Design. Elmod uses the FWD data to back






GRAPH 13
SITE 6
ELMOD MR
ROADHOG
RESILIENT MODULUS, KSI
12.8
11.8
10.8
9.8
8.8
7.8
6.8
06/91
07/91
08/91
09/91
10/91
MONTH

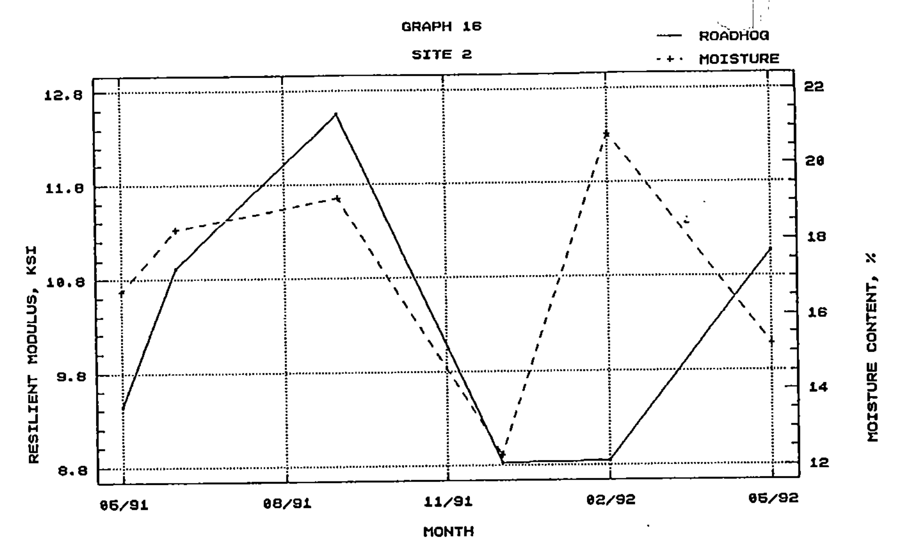

GRAPH 15
SITE 16
ELMOD MR
ROADHOG MR
RESILIENT MODULUS, KSI
30
26
22
18
14
06/91
08/91
10/91
12/91
02/92
MONTH

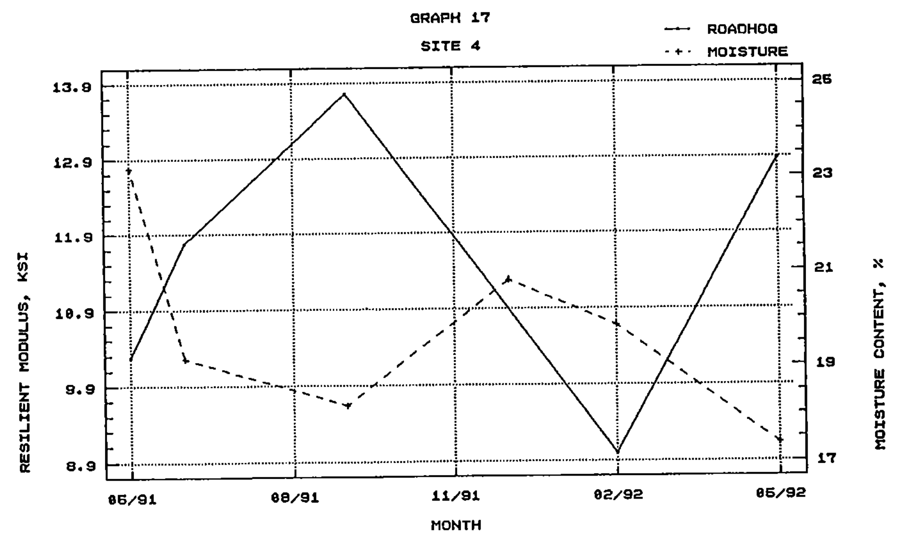

GRAPH 14
SITE 7
ELMOD MR
ROADHOG MR
RESILIENT MODULUS, KSI
13
11
9
7
5
06/91
08/91
10/91
12/91
02/92
MONTH

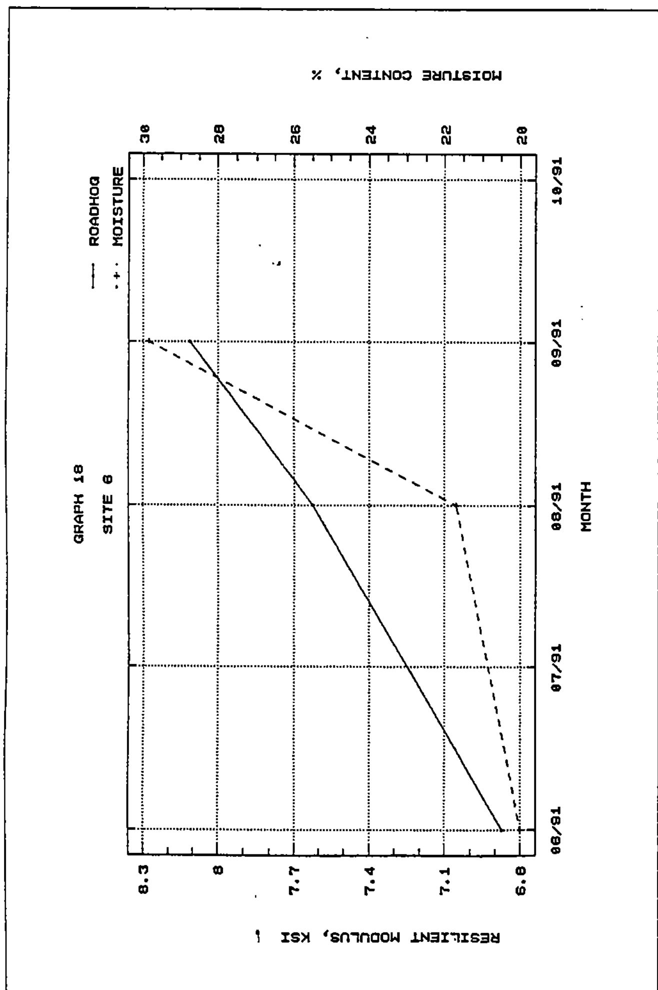
calculate the Resilient Modulus of the subgrade using layer moduli. Roadhog was developed by at The University of Arkansas at Fayetteville by Elliott, Hall, Morrison and Hong. This method of calculating the Resilient Modulus of the subgrade is based on FWD data and is a finite element pavement analyses program. Roadhog uses the deflection of the data gathered at the geophone sensor located at 36 inches from the loading plate. The Roadhog program was developed based upon several Arkansas soil types and several of the 18 sites that are being monitored are the same sites that were used in the development of this program.

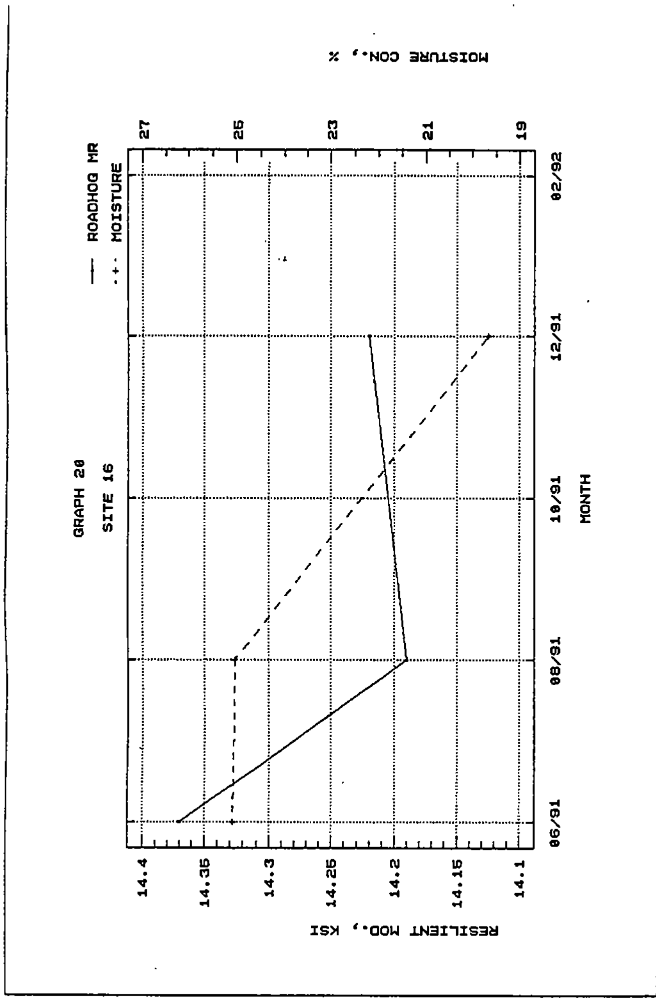
Graphs 16 - 20 show the relationship between the Resilient Modulus as computed by the Roadhog Program versus the Moisture Content. The curves are similar to the ELMOD curves, however the values vary somewhat. The Roadhog program tends to be more conservative than ELMOD on the whole. This set of graphs show the Resilient Modulus to be more dependent on the moisture content of the soil and base than ELMOD. The graphs still show the decrease of MR with the increase in moisture content and an increase of MR with a decrease of moisture content.

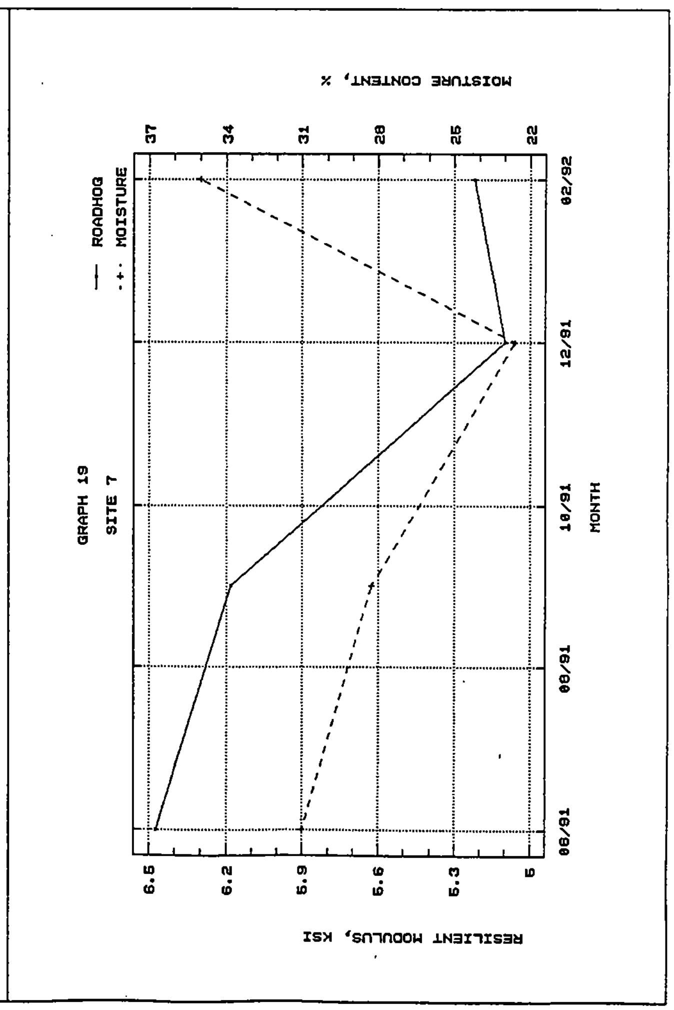
Graphs 21 - 25 show the relationship between the Roadhog Resilient modulus and the rainfall for the given months. Because of the time lag between a rainfall event and the actual effect on the moisture conditions in the subgrade, there can be no real correlation made. The time lag between the rainfall event and the effect on the resilient modulus is

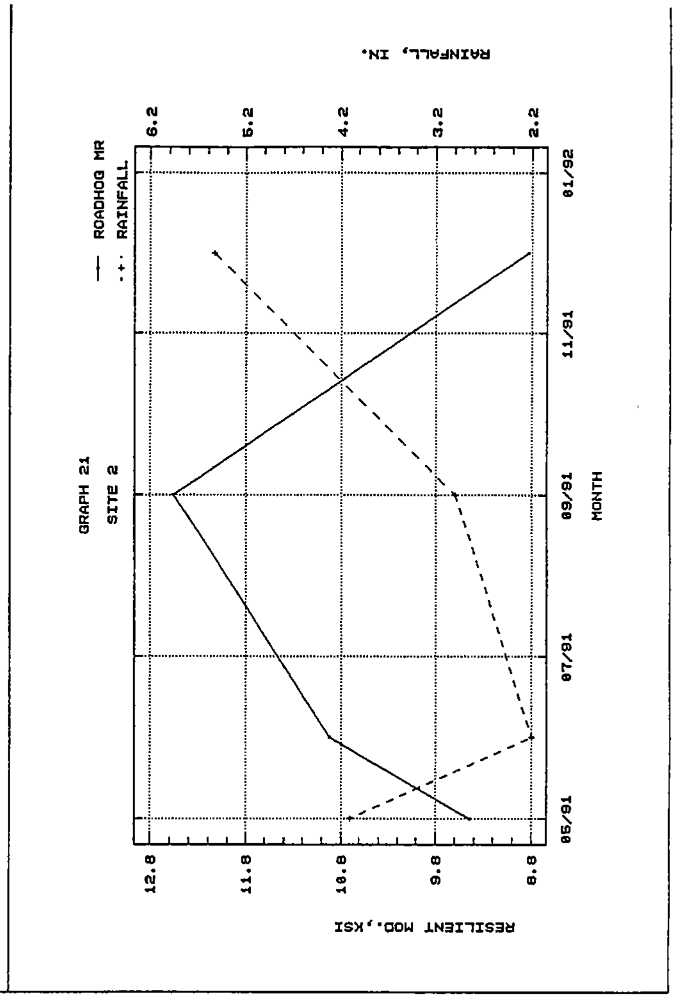


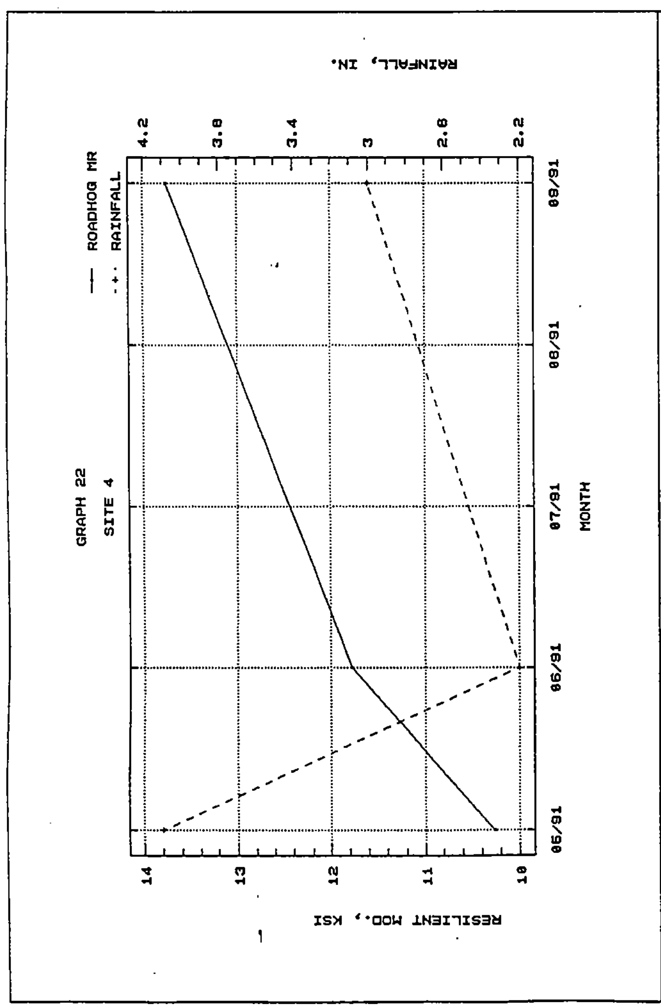


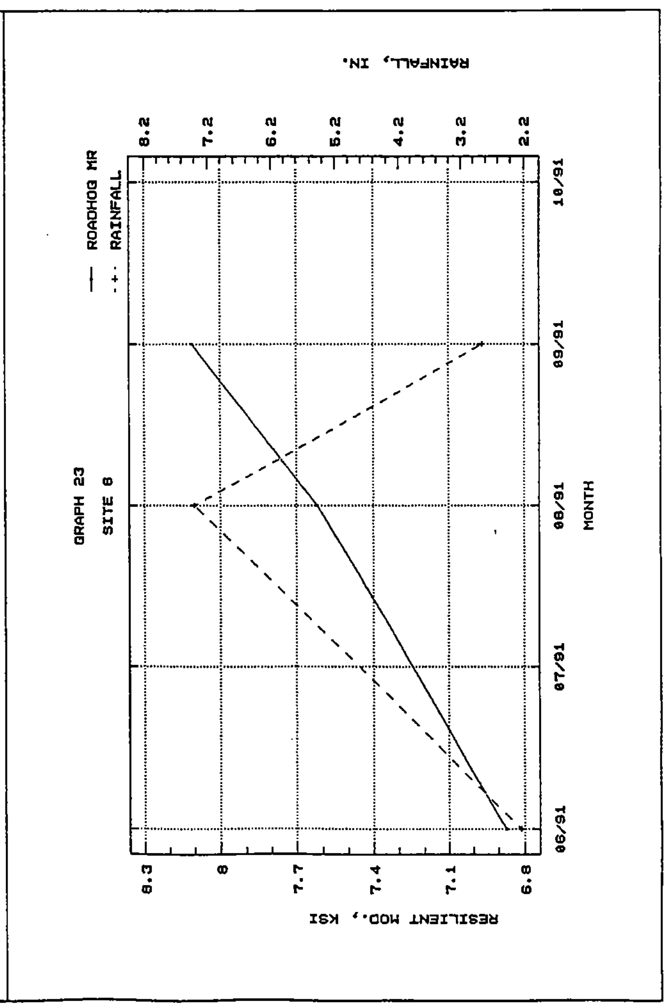

GRAPH 18
SITE 6
ROADHOG
MOISTURE
RESILIENT MODULUS, KSI
MOISTURE CONTENT, %
MONTH
06/91
07/91
08/91
09/91
10/91
8.3
8
7.7
7.4
7.1
6.8
30
28
26
24
22
20

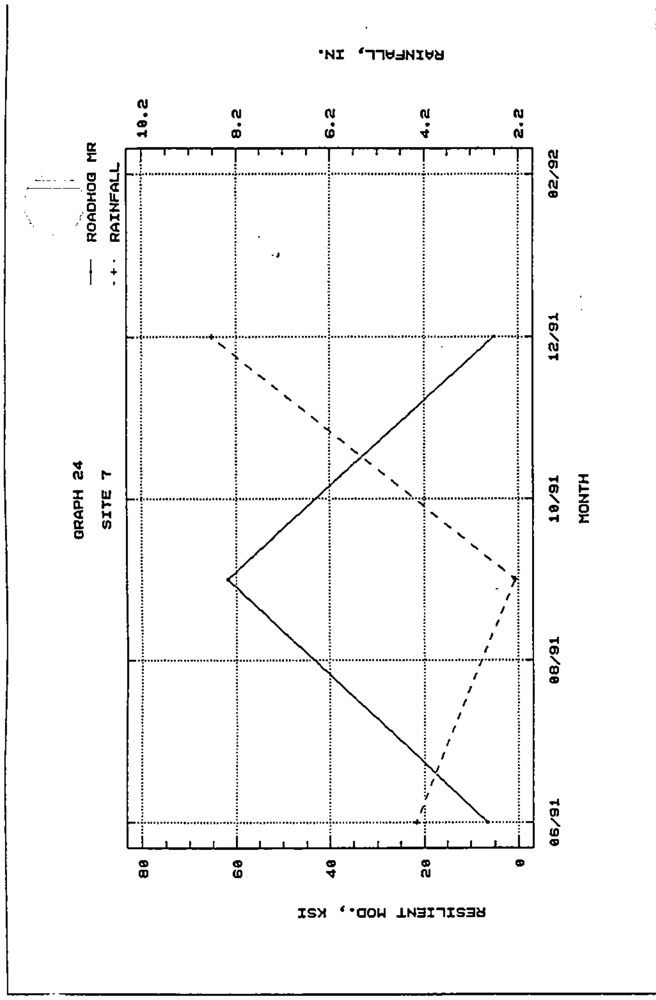

GRAPH 20
SITE 16
ROADHOG MR
MOISTURE
RESILIENT MOD., KSI
MOISTURE CON., %
MONTH
06/91
08/91
10/91
12/91
02/92
14.4
14.35
14.3
14.25
14.2
14.15
14.1
27
25
23
21
19

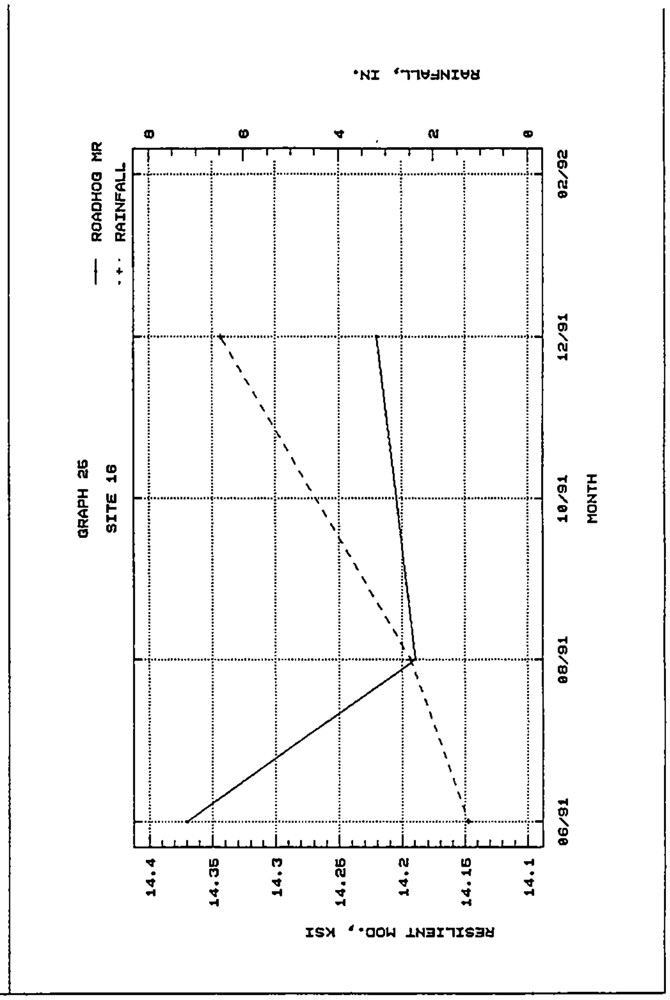

GRAPH 19
SITE 7
ROADHOG
MOISTURE
RESILIENT MODULUS, KSI
MOISTURE CONTENT, %
MONTH
08/91
08/91
10/91
12/91
02/92
6.5
6.2
5.9
5.6
5.3
5
37
34
31
28
25
22


GRAPH 21
SITE 2
ROADHOG MR
RAINFALL
RESILIENT MOD.,KSI
RAINFALL, IN.
MONTH
05/91
07/91
09/91
11/91
01/92
12.8
11.8
10.8
9.8
8.8
6.2
5.2
4.2
3.2
2.2


GRAPH 22
SITE 4
ROADHOG MR
RAINFALL
RESILIENT MOD., KSI
10
11
12
13
14
RAINFALL, IN.
2.2
2.6
3
3.4
3.8
4.2
05/91
06/91
07/91
08/91
09/91
MONTH


GRAPH 23
SITE 8
ROADHOG MR
RAINFALL
RESILIENT MOD., KSI
6.8
7.1
7.4
7.7
8
8.3
RAINFALL, IN.
2.2
3.2
4.2
5.2
6.2
7.2
8.2
06/91
07/91
08/91
09/91
10/91
MONTH


GRAPH 24
SITE 7
ROADHOG MR
RAINFALL
RESILIENT MOD., KSI
0
20
40
60
80
RAINFALL, IN.
2.2
4.2
6.2
8.2
10.2
06/91
08/91
10/91
12/91
02/92
MONTH

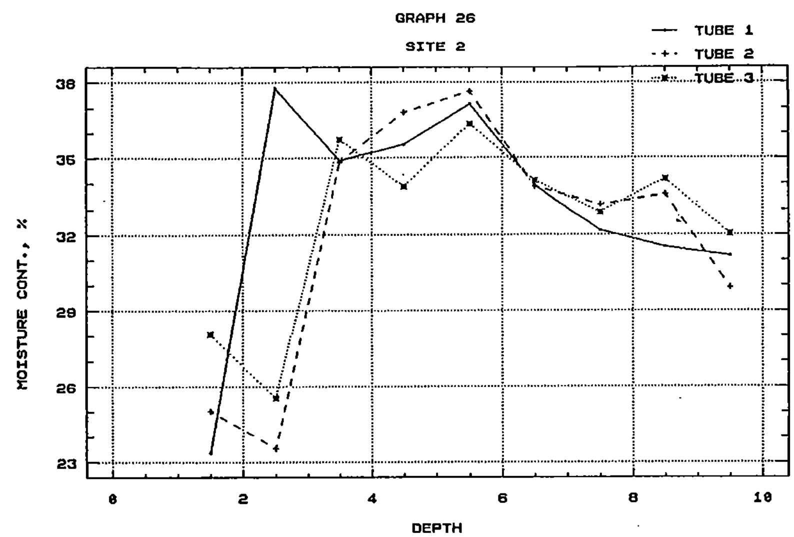

GRAPH 25
SITE 16
ROADHOG MR
RAINFALL
RESILIENT MOD., KSI
14.1
14.15
14.2
14.25
14.3
14.35
14.4
RAINFALL, IN.
0
2
4
6
8
06/91
08/91
10/91
12/91
02/92
MONTH

noticeable however. This reinforces the assumption that the moisture content is one of the major role players in the insitu resilient modulus.

Graphs 26 and 27 are sample graphs of the change in moisture content as a function of depth. Generally the moisture content stabilizes as the depth decreases. The nearness of the water table and the effect of a perched water table have a great effect on the moisture content in the subgrade. As the seasonal changes in the water table takes place, the increase or decrease in the subgrade moisture content is noticeable.

Determination of the water table depths was achieved by using the USGS Quad Maps and the information that is available from water well drillers logs. The two sites that are represented in these graphs are Site #2, located in a low laying area northeast of Sheridan and Site #7, located north of Fresno. The changes that are represented are typical of changes that have been encountered throughout the state during this project.

Table 5 illustrates the yearly variation of moisture constent on a per level basis. The seasonal variation is what will determine the actual moisture content for the resilient modulus test. The wide range of values is partly due to flooded rice fields at one site (7) and low laying a area adjacent to the site (16). The very high values at sites 2 and 4 are an indication of the perched water table at the sites and a highly varible saturation level.



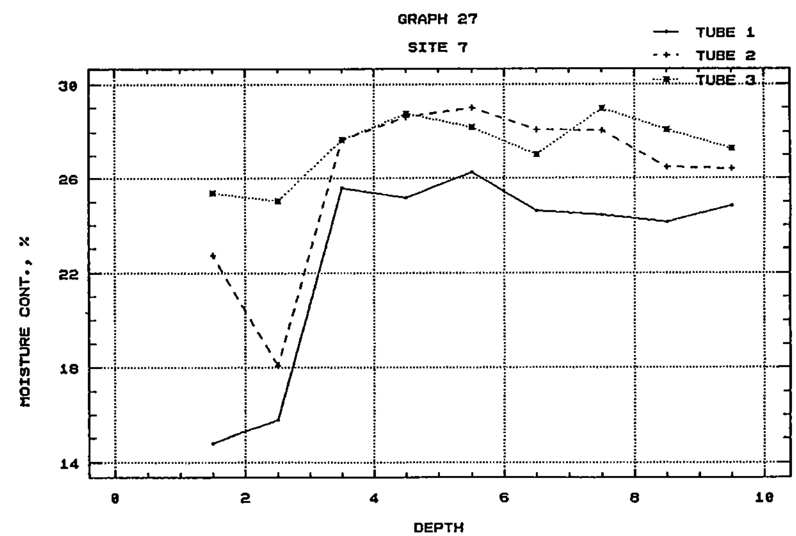


TABLE 5

Site #	Level	High/Low Tube 1	2	3	Average 1/2/3
2	-1.5	19/28	19/26	19/27	24/24/24
	-2.5	17/25	15/20	19/28	21/18/25
	-3.5	20/37	21/29	20/28	28/25/26
	-4.5	19/44	20/36	25/44	28/27/30
	-5.5	17/56	20/40	25/120	31/29/55
	-6.5	17/68	18/39	26/193	34/28/75
	-7.5	18/95	18/41	25/140	41/28/60
	-8.5	19/63	17/37	24/187	33/26/73
	-9.5	26/25	17/24	24/161	13/20/67
4	-1.5	5/9	18/11	9/18	7/14/11
	-2.5	11/27	13/22	14/17	14/15/16
	-3.5	15/18	17/27	17/27	16/20/21
	-4.5	20/24	19/28	18/33	22/23/25
	-5.5	22/28	19/113	17/28	25/31/24
	-6.5	22/214	20/27	22/27	46/24/24
	-7.5	21/55	19/52	25/31	27/26/28
	-8.5	16/32	19/21	22/28	24/23/25
	-9.5	21/30	20/31	20/26	25/24/24
6	-1.5	10/15	8/27	15/37	11/17/20
	-2.5	19/28	14/30	19/29	22/22/22
	-3.5	21/31	20/40	25/24	25/30/29
	-4.5	20/29	22/28	24/32	24/25/26
	-5.5	20/29	20/30	23/32	23/26/27
	-6.5	19/26	19/29	22/28	23/24/24
	-7.5	20/27	20/26	22/27	23/23/24
	-8.5	20/26	20/27	23/26	23/24/25
	-9.5	21/26	20/28	22/24	23/24/23
7	-1.5	15/23	23/32	25/38	18/26/31
	-2.5	16/35	18/24	25/46	22/21/32
	-3.5	26/39	28/39	28/94	32/33/46
	-4.5	25/76	28/38	28/49	42/34/36
	-5.5	26/40	29/42	28/47	34/35/37
	-6.5	25/79	25/37	27/111	42/31/49
	-7.5	24/22	25/38	28/108	36/32/49
	-8.5	25/51	25/36	28/105	35/31/48
	-9.5	25/48	26/69	27/51	34/38/35
16	-1.5	19/28	19/26	19/27	24/23/24
	-2.5	17/25	15/20	19/28	21/18/25
	-3.5	20/37	21/29	20/28	28/25/26
	-4.5	19/44	20/36	25/44	28/27/31
	-5.5	17/56	20/40	25/120	31/29/55
	-6.5	17/68	18/39	25/193	34/28/75
	-7.5	19/95	17/41	25/140	41/28/61
	-8.5	19/63	17/37	24/187	32/26/74
	-9.5	26/25	17/24	24/161	13/20/67

CONCLUSIONS

While no final conclusions can be drawn at this time, there are some preliminary conclusions that can be drawn. The moisture content of the soil does have an effect on the in-situ resilient modulus of the subgrade. The per cent saturation may prove to be a major factor when all the data is collected and laboratory testing is completed. The change in the densities of each level of each site bears closer inspection and analysis. The data that has been collected to this point shows that a great deal of change occurs under the pavement sections and a total analysis of the subgrade and the effects of moisture and moisture movement through the soil should provide a clearer picture of subgrade conditions. When the complete data on rainfall is collected, it may show a more pronounced relationship between the rainfall and the resilient modulus of a given soil classification. The data should provide a basis on which to determine the laboratory moisture content and density at which Resilient Modulus testing should take place.

INVESTIGATION AND REMEDIATION OF UNDERMINED HIGHWAY

By
Robert Henthorne*
William Jones*
Larry Rockers*

INTRODUCTION

The Kansas Department of Transportation has known for some time that portions of K-7 highway in Southeast Kansas were undermined (Fig. 1). However, it has only been in recent years that the results of this coal mining have become a major problem for our highway system in this area. In late 1986, traffic, including heavy truck traffic, was diverted from US-69, a major north-south route, to K-7 during construction of a bridge replacement on US-69. Shortly after this, K-7 began to show increased stress in a number of shallow depressions in the wheel paths. It soon became apparent these failures were associated with the old coal mining operations in the area. At this point, a 24 ton load limit was imposed on this section of the road while a condition survey was initiated.

MINING HISTORY

Cave-ins have occurred through the years on this area of K-7 outside of the traveled way. A 1965 investigation by KDOT's Geology Section found that more than 50 cave-ins had occurred within an area 60 feet either side of centerline in a 4 mile section. These cave-ins range from 6 feet to more than 20 feet in diameter, where two or more cave-ins had coalesced.

Major caving along K-7 had generally occurred in the outside quadrants and in the ditches. This is attributed to poor drainage and surface water soaking in and softening the already weakened overburden of the mine. The inside quadrants are generally protected from surface water by the asphalt road surface.

The mining in this area was done between the late 1890's and the 1940's. The coal mined was from the Weir-Pittsburg vein. This vein ranges from 3.2 to 5 feet in thickness, lying 14 to 75 feet below the surface in this two county area. The original mining was done by two major coal mining companies, the Missouri-Kansas-Texas Railroad Company and the St. Louis-San Francisco Railway Company (Abernathy 1944).

When the large companies left, the mines were in a relatively safe condition with sufficient pillars to maintain

*Bureau of Materials and Research, Kansas Department of Transportation

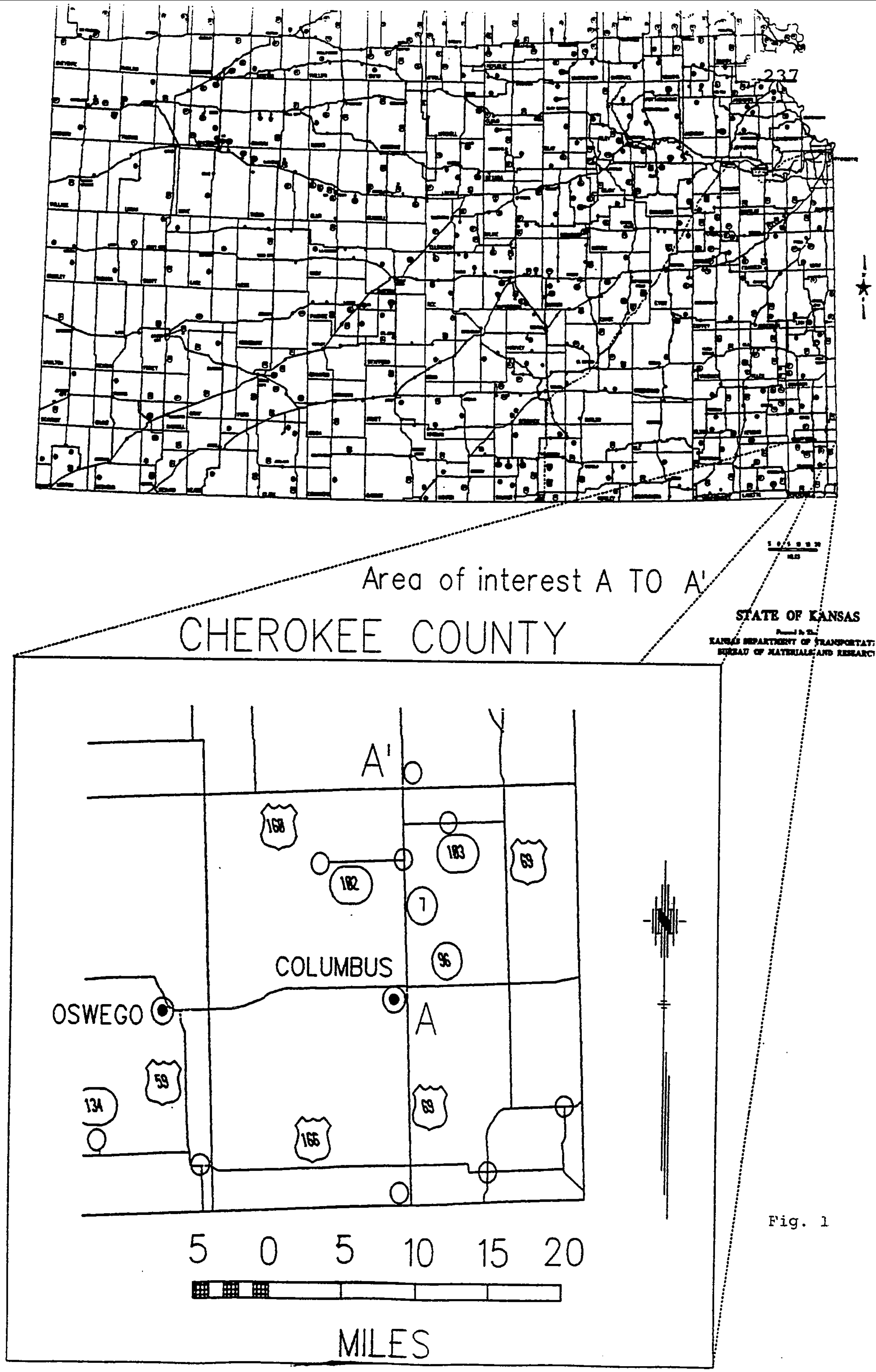

STATE OF KANSAS
Area of interest A TO A'
CHEROKEE COUNTY
A'
168
183
182
69
7
96
COLUMBUS
OSWEGO
A
59
13A
69
166
5 0 5 10 15 20
MILES

Fig. 1

the integrity of the mine. During this era the mining pattern and procedures were much like those shown in Figures 2 and 3 (Abernathy 1944). The coal was mined up to the roadway with properly spaced 6 foot wide crossing drifts beneath the roadway. Subsequent mining took place in times of high coal prices and during the depression of the 1930's. In the latter period, not only were new mines opened, but pillars in the old mines were removed. Old miners and local residents through the years have relayed numerous stories to KDOT personnel about various aspects of the mining operations. They tell of working in this area and that the roadway was extensively undermined. An example was given of a company that operated for four years in an area, where the only remaining coal was under the roadway. Consequently, there are no accurate maps of the areas actually mined out.

MINE INVESTIGATIONS

The history of the area along with the observed distress in the roadway prompted a thorough investigation of the area. This investigation was initiated and carried out over the next 3 1/2 years. The initial investigation was in the areas of depressions resulting from the additional heavy loads and traffic. The mines are no longer accessible from the outside so a drilling program was initiated to determine the mine conditions and the extent of the undermining. Two and one-half inch diameter, continuous auger soundings were made on 25 foot centers alternating between the edge of the pavement and three feet either side of centerline. These soundings were advanced until either the mine opening was detected or the coal zone was encountered. The depth to the floor of the mine or bottom of the coal ranged from 20 to 35 feet. This spacing and logging routine provided the information needed to determine overburden thicknesses, height of mine opening, possible location of walls and pillars and ground water conditions.

Once the soundings were plotted and open areas determined, as shown in fig. 4, the next step was to drill and case a series of 8 1/4 inch holes for securing photographs of the mined out areas (MCHRP Final Report 80-2). Through photography, both still and video, we were better able to determine size, shape and general condition of the mines. Most of our work was done with a Nikon 2002 35mm camera and an 8 mm Yashica Camcorder. The 35mm camera was placed in a 6" clear plastic tube with the pictures being shot through the tube in the beginning. Later an opening was made in the side of the tube as moisture condensation in the tube, scratching and dirt interfered with getting clear pictures. The camcorder was placed in a 6" pvc pipe for protection and hinged for rotating to the picture taking position (Fig. 5-8). Obtaining adequate light for the video camera proved to be a problem. With experience, excellent pictures were obtained in mines with less than a foot of water. No attempt was made to waterproof the camera system for taking pictures under water.

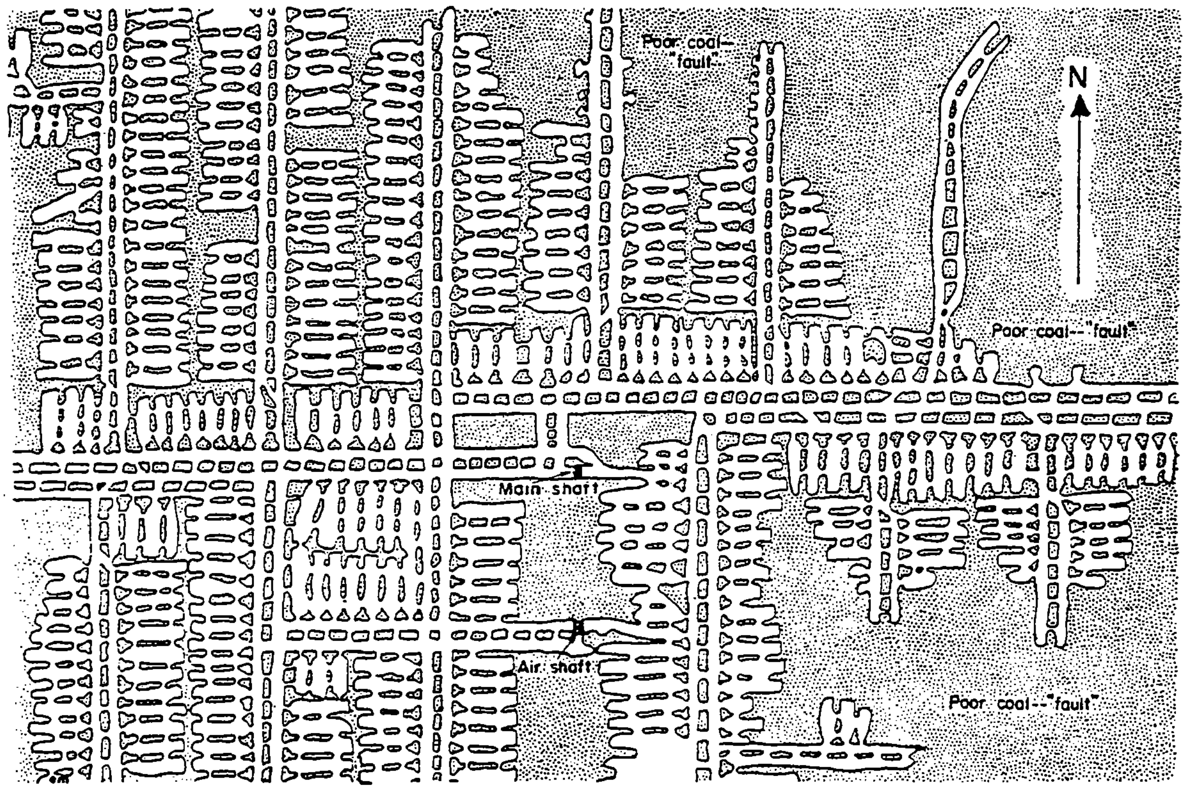


Fig. 2. Section of a mine map showing room and pillar system of mining and ratio of faulty coal areas to mined areas. (Abernathy 1944)

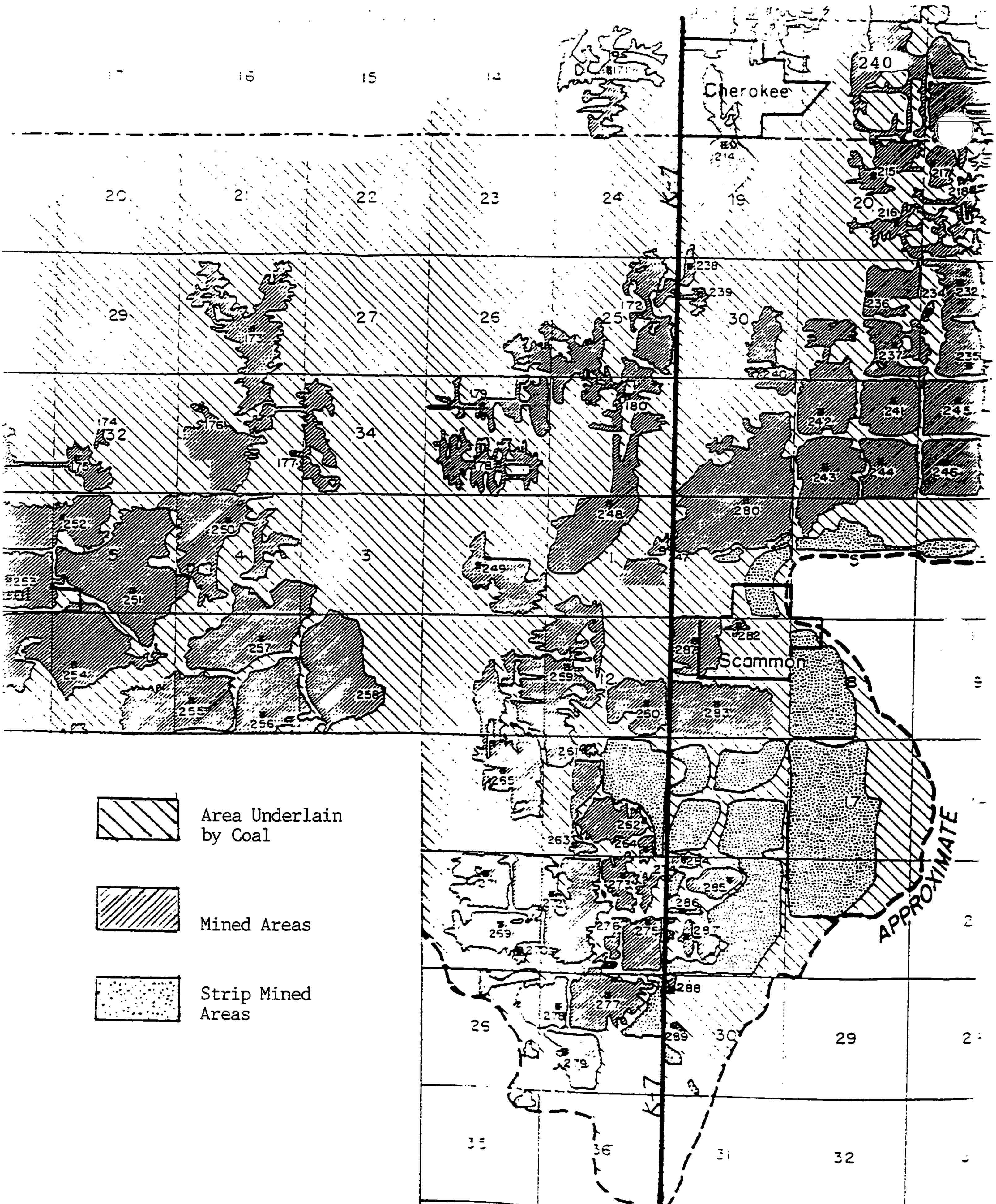


Fig. 3. Modified map of mined areas, Weir-Pittsburg Coal Bed in report area. (Abernathy 1944)

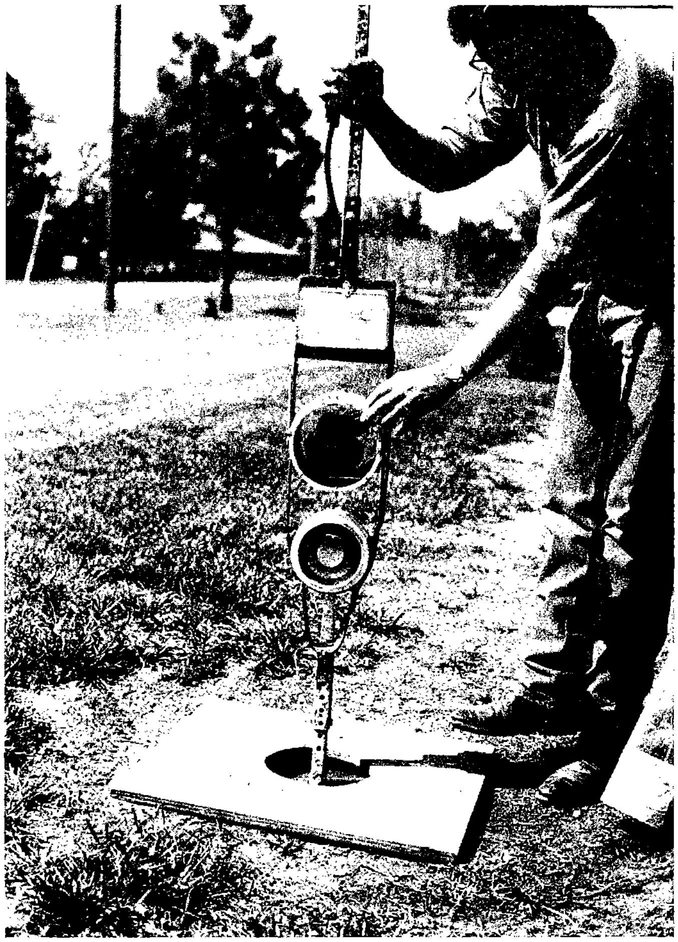


Fig.5 Video Camera in picture taking position in mine.

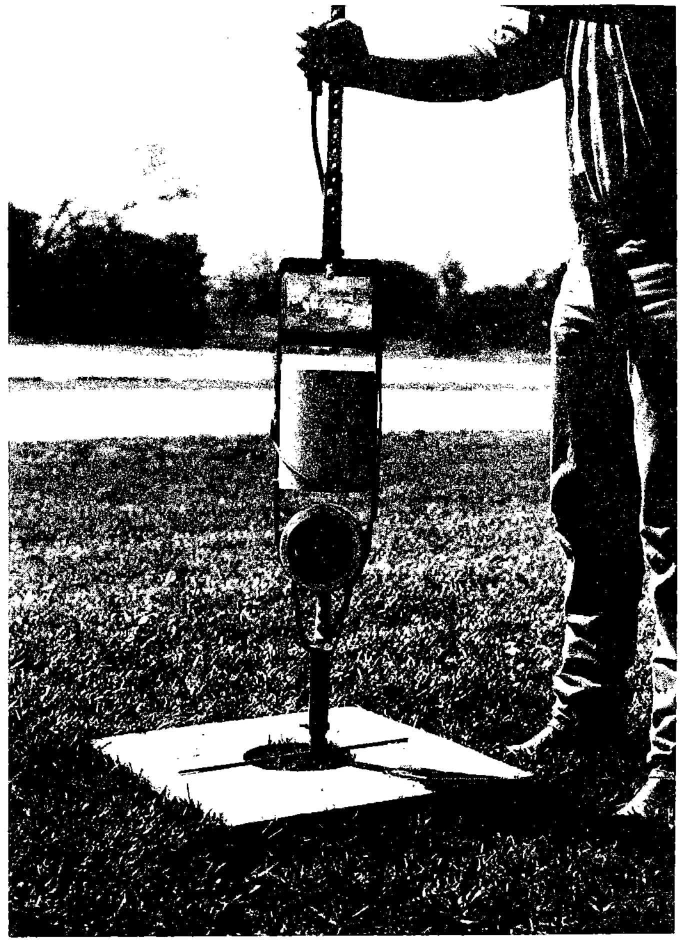


Fig.6 Video Camera in down hole position.

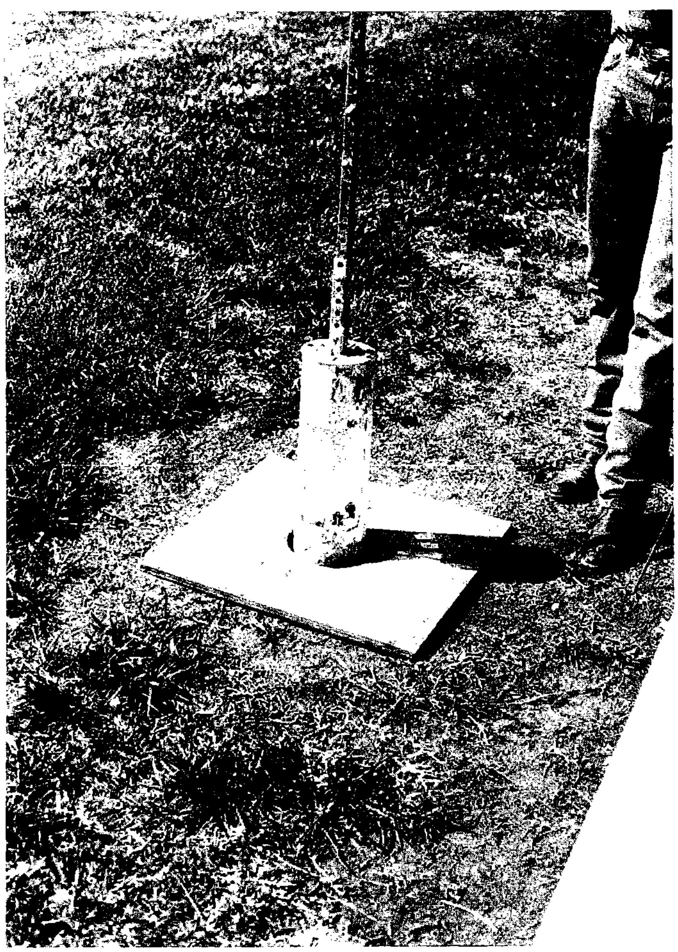


Fig.7 Plastic tube for 35mm

Fig.8 Plastic tube with 35mm

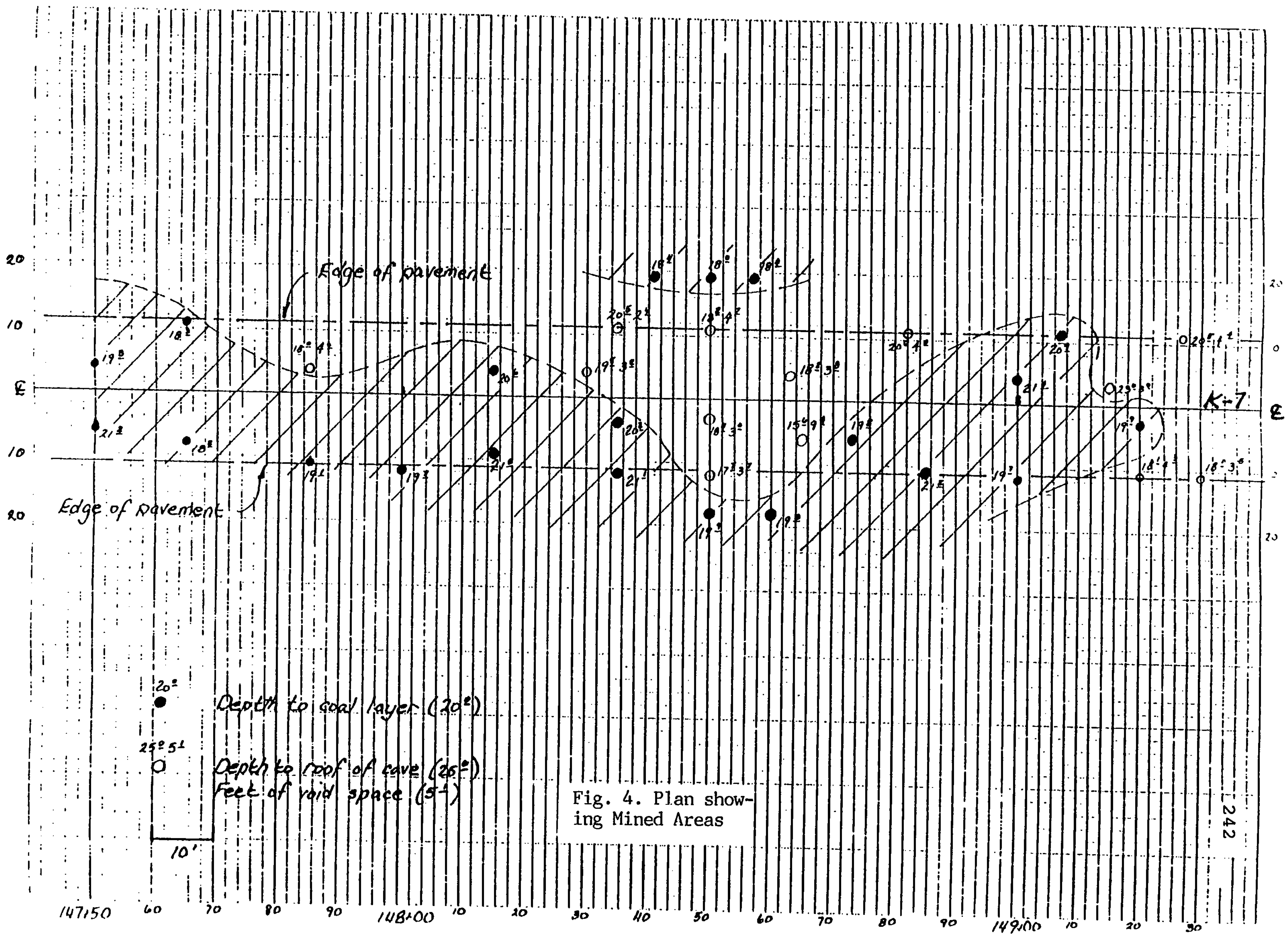


Fig. 4. Plan showing Mined Areas

MINE CONDITIONS

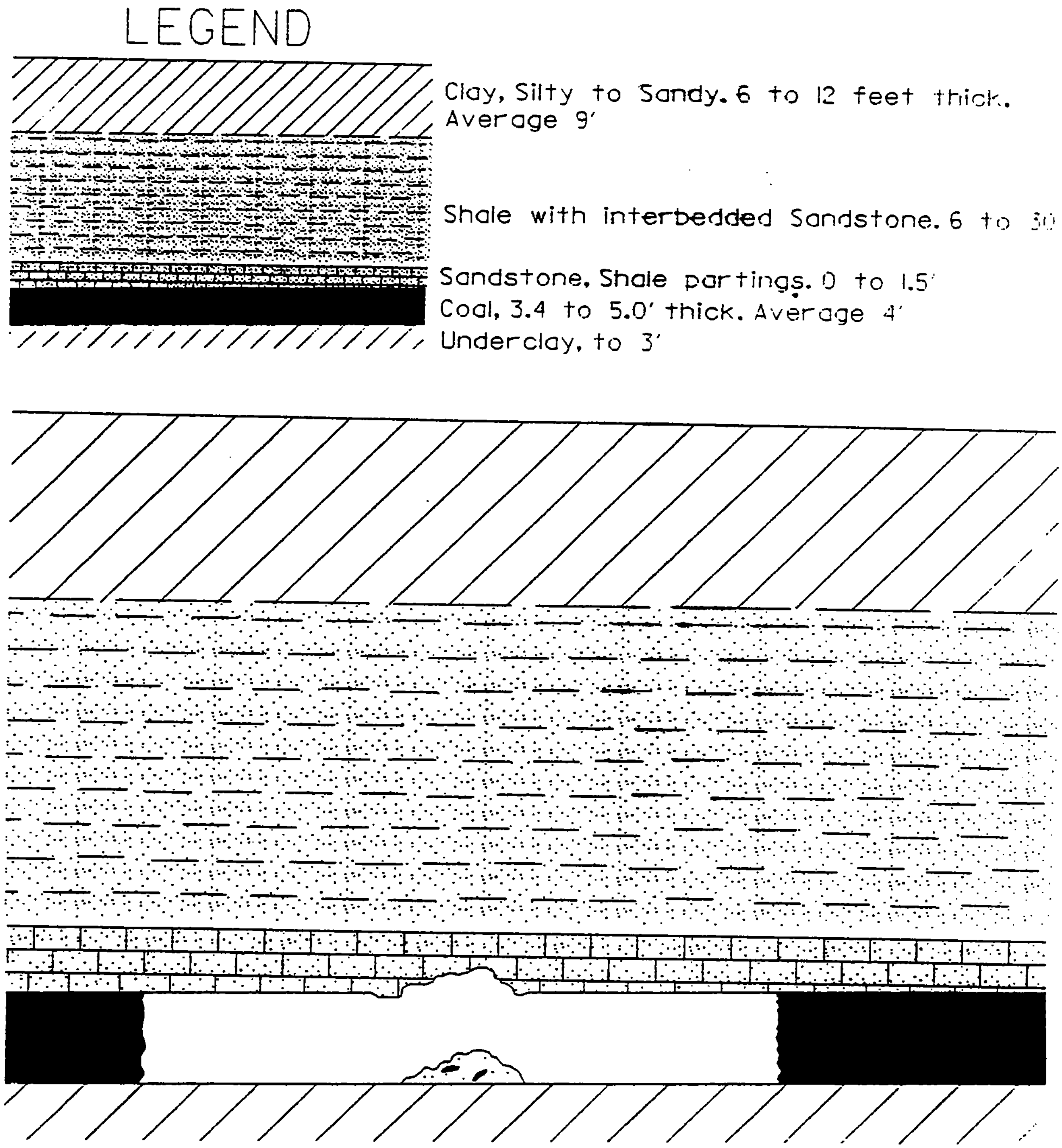
Of the approximately 6 mile project area in which coal was mined by the underground method, not all areas have the same potential for roadway subsidence. There are areas of non-deposition or poor coal quality. In one segment, the coal is more than 40 feet deep and a minimum of subsidence has occurred. Within the 6 mile area, 4 areas totaling approximately 1 1/2 miles were found to have the greatest potential for collapse. Segments within this 1 1/2 miles were then ranked for priority for remediation based on the soundings and pictures.

The soundings and pictures showed the mine had migrated upward through the sandstone and weathered sandy shale and in some cases had nearly reached the mantle. The sequence of this migration is depicted in Figures 9 to 13 (Whittaker and Reddish 1989) and was verified by photography. Standing water and storm water movement through the mines in some locations allows the slumped material to flow and/or be carried away. This results in a larger than normal mine opening and accelerates collapse in some cases. Other areas showed a more competent sandstone roof where the pillars had been replaced by posts that were showing varying degrees of deterioration.

When the mines were operating, it was necessary to find an economical method to drain the mines of water. This was done by developing an outlet into Cherry Creek approximately 2 1/2 miles west of the mined out areas on K-7. This drainage system must still be in operation or all of the mines would be full of water. As the mines were closed and cave-ins developed, the natural surface drainage was and still is being captured and flows into the mines. Most of the surface water drainage on, or adjacent to K-7, is now into the old shafts and cave-ins. Ground water and directly induced surface water flows through and/or stands in these mines. Some of the mines will drain and become relatively dry between periods of precipitation, while others will remain flooded. The depth of the water varies from several inches to nearly full. In those mines in which the water stands the acidity ranges from a ph value from 2.9 to 5.2.

Filling the mines may disrupt the drainage pattern that has been developing over the past 50 to 60 years. It is difficult to forecast the effect of this on the mine drainage system. Drainage openings are being designed for future remediation efforts to ensure the passage of water under the roadway.

After completing the borings and photography in the winter of 1990-91, and reviewing the deteriorating condition of the mines, it became evident that remedial action would have to begin immediately to protect the traveling public and KDOT against the possibility of large liability suits, since the mine condition was known.

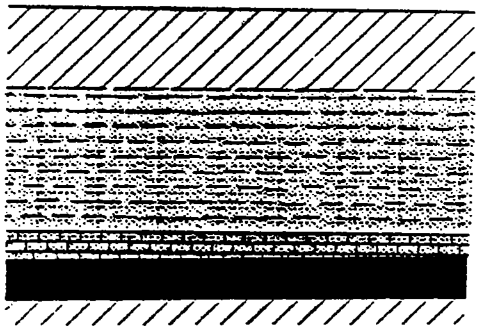


PHASE I

Breakdown of Sandstone Roof

Fig. 9

LEGEND



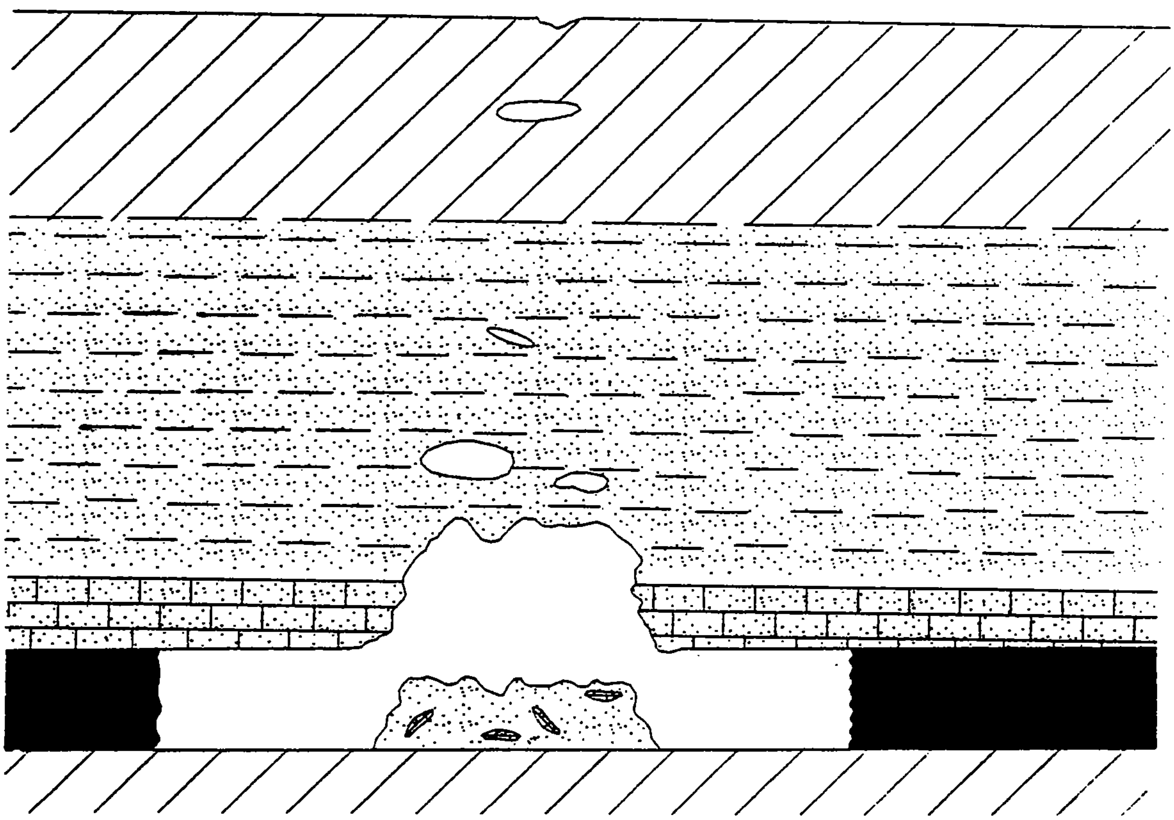
Clay, Silty to Sandy. 6 to 12 feet thick, Average 9'

Shale with interbedded Sandstone. 6 to 30'

Sandstone. Shale partings. 0 to 1.5'

Coal. 3.4 to 5.0' thick. Average 4'

Underclay, to 3'

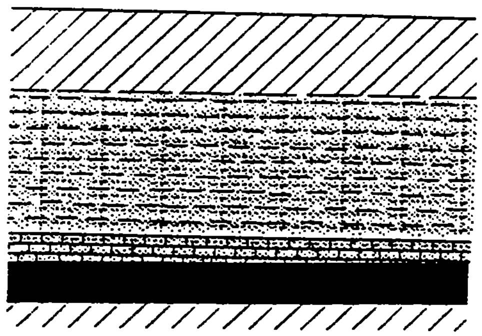


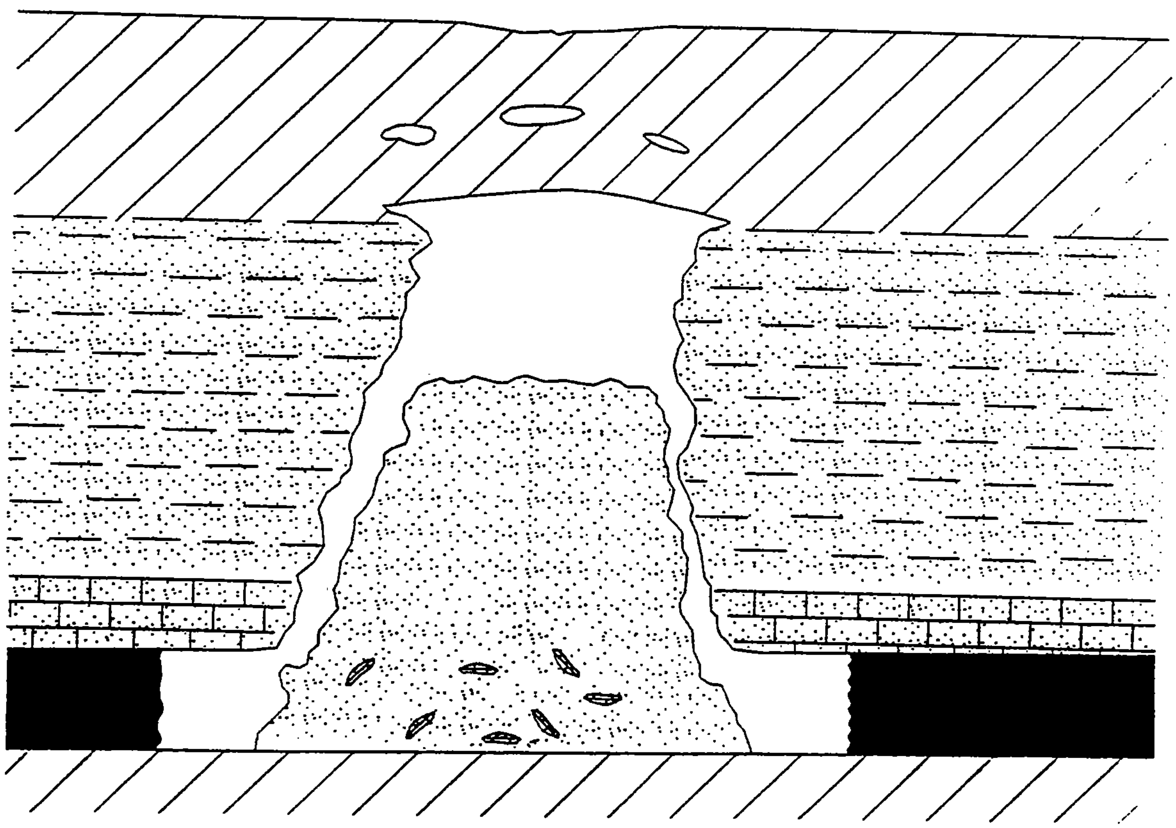
PHASE 2

1. Roof continues to spall
2. Voids & materials separation mainly in bedrock
3. Small indentation appears at surface - 12" to 18"

Fig. 10

LEGEND

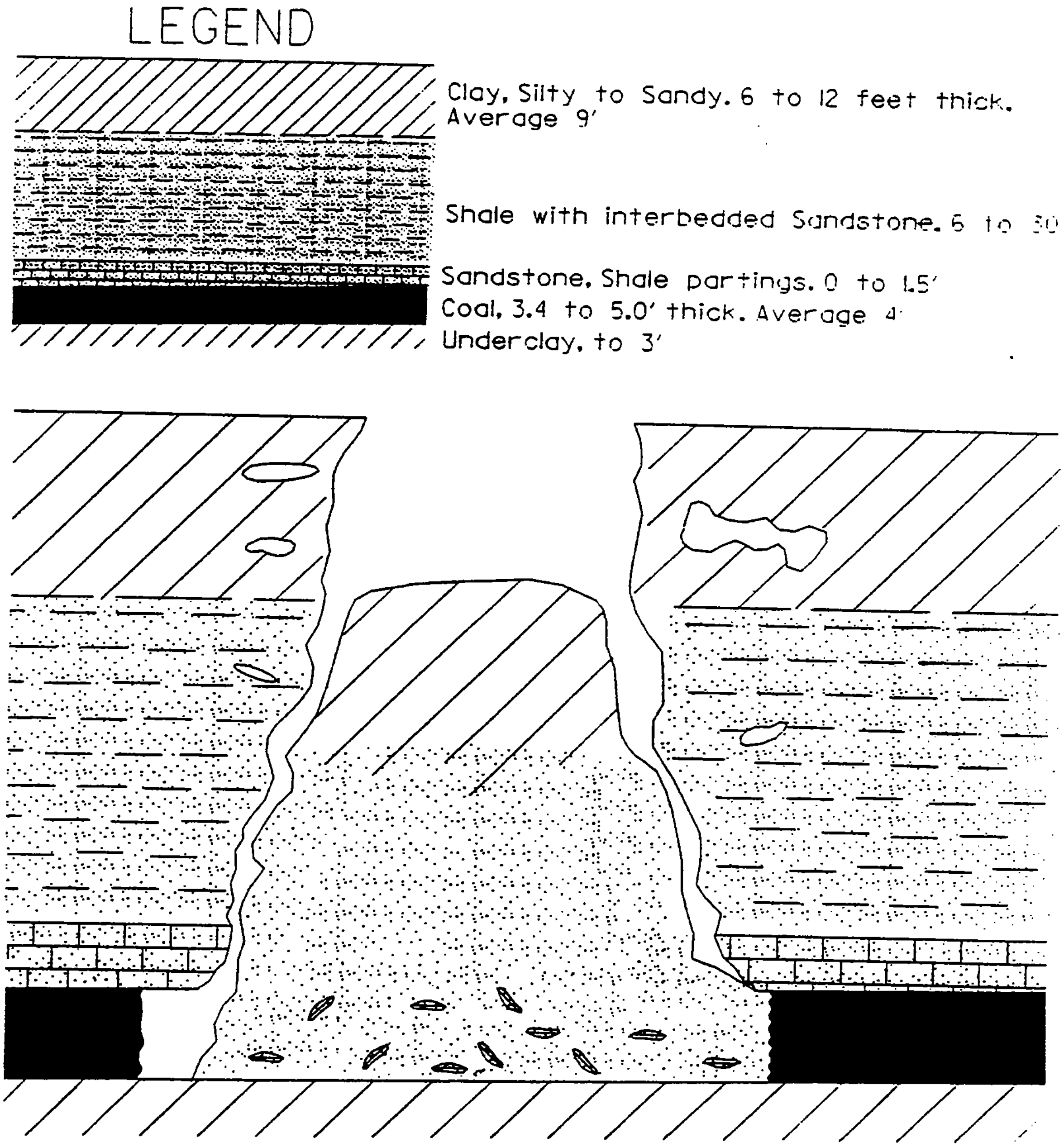




PHASE 3

1. Collapse passes through Mantle-Bedrock contact.
2. Voids & materials separation in mantle and bedrock
3. Surface indentation 5' to 6'.
4. Circular cracking may appear in surfacing.

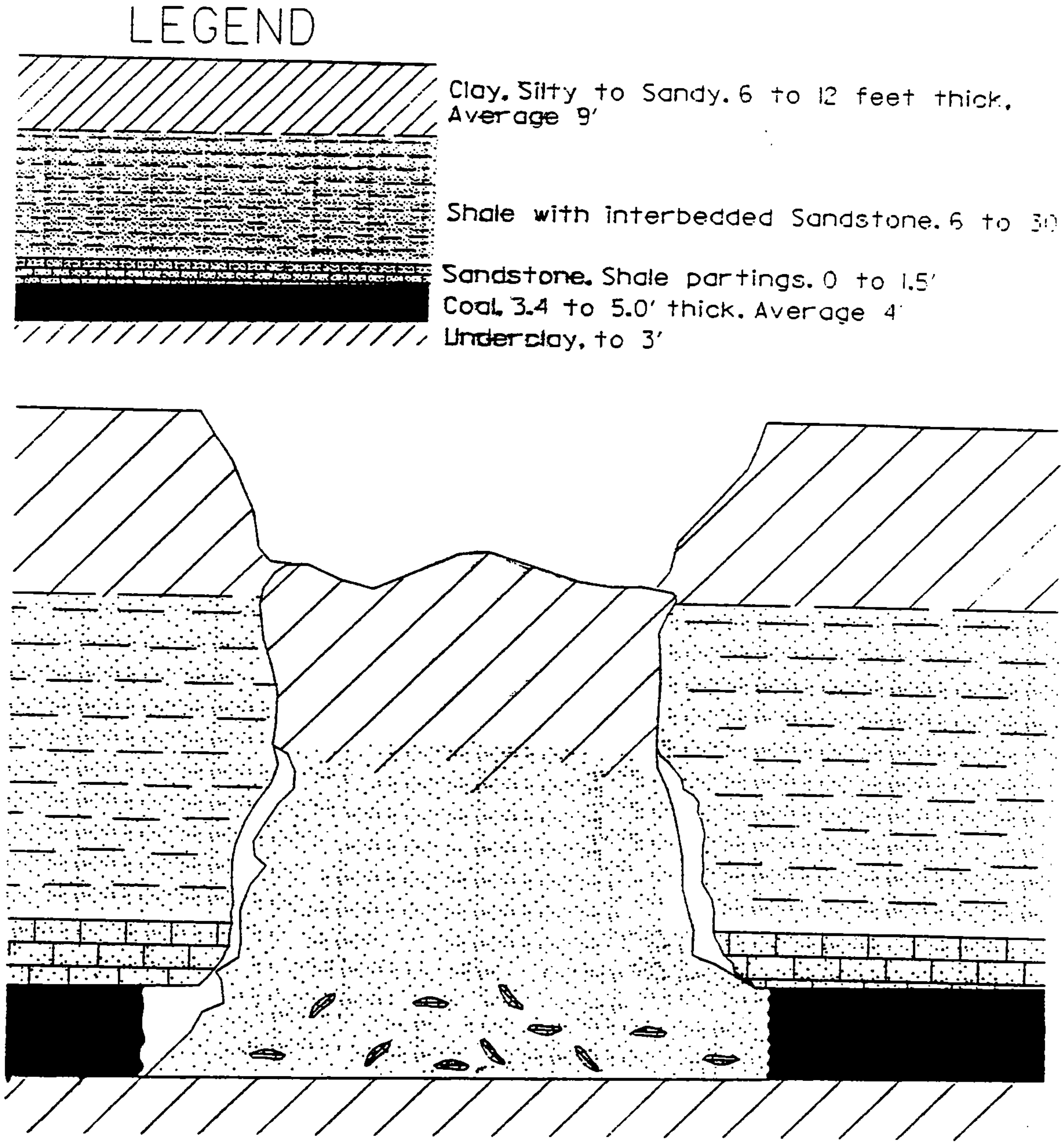
Fig. 11



PHASE 4

Mantle zone collapse - Cylindrical shape

Fig. 12



PHASE 5

Erosional material into cavity

Generally plugs around mantle - bedrock contact

Fig. 13

REMEDIATION

The load limit was reduced from 24 tons to 15 tons while KDOT initiated emergency repairs. A sum of $400,000 was allocated to emergency repairing of the most critical areas.

Collapse and reconstruction as a method of stabilization, although the most positive, was not seriously considered because of problems relating to blasting liability, high acidity of the mine water, mine depth and compaction. Controlling the collapse and manipulating the amount of excavation would have required the purchase of additional right of way, relocation of utilities and resulted in closing of the road during construction.

Grouting of the mined out areas was chosen as the method most likely to achieve the stability required within the time and environmental contraints imposed. Three methods of stabilizing sections of the mine by grouting were considered (Carr 1991; Gray and Myers 1970). The three methods included (1) building grout columns, (2) constructing barrier walls and filling the interior voids and (3) straight volume fly ash fill. Limited previous experience with the straight volume fill method resulted in our rejecting this method.

In May of 1991 an emergency contract was awarded to begin stabilizing the areas of advanced roof migration and greatest instability. A decision was made to use grout columns in an area in which photography showed what appeared to be a 300 foot area of roadway being supported by deteriorating posts. The amount of roof collapse was minimal leaving the mine floor nearly clear of debris. The columns were constructed by pumping a low slump (3 to 4 inches) class B concrete into the mined out areas. The 3" injection pipe was lowered to the bottom of the mine through a 4" bore hole drilled with an air rotary drill. The holes were drilled on 15' centers near the centerline of each lane. The columns were pumped from within. The injection pipe was raised as the pump pressure increased. The quantity of concrete was based on an angle of repose of approximately 20 degrees. Theoretically the columns would be approximately 5 feet in diameter where they come into contact with the roof of the mine, have a 13.5 foot diameter base and each would require approximately 12 cubic yards of concrete (Carr 1991). Thirty-three columns were constructed in this area averaging approximately 14 cubic yards per column. Color pictures taken during construction verified the validity of this construction method. The grout quantities varied with mine conditions and height of opening.

The remaining areas worked under this contract had extensive roof caving, large piles of debris and mine water from several inches deep to nearly filling the mined out area. Because of these conditions, grout columns were not considered for stabilization. Barrier walls with void fill was the chosen method. Barrier walls were constructed in the manner described above for the columns with closer hole spacing. The same concrete mix was also used. The grout holes were drilled on five foot centers in the ditches on either side of the road. The grout was also pumped in the same manner as the grout columns. As the grouting progressed each successive hole was monitored for grout movement. In some cases it was not necessary to pump grout into each hole as it could be seen rising in an adjacent hole. These walls were required to be in place a minimum of 24 hours before void filling could begin.

The void fill holes were drilled on 10 foot centers in the center of each lane. The injection pipe was again lowered to the bottom of the mine and the grout was pumped from within to displace any water and ensure adequate fill. The void fill grout was a 5-2-1 mixture of fine aggregate, type C fly ash and cement with enough water to provide an 8 to 10 inch slump. Grout was pumped in each hole until the pump pressure reached a predetermined pressure or grout came up in to an adjoining injection hole. Many of the holes drilled during the investigative process were left open and provided another means of monitoring grout flow.

Approximately 500 lineal feet of roadway was stabilized using this process at a cost of approximately $128,000. This concluded the work for the initial grouting contract.

Verification coring and drilling showed that in most cases both methods of stabilization were effective. The columns had good roof contact for several feet around the injection hole. Barrier wall construction was adequate to allow the void fill in most instances to fill the mine within a foot of the roof. The interlocking of the coarse aggregate used in the barrier wall grout restricted its flow leaving openings in the wall. This resulted in a less than complete fill in one area, yet still would provide sufficient fill to prevent a major collapse.

Building on what was learned from the initial contract, another emergency contract was awarded in October of 1991. The remaining areas to be remediated would require the barrier wall and void fill methods because of the large amount of roof collapse and rubble. The same basic grouting procedures were used as in the previous contract. The 5-2-1 mix was used on this contract for the barrier walls with a 3 to 4 inch slump. This mix had sufficient flowability with the 3 to 4" slump and resulted in a more positive barrier. The barrier wall holes were shifted to the edge of the pavement because of site conditions and to reduce grout quantities. The void fill grout mix on this project was changed to take advantage of a cheaper bid price and to use an additional waste by-product. The mix consisted of 4 parts fine aggregate, 3 parts boiler slag and 1 part cement. This mix was pumped with a slump of between 8 and 10 inches.

Verification coring at the completion of the contract was very encouraging. Eight soundings, with cores randomly spaced through 800 linear feet of remediation showed good roof contact under the driving lanes.

Unconfined compressive strengths for the various grout mixes were determined from cube samples during the design phase and project cylinders during construction. The 7 day unconfined strengths varied from 3210 psi for the 5-2-1 mix to 370 psi for the 4-3-1 mix.

COST

The work was completed using two contractors over a period of approximately 8 months. Approximately 1318 linear feet of mine was included in the remediation using approximately 2616 cubic yards of grout. Approximately 11,626 linear feet of hole was drilled in the grouting process with approximately 20% being coal holes. Drilling costs were approximately 50% of the actual grouting costs. The cost of remediation averaged about $12.50 per square foot of roadway, including traffic control, hole plugging and cleanup.

CONCLUSIONS

Several sections of K-7 highway in Southeast Kansas were stabilized by grouting. Two methods were used in these relatively shallow mines, construction of free standing grout columns and barrier walls with interior volume fill. The latter proved to be the more effective of the two in the mine environment of our project. However, in a situation where competent roof rock is present and little caving has occurred, the grout column would be very effective and much less expensive.

Plans have been completed and a contract let to further stabilize additional areas of the highway during the 1992 construction season.

References

Abernathy, G.E., Mined Areas of the Weir-Pittsburg Coal Bed: Kansas Geological Survey Bulletin 52, Part 5, October 1944.

Carr, P.H., 1991, Project Proposal from Judy Company Inc., Engineers and Contractors.

Gray, R.E. and Meyers, J.M., Mine Subsidence and Support Methods in Pittsburg Area: Journal of the Soil Mechanics and Foundations Division, Vol. 96, No. SM4, July 1970

__________, Development of a System for Photography of Underground Openings Through Drill Holes: Missouri Cooperative Highway Program, Final Report 80-2, March 1981

Whittaker, B.N. and Reddish, D., Subsidence; Occurrence, Prediction, Control: Elsevier Science Publishing Co., New York, 1989

NEW CORRELATIONS FOR PILES DRIVEN INTO COHESIONLESS SOILS

by A. Amr Darrag[1], C. W. Lovell[2], and A.M.K. Karim[3]

ABSTRACT

Several useful methods have been proposed for predicting axial load capacity of single piles driven into cohesionless soils. Static formulae based on limit equilibrium theories may be considered as one of those. However, misleading predictions are often provided. Although load-deformation solutions utilizing mathematical and/or numerical techniques provide reasonable results, yet they cannot be considered quick and handy tools. Researchers have been working within the last few years to improve empirical prediction methods. However, even the most recent empirical methods neglect or improperly incorporate one or more of four factors; namely: residual stresses due to pile driving, actual parameters mobilized at ultimate soil capacity, mean normal stress levels, and stress history. In this study, available load test data were adjusted for residual stresses, and were then used to develop a new empirical method for predicting ultimate capacity of driven piles in cohesionless soils that takes into account the above factors. Using the adjusted load test data, this new method

---

[1]Assistant Professor of Civil Engineering, Department of Public Works, Cairo University, Giza, Eqypt

[2]Professor of Civil Engineering, Purdue University, West Lafayette, Indiana 47907, USA

[3]Graduate Research Assistant, Purdue University, West Lafayette, Indiana 47907, USA

shows better accuracy and less scatter compared to other available empirical techniques.

INTRODUCTION

Driven piles have been used to support structures for many years. However, accurate prediction of their capacity and behavior, in many cases, is still beyond the state-of-the-art. The problem is particularly complex for piles driven into cohesionless soils, due to the nature of the factors that affect the pile behavior, which are difficult to quantify.

Static formulae based on limit equilibrium theories for rigid-plastic materials have been used to predict pile capacity. These formulae, however, model the system behavior rather poorly and they have many limitations (Darrag, 1987). Therefore, they may be used to extrapolate load test data to other pile dimensions for the same site, while their use on other sites will have limited accuracy. On the other hand, numerical techniques have been developed to overcome the shortcomings of static formulae. Among these techniques are the transfer function approach, the elastic solid approach, and the finite element approach (Vesic, 1977). They not only provide pile capacity prediction, but also predict the load transfer mechanisms which may be important in studying subsequent behavior of the foundation. However, these methods require measurement or assumption of many parameters and the extensive use of computers, and hence they may not be suitable for routine use.

Because of the above considerations, the tendency during the current decade is to develop direct empirical correlations between simple soil parameters and pile capacity in both end-bearing and friction, based on load test results (e.g. Meyerhof, 1976; Coyle and Castello, 1981; Dennis and Olson, 1983; API, 1984; Randolph, 1985). Review of these methods indicates that they may not apply for many situations. Either they may oversimplify, or they may consider the effect of some factors improperly.

The senior author compiled a comprehensive data base of good quality pile load test results from available literature. These data were studied and it was concluded that new correlations, which take into account the effect of several important factors, are needed. The data base was used to develop a new empirical method for tip and shaft capacity prediction for driven piles in cohesionless soils, which accounts for all the important parameters. The method is introduced in terms of easy-to-use charts and tables. Finally, capacities were predicted using the new method, as well as four other methods (Meyerhof, 1975; Coyle and Castello, 1981; API, 1984; and Randolph, 1985). Predictions were compared with available measurements from the data base. This analysis shows that the new method provides the most accurate and consistent prediction technique, compared to the other methods.

DATA BASE

A comprehensive data base of 59 good quality pile load tests was accumulated from available literature. All of the tested piles were driven into cohesionless soils. These tests covered a wide variety of soil relative density; stress history; geographic locations; and pile types, lengths and diameters. Thirty nine tests were performed on instrumented piles, so that separate data on tip and shaft capacities were available. For the other 20 piles, only the total capacity was measured. Data for instrumented piles were used to develop the new correlations, while other data were used to verify predictions obtained by the new method, and to compare them with predictions utilizing other procedures.

The data base, which is described in detail by Darrag (1987), includes the test sites, subsurface conditions, standard penetration test (SPT) and cone penetration test (CPT) data if available, soil parameters, pile materials and dimensions, and the measured shaft and tip capacities for instrumented piles.

Failure Criteria

In order to define a consistent criterion for the ultimate soil capacity (a failure criterion), the authors utilized a modified Chin (1970) procedure, introduced by Darrag and Lovell (1987). The method assumes that the load-movement curve is of hyperbolic shape, where the failure load equals 85% of the asymptotic value of the hyperbola. This procedure was found to provide excellent and consistent interpretation of load tests results for piles in cohesionless soils, in addition to many other advantages (Darrag and Lovell, 1987).

MAJOR FACTORS CONSIDERED FOR DEVELOPING THE NEW CORRELATIONS

Examination of available empirical procedures for capacity prediction for driven piles into cohesionless soils indicated that they are based on parameters that may not properly represent actual conditions in many cases (Darrag, 1987). For example, using the relative depth/diameter ratio (D/d) as a major correlation parameter (Coyle and Castello, 1981) does not reflect actual stress conditions for some cases, e.g. for large-diameter piles. Using visual classification as the major prediction parameter (API, 1984) is often a gross oversimplification. Procedures that depend on the blow count from the SPT results (Meyerhof, 1976) involve the significant scatter inherent in SPT.

Examination of available correlations indicates that they either overlook or oversimplify the effect of four major parameters: residual stresses accumulating during pile driving; actual parameters mobilized at ultimate soil capacity; the mean normal stresses (rather than the effective overburden pressure) at a particular depth; and the stress history of the soil deposit. It was found that these factors have considerable effect on the capacity of driven piles in sand, and consequently they should be incorporated in any empirical technique that predicts such capacity.

1. Effect of Residual Stresses

Residual stresses in driven piles have significant effects on pile-soil interaction. This phenomenon was explained in detail by Holloway, et al. (1975) and Briaud and Tucker (1984). The most important factors affecting magnitude and distribution of residual stresses, as well as a simplified procedure for their prediction, have been recently introduced by Darrag and Lovell (1989). Although the effect of residual stresses on the total pile capacity is zero, these stresses may have a considerable effect on <u>load transfer mechanism</u>, i.e. on the proportions of load transferred through the pile shaft and the pile tip. Therefore, any prediction technique, based on load test results, that separate the tip and shaft loads, should consider residual stresses. Available correlations either neglect this factor, or include it improperly. For example, Coyle and Castello (1981) used the Hunter-Davisson approach (Hunter and Davisson, 1969), which have many limitations (Darrag, 1987). Dennis and Olson (1983) utilized limited residual stress data from a few load tests, and generalized this information to the entire data base, regardless of the conditions and the parameters involved.

For compression loading, residual stresses result in an increase of the shaft load and a decrease of the tip load. To demonstrate this effect, load test results reported by Vesic (1970) and by Tavenas (1971) were adjusted for residual stresses to examine their effect on unit tip and friction capacities (q_o and f_o). Residual stresses were computed using the procedure described by Darrag and Lovell (1989). The measured and adjusted values of q_o and f_o are plotted versus the pile penetration in Figure (1). It is interesting to note that residual stresses account for the observation of limiting unit tip and shaft capacities, as is clear by comparing the measured and the adjusted values in Figure (1). Recent researchers (e.g. Kulhawy, 1984; Randolph, 1985) discount the concept of limiting unit capacities that was well established during the 1970's (e.g. by Meyerhof, 1976 and by Vesic, 1977).

2. <u>Actual Parameters at Ultimate Soil Capacity</u>

Most of the available static formulae or empirical correlations for pile capacity prediction are based either on the initial relative density (or initial angle of shearing resistance) determined from site investigations prior to pile driving or loading. However, these original parameters may have limited importance because driving the pile significantly alters the initial condition. In addition, the stress levels involved during loading, especially below the pile tip, are so large that the angle of shearing resistance may be significantly reduced (Vesic and Clough, 1968; Baligh, 1976). Although these factors have been well recognized, their effect was not well incorporated in available correlations.

The effect of soil densification and prestressing due to pile driving could be introduced via the post driving average coefficient of lateral earth pressure K_s to calculate the shaft friction:

$$f_o = K_s \cdot \overline{\sigma}_v \cdot \tan\delta \qquad (1)$$

where f_o is the average unit friction along the pile shaft, $\overline{\sigma}_v$ is the average effective vertical stress over the pile length, and δ is the pile-soil friction angle. Using the data base, the values of K_s were not found to be a simple function of the coefficient of earth pressure at rest K_o (e.g. refer to Kulhawy, et al., 1983). Instead, it was found that K_s also depends on the degree of pile displacement, stress history, and relative density.

Recommendations were then formulated for K_s, normalized with respect to K_o, accounting for the important variables. Table 1 summarizes such recommendations. For developing Table 1, values of K_s were backcalculated from load tests results using Equation (1), and the ratio K_s/K_o was obtained after computing K_o using the modified Mayne-Kulhawy procedure (Mayne and Kulhawy, 1982 and Schmertmann, 1983).

To calculate the ultimate tip capacity, it is important to recognize that the stress levels may be high enough to reduce the

TABLE 1 Ratio K_s/K_o for Driven Piles with Uniform Cross Section in Cohesionless Soils

		K_s/K_o	
Soil Condition	Range	Average	Coefficient of Variation
A. Partial Displacement Piles			
- Normally consolidated or slightly overconsolidated	1.19-1.29	1.23	3.8%
- Highly overconsolidated	1.75-1.98	1.87	8.7%
B. Displacement Piles			
- Normally consolidated or slightly overconsolidated	1.25-2.00	1.66 (all data)	14.2%
- dense	1.26-1.35	1.30	3.6%
- medium dense	1.43-2.00	1.71	12.6%
- loose	1.66-2.00	1.76	7.2%
- Highly overconsolidated	1.80-2.20	1.96	10.8%

friction angle. The authors examined variation of the angle of shearing resistance (ϕ) with applied stress levels, based on data available in the literature. Details of this analysis are given in Darrag (1987). The main conclusion was that ϕ decreases approximately linearly with the increase of the logarithm of mean normal stress ($\bar{\sigma}_o$), down to an asymptotic value which equals the angle of shearing resistance at constant volume (ϕ_{cv}). After this, ϕ no longer decreases. The stress corresponding to this condition is termed the breakdown stress ($\bar{\sigma}_b$), which usually varies within the range of 50.0 to 150.0 kg/cm^2, depending on inherent soil properties (Vesic and Clough, 1968). Therefore, the angle ϕ at a certain stress level $\bar{\sigma}_o$ could be computed from the following empirical equation:

$$\phi = \phi_{cv} + (\phi_1 - \phi_{cv}) \frac{\log(\frac{\bar{\sigma}_b}{\bar{\sigma}_o})}{\log(\bar{\sigma}_b)} \quad \textit{for } \bar{\sigma}_o \le \bar{\sigma}_b \tag{2.a}$$

$$\sigma = \sigma_{cv} \quad \textit{for } \bar{\sigma}_o > \bar{\sigma}_b \tag{2.b}$$

where ϕ_1 is the angle of shearing resistance under an effective pressure of 1.0 kg/cm^2 (9.81 kN/m^2). The value of ϕ_1 can be obtained from a triaxial or direct shear test at low pressure ranges, applying large values of strains.

Careful examination of equations 2.a and 2.b indicates that the value of ϕ at stress levels close to the $\bar{\sigma}_b$, which is usually the case below the pile tip, is not very sensitive to variations of $\bar{\sigma}_b$ or ϕ_1. The value of ϕ_{cv} is the most critical parameter in these equations. The important conclusion to be drawn is that the initial soil relative density at the tip level has little significance on the unit tip capacity. Using high values of the initial ϕ may seriously overestimate the tip capacity for dense deposits. This is actually a major limitation of many static formulae or empirical procedures that are available for pile capacity estimation.

3. Effective Mean Normal Stress

Static formulae, as well as some empirical methods (e.g. Randolph, 1985), assume that the unit pile capacities, q_o for the end bearing and f_o for the average friction along the shaft, are directly related to the average effective overburden pressure ($\bar{\sigma}_v$). The physics of the problem, however, indicates that these capacities should be functions of the entire stress field in the ground, rather than the vertical pressure only. Experimental evidence (Vesic, 1972; Al-Awkati, 1975) confirmed this concept. However, this idea has not been implemented in design procedures due to the lack of load test data interpreted in terms of $\bar{\sigma}_o$. Average stress conditions for driven piles can be represented by the average effective mean normal stress ($\bar{\sigma}_o$), which is given by the equation:

$$\bar{\sigma}_o = \frac{1+2K_s}{3}\,\bar{\sigma}_v \qquad (3)$$

The available load test data were interpreted in terms of $\bar{\sigma}_o$, and the resulting q_o and f_o (adjusted for residual stresses) were plotted against $\bar{\sigma}_o$ in Figures 2 and 3, respectively (ϕ_f is the angle of shearing resistance at failure stress levels). These plottings clearly indicate good correlations with $\bar{\sigma}_o$. Some of the scatter can be attributed to imperfect estimates of K_o and K_s, which are in turn used to compute $\bar{\sigma}_o$.

Figures 2 and 3 indicate that both q_o and f_o vary linearly with $\bar{\sigma}_o$. Variation of the tip capacity (q_o) with $\bar{\sigma}_o$ can be represented by a bearing capacity factor N_σ defined such that:

$$q_o = N_\sigma \cdot \bar{\sigma}_{ot} \qquad (4)$$

where $\bar{\sigma}_{ot}$ is the mean normal stress at tip.

Variation of the average unit shaft friction f_o with $\bar{\sigma}_o$ can be represented by a friction factor β', defined such that:

$$f_o = \beta' \cdot \bar{\sigma}_o \qquad (5)$$

More details regarding N_σ and β' will follow.

4. Stress History

It is expected that stress history of the cohesionless soil deposits affects the unit capacities for driven piles, q_o and f_o, for several reasons. First, the coefficient of lateral earth pressure at rest increases with increasing overconsolidation ratio (OCR), as was quantified by Mayne and Kulhawy (1982). Therefore, friction forces along the pile shaft should increase with increasing OCR. In addition, the increase of K_o (and consequently K_s) should result in an increase of the mean normal stress $\bar{\sigma}_o$

(Equation 3), which directly increases both q_o and f_o (Equations 4 and 5). Furthermore, the compressibility of the cohesionless deposit is reduced by prestressing (for reasons other than pile driving). Vesic (1972) indicated that the soil compressibility is a very important factor in determining the unit tip capacity, which is not considered in any static or empirical procedure, except for the theory of expansion of cavities in a soil mass. For the above reasons, overconsolidated deposits would produce higher values of q_o and f_o than normally consolidated deposits with the same intrinsic strength parameters (ϕ and ϕ_{cv}). This concept was confirmed by analysis of the available data base, as will be shown in the following section.

Combined Effect of Actual Conditions, Mean Normal Stress and Stress History

a. Tip Capacity

The combined effect of neglecting the above mentioned factors can be inferred from Figure 4. An attempt was made to correlate the backfigured bearing capacity factor N_q, obtained in terms of the effective overburden pressure, with the initial angle of shearing resistance (ϕ) obtained from site investigations prior to pile driving, for all available load test data. The data points shown in Figure 4 indicate that the scatter involved is tremendous, even if the effect of stress history is included. Thus, it is concluded that no reliable correlation can be obtained in terms of the parameters used, although these parameters are frequently employed in well known correlations and static formulae.

Alternatively, the bearing capacity factor N_σ (defined in Equation 4) can be very successfully correlated with the angle of shearing resistance at failure stress levels (ϕ_f), provided that the data are subdivided according to stress history, as shown in Figure 5. The data shown on Figure 5 indicate that N_σ is uniquely related to the angle ϕ_f and the stress history. Figure 4 and 5 confirm the concepts discussed earlier, regarding the factors not considered in static and empirical prediction methods, in the following manner:

1. A good correlation could be obtained after adjusting the data for residual stress effect (Figure 5).
2. A good correlation could be obtained for the bearing capacity factor N_σ in terms of the mean normal stress (Figure 5), as opposed to the factor N_q in terms of the effective vertical overburden pressure.
3. A good correlation could be obtained in terms of the angle of shearing resistance at failure stress levels (Figure 5), as opposed to the initial angle before driving (Figure 4).
4. Stress history has a pronounced effect on the tip capacity of driven piles in cohesionless soils. It was possible to describe the effect of stress history in terms of three main groups; normally consolidated soils (OCR = 1.0), slightly overconsolidated soils ($1.0 < OCR \leq 4.0$) and highly overconsolidated soils (OCR > 4.0). It is clear from Figure 5 that prestressing may significantly increase the tip bearing capacity. It is also indicated that the effect of stress

history is more pronounced for low values of ϕ_f. The explanation is that for such cases, where N_σ values are relatively low (relatively high compressibility), the compressibility is much reduced by prestressing, as confirmed by tests performed by Leonards, et al, (1986). Although the stress history factor proved to be very important, it has not been included in any analytical or empirical procedure for this purpose.

b. Average Unit Shaft Capacity

To calculate the average unit shaft capacity (f_o), equation (1) can be rewritten in the form:

$$f_o = \beta \, \overline{\sigma}_v \tag{6}$$

where $$\beta = K_s \tan \delta \tag{7}$$

Several studies (e.g. Yazdanbod, et al., 1984) indicated that f_o varies linearly with $\overline{\sigma}_v$(i.e. β is a constant value). This concept was examined using the backfigured parameters for the available load test data. Values of the coefficient β are plotted against $\overline{\sigma}_v$ for normally consolidated sands in Figure 6 and relatively large scatter can be observed. This indicates that the concept represented by Equation 6 is not totally valid. On the other hand, when the coefficient β', defined earlier in Equation 5 in terms of the average effective mean normal stress over the pile shaft, was used, the scatter was much reduced, as shown in Figure 7. The remaining scatter may be explained in terms of the uncertainties in test measurements and in determining the coefficient K_s which is used in calculating the value of $\overline{\sigma}_o$.

Stress history affects values of β'. Analysis of the data base resulted in the recommended values of β' presented in Table 2.

TABLE 2 Variation of β' with Stress History

Stress History	Average Value of β'	Coefficient of Variation (%)
Normally Consolidated	0.44	9.0
Slightly Overconsolidated	0.54	20.0
Highly Overconsolidated	0.68	29.0

As indicated from Table 2, the scatter associated with overconsolidated sands is relatively large. This may be attributed to uncertainties in estimating the overconsolidation ratio and the coefficient of lateral earth pressure K_s. Also, few data points were available for these conditions and more data are needed to confirm the quantitative aspects. However, from a qualitative point of view, it is clear that overconsolidation may significantly increase the pile shaft capacity, and this should be taken into account in the design.

NEW CORRELATIONS FOR PILE CAPACITY PREDICTION

New empirical correlations that include effects of residual stresses, actual soil conditions near failure, mean normal stress, and stress history will enhance the accuracy and reduce the scatter in pile capacity predictions. The available data base was used to develop such correlations based on the conclusions and parameters mentioned earlier.

1. Correlations for Unit Tip Capacity

The prediction procedure for the unit tip capacity (q_o) involves the following steps:

1. Determine the stress history condition of the soil using any appropriate procedure (e.g. using pressuremeter, dilatometer, etc.)
2. Calculate the K_o coefficient using the modified Mayne-Kulhawy procedure (Mayne and Kulhawy, 1982 and Schmertmann, 1983) by the following equation:

$$K_o = (1-\sin\phi') \cdot OCR^{0.8\sin\phi'} \tag{8}$$

where ϕ' is the drained angle of shearing resistance.

3. Select an appropriate value of K_s/K_o based on the guidelines given in Table 1. Then, compute K_s.
4. Compute the effective mean normal stress at tip ($\overline{\sigma}_{ot}$) using Equation 3 in terms of K_s and the effective overburden pressure at tip.
5. As a first approximation for ϕ_f, assume it to be equal to the angle of shearing resistance at constant volume (ϕ_{cv}), since the ultimate tip pressure is usually close to the soil breakdown stress (Darrag, 1987).
6. Enter Figure 5 with the first chosen value of ϕ_f and the appropriate stress history conditions, and evaluate N_σ.
7. Compute a first approximation of q_o using Equation 4. If q_o is substantially lower than the breakdown stress, compute the appropriate value of ϕ_f using Equation 2 and the first approximation of q_o stress value.
8. Enter Figure 5 with the new ϕ_f, obtain N_σ and compute a second value for q_o. Usually one iteration is enough for a ϕ_f-q_o selection. The iterative procedure is not required if q_o was found to be very close to the breakdown stress. It should be noted that the q_o is limited by the crushing strength of the soil grains.

2. Correlations for Average Unit Shaft Capacity

In order to obtain the average unit shaft capacity (f_o), the same steps 1 through 4 prescribed for q_o prediction should be followed. Next, determine the appropriate value of β' using Table 2. The value of f_o can be obtained using Equation 5, in which $\overline{\sigma}_o$ is the average mean normal stress along the pile shaft.

Darrag (1987) indicated that the capacity prediction is not very sensitive to the assumed value of K_s/K_o, so long as reasonable values are chosen based on the recommendations given in Table 1. The procedure outlined in the above steps for q_o and f_o predictions has the following advantages over available empirical techniques:

1. Values of q_o and f_o are appropriately adjusted for residual stress effects.
2. Correlations take into account the stress history, the effective mean normal stress, and the angle of shearing

resistance at failure stress levels.

3. The bearing capacity factor N_σ is not extremely sensitive to slight variations in ϕ_f, unlike N_q which is very sensitive to slight variations of ϕ, especially for dense soils.

4. The factor β' is independent of the initial soil density along the pile shaft.

5. It is much easier to determine the angle of shearing resistance at constant volume (ϕ_{cv}), and consequently the angle ϕ_f, than the in-situ angle ϕ or soil relative density. The angle ϕ_{cv} is independent of soil density; therefore, it is not necessary to obtain undisturbed cohesionless soil samples or to use any correlations with in-situ tests.

<u>Illustrative Example</u>:

A precast concrete pile is to be driven into a normally consolidated loose sand. It is required to predict the static pile capacity, provided the following data:

$K_o = 0.42$ $\quad\quad$ $\phi_{cv} = 33°$, at tip $\quad$ $\sigma'_v = 141$ KN/m^2

pile length = 12.5 m $\quad\quad$ cross section area = 0.086 m^2

surface area of the pile shaft = 13 m^2

<u>Solution</u>

From Table 1, $K_s/K_o = 1.76$ (loose sands displacement piles)

$$K_s = 1.76 * 0.42 = 0.74$$

at tip $\quad$ $$\overline{\sigma}_{oc} = \frac{(1 + 2 * 0.74)}{3} * 141 = 116.6\,KN/m^2$$

From Fig. 5, assuming $\phi_f = \phi_{cv} = 33°$, for normally consolidated sand $N_\sigma = 74$, hence, $q_o = 8.6$ MN/m^2

This value is close to the sand breakdown stress, hence the assumption of $\phi_f = \phi_{cv}$ is reasonable.

From Table 2 the value of β' for normally consolidated sand = 0.44

$$f_o = 0.44 * \frac{116.6}{2} = 25.65\ KN/m^2$$

Therefore,

$$Q_p = A_p\ q_o = 0.086 * 8.6 = 0.74\ MN = 740\ KN$$

$$Q_s = A_s\ f_s = 13 * 25.65 = 333.5\ KN$$

$$Q_t = Q_p + Q_s = 1073.5\ KN$$

OTHER USEFUL CORRELATIONS

Some useful ratios such as N_σ/K_s and f_o/q_o (%) were evaluated using the available data base. These ratios are helpful in computing the shaft capacity if data are scarce, and in checking predictions made using other methods or results of load tests on instrumented piles. Ranges of N_σ/K_s and f_o/q_o (%) are given in Tables 3 and 4, respectively. It should be noted that all these parameters are corrected for residual stress effects.

TABLE 3 Ratio N_σ/K_s for Driven Piles in Cohesionless Soils

Driving Status	Stress History	$(N_\sigma/K_s)_{av.}$	Coefficient of Variation	Range
Partial displacement piles	- N.C.	145	13.0%	124-160
	- Slightly O.C.	insufficient data		
	- Highly O.C.	28.5	12.4%	26-31
Displacement piles	- N.C.	106	31.5%	47-147
	- Slightly O.C.	87	20.7%	49-108
	- Highly O.C.	39	50.8%	25-53

TABLE 4 Percentage f_o/q_o (%) for Driven Piles in Cohesionless Soils

Driving Status	Stress History	$(f_o/q_o)_{av.}$ (%)	Coefficient of Variation	Range
Partial displacement piles	- N.C.	0.24%	15.8%	1.21%-0.28%
	- Slightly O.C.	insufficient data		
	- Highly O.C.	0.385%	5.5%	0.37%-0.40%
Displacement piles	- N.C. (dense sand, short piles)	0.30%	16.4%	0.22%-0.36%
	- N.C. (loose sands, long piles)	0.80%	25.4%	0.54%-1.10%
	- Slightly O.C.	0.34%	41.8%	0.2%-0.6%
	- Highly O.C.	0.30%	38.9%	0.16%-0.37%

The relationship between backcalculated q_o values from load tests (corrected for residual stresses) and the measured q_c values from in-situ static cone penetration tests (CPT) was also examined. The following correlation was obtained:

$$q_o = 0.60\ q_c \tag{9}$$

The r^2 value was 0.636 for the above correlation.

COMPARATIVE ANALYSIS

In this section, prediction of the capacity of driven piles in cohesionless soils by five different empirical methods is compared with actual measurements. These methods are: Meyerhof (1976), Coyle and Castello (1981), the American Petroleum Institute (API, 1984), Randolph (1985), and the method proposed in this paper.

Figure 8 shows these comparisons between predictions and actual measurements. These data clearly indicate the superiority of the suggested procedure over other methods. This is mainly because the new method takes the combined effect of all relevant factors into account.

While both the Meyerhof and Coyle-Castello methods gave predictions which are scattered around measured values, the API method tends to underestimate the capacity. The Randolph method tends to overestimate it. The API method provides limiting unit capacities, which for many cases result in a serious underprediction, especially for long, large diameter piles. The Randolph procedure mainly overestimates the shaft capacity due to the assumed value of N_q/K_s = 50 (Randolph, 1985). Available data show that this ratio may be much higher. The Meyerhof method includes the scatter involved in the SPT procedure. The Coyle-Castello procedure was based on relative pile embedment (length/diameter), rather than considering actual stress conditions. This may result in serious errors for either very short or very long piles, or for the case of large-diameter piles.

On the other hand, all methods, except the new procedure, resulted in serious errors for piles driven into very dense soils. Generally, they tend to overestimate the tip capacity for such conditions because they utilize the initial relative density or the initial angle of shearing resistance. This angle could be much higher, for dense soils, than the actual value at high stress levels below the pile tip, as was shown earlier. This is also one of the serious limitations of static formulae based on limit equilibrium theories.

Conclusions

Available empirical correlations for prediction of capacity of driven piles in cohesionless soils may result in significant errors for certain conditions. This is mainly due to neglecting, or improperly including, four key factors. These factors are residual stresses due to pile driving, actual soil parameters prior to failure, mean normal stress levels, and stress history. In this study, available load test data were appropriately interpreted in terms of the above factors, and other relevant parameters. Consequently, a new empirical procedure for pile capacity prediction was developed. Comparison of the predictions using this method, as well as four other methods, to actual measurements indicated the superiority of the new procedure. More data are needed to improve this procedure for certain cases, e.g. for piles driven into originally overconsolidated sand deposits.

Acknowledgements

The financial support for this research was provided by the Indiana Department of Highways and the Federal Highway Administration. The research was administered through the Joint Highway Research Project, Purdue University, West Lafayette, Indiana.

References

1. Al-Awkati, Z. (1975) "On Problems of Soil Bearing Capacity at Depth", Ph.D. Dissertation, Duke University, Durham, NC.
2. American Petroleum Institute (1984), "API Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms", API, Dallas, Texas.
3. Baligh, M. M. (1976) "Cavity Expansion in Sands with Curved Envelopes", Journ. Geotech. Eng. Div., ASCE, Vol. 102, No. GT11, pp. 1131-1146.
4. Briaud, J. L. and Tucker, L. M. (1984) "Piles in Sand: A Method Including Residual Stresses", Journ. Geotech. Eng. Div., ASCE, Vol. 110, November, pp. 1666-1680.
5. Chin, F. K. (1970) "Estimation of the Ultimate Load of Piles Not Carried to Failure", Proc., 2nd Southeast Asian Conf. on Soil Eng., pp. 81-90.
6. Coyle, H. M. and Castello, R. R. (1981) "New Design Correlations for Piles in Sand", Journ. Geotech. Eng. Div., ASCE, Vol. 107, No. GT7, pp. 965-986.
7. Darrag, A. A. (1987) "Capacity of Driven Piles in Cohesionless Soils, Including Residual Stresses", Ph.D. Dissertation, School of Civil Engineering, Purdue University, West Lafayette, Indiana.
8. Darrag, A. A. and Lovell, C. W. (1987) "Recommendations for Quick Pile Axial Load Testing and Its Interpretation for Highway Bridge Design", Proc. 23rd Symp. on Eng. Geology and Soils Eng., Utah State Univ., Logan, Utah, April, pp. 355-373.
9. Darrag, A. A. and Lovell, C. W. (1988) "A Simplified Procedure for Predicting Residual Stresses for Piles", Proc., XII Int. Conf. Soil Mech. and Found. Eng., Rio de Janeiro, Brazil.

10. Dennis, N. D. and Olson, R. E. (1983) "Axial Capacity of Steel Pipe Piles in Sand", Geotechnical Practice in Offshore Engineering, ASCE, April, pp. 389-401.

11. Holloway, D. M., Clough, G. W. and Vesic, A. S. (1975) "The Mechanics of Pile-Soil Interaction in Cohesionless Soils", U.S. Army Eng. Waterways Exp. Sta., Contract Report S-75-5.

12. Hunter, A. H. and Davisson, M. T. (1969) "Measurements of Pile Load Transfer", Performance of Deep Foundations, ASTM, STP 444, pp. 106-117.

13. Kulhawy, F. H. (1984) "Limiting Tip and Side Resistance: Fact or Fallacy?", Symposium Proceedings on the Analysis and Design of Pile Foundations, ASCE, October, pp. 80-98.

14. Kulhawy, F. H., Troutman, C. H., Beech, J. F., O'Rourke, T. D., McGuire, W., Wood, W. A., and Capano, C. (1983) "Transmission Line Structure Foundations for Uplift-Compression Loading", Report EL-2870, Electric Power Research Institute, Palo Alto, California, February, 412 p.

15. Leonards, G. A. et al. (1986), Discussion of the paper "Dynamic Penetration Resistance and the Prediction of the Compressibility of a Fine Grained Sand - A Laboratory Study", Geotechnique, Vol. 38, No. 2, pp. 275-279.

16. Mayne, P. W. and Kulhawy, F. H. (1982) "K_o-OCR Relationships in Soil", Journ. Geotech. Eng. Div., ASCE, Vol. 108, No. GT6, June, pp. 851-872.

17. Meyerhof, G. G. (1976) "Bearing Capacity and Settlement of Pile Foundations", Journ. Geotech. Eng. Div., ASCE, Vol. 102, No. GT3, March, pp. 196-228.

18. Randolph, M. F. (1985) "Capacity of Piles Driven into Dense Sands", Cambridge University, Engineering Department, Soils TR171, 32 p.

19. Schmertmann, J. H. (1983) "Revised Procedure for Calculating K_o and OCR from DMT's with I.D. > 1.2 and Which Incorporates the Penetration Force Measurements to Permit Calculating the Plane Strain Friction Angle", DMT Workshop, Gainesville, Florida.

20. Tavenas, F. A. (1971) "Load Test Results on Friction Piles in Sand", Canadian Geotechnical Journal, Vol. 8, pp. 7-22.

21. Vesic, A. S. (1970) "Tests on Instrumented Piles, Ogeechee River Site", Journ. Soil Mech. Found. Div., ASCE, Vol. 96, No. SM2, March, pp. 561-584.

22. Vesic, A. S. (1972) "Expansion of Cavities in Infinite Soil Mass", Journ. Soil Mech. and Found. Div., ASCE, Vol. 98, No. SM3, March, pp. 265-290.

23. Vesic, A. S. (1977) "Design of Pile Foundations", TRB, No. 42.

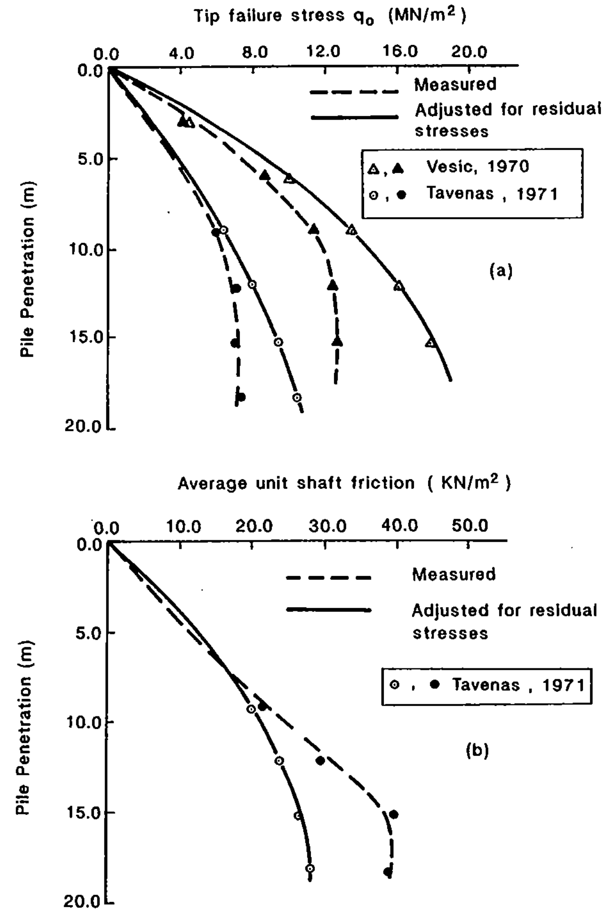
24. Vesic, A. S. and Clough, G. W. (1968) "Behavior of Granular Materials Under High Stresses", Journ. Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM3, May, pp. 661-688.

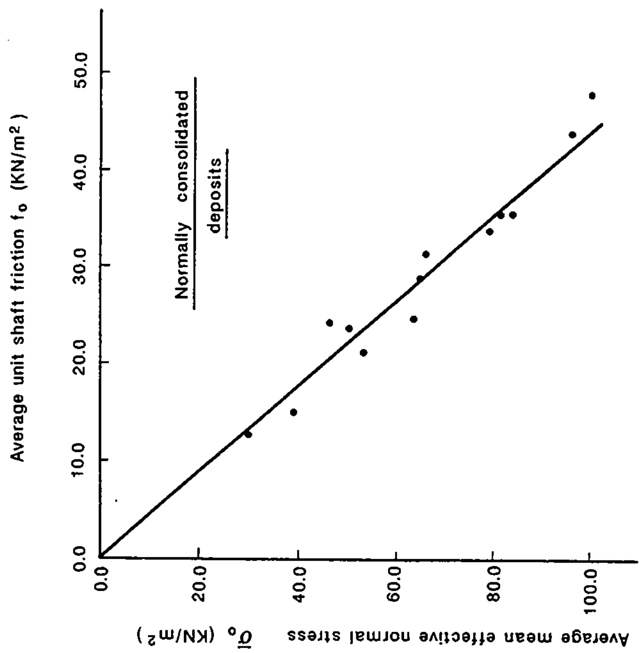
25. Yazdanbod, A., O'Neill, M. W. and Aurora, R. P. (1984) "Phenomenological Study of Model Piles in Sand", Geotechnical Testing Journal, GTJODJ, Vol. 7, No. 3, September, pp. 135-144.

LIST OF SYMBOLS

A_p	=	area of pile cross section
A_s	=	surface area of pile shaft
CPT	=	cone penetrometer test
d	=	pile diameter
D	=	pile penetration
f_o	=	average unit shaft friction
K_o	=	coefficient of lateral earth pressure at rest
K_s	=	coefficient of lateral pressure after driving
N_q	=	bearing capacity factor
N_σ	=	bearing capacity factor based on mean normal stress
N.C.	=	normally consolidated
O.C.	=	overconsolidated
OCR	=	overconsolidation ratio
OCR_{max}	=	maximum overconsolidation ratio to which the soil has ever been subjected
Q_p	=	pile tip capacity
Q_s	=	pile shaft capacity
Q_t	=	pile total capacity
q_c	=	cone stress measured by CPT
q_o	=	unit tip capacity
q_{res}	=	unit residual stress beneath pile tip
SPT	=	standard penetration test
β	=	skin friction coefficient in terms of overburden pressure
β'	=	skin friction coefficient in terms of mean normal stress
δ	=	friction angle between pile and soil
$\bar{\sigma}_b$	=	breakdown stress
$\bar{\sigma}_o$	=	average effective mean normal stress
$\bar{\sigma}_{ot}$	=	effective mean normal stress at pile tip
$\bar{\sigma}_v$	=	average effective overburden pressure over the pile length
σ'_v	=	effective overburden pressure
ϕ	=	angle of shearing resistance
ϕ'	=	drained angle of shearing resistance
ϕ_1	=	angle ϕ at pressure of 1.0 kg/cm^2
ϕ_{cv}	=	angle ϕ at constant volume
ϕ_t	=	angle ϕ prior to failure of soil beneath pile tip at high stress levels

LIST OF FIGURES




Average unit shaft friction f_o (KN/m²)
Normally consolidated deposits
Average mean effective normal stress $\bar{\sigma}_o$ (KN/m²)
0.0
10.0
20.0
30.0
40.0
50.0
0.0
20.0
40.0
60.0
80.0
100.0

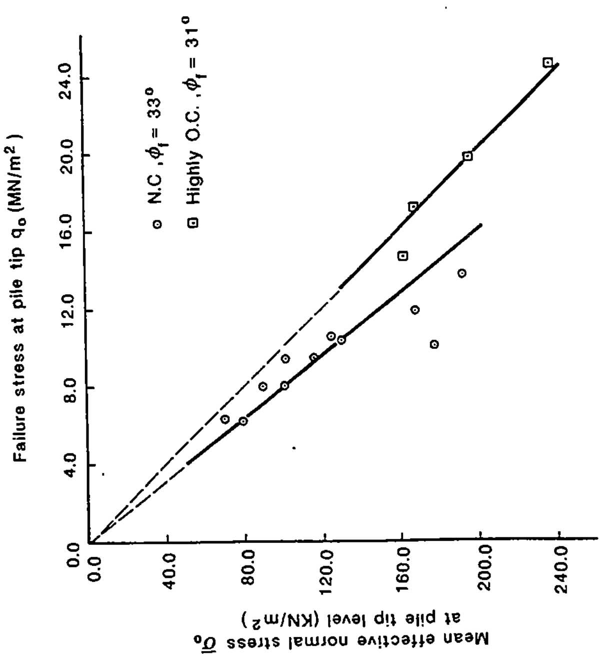

Failure stress at pile tip q_o (MN/m²)
N.C , ϕ_1 = 33°
Highly O.C. , ϕ_1 = 31°
Mean effective normal stress $\bar{\sigma}_o$ at pile tip level (KN/m²)
0.0
4.0
8.0
12.0
16.0
20.0
24.0
0.0
40.0
80.0
120.0
160.0
200.0
240.0

Fig 4.

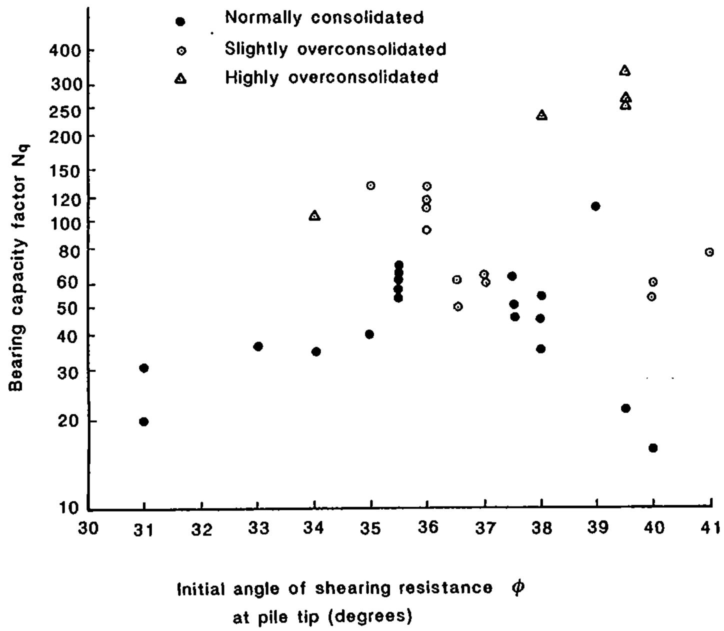
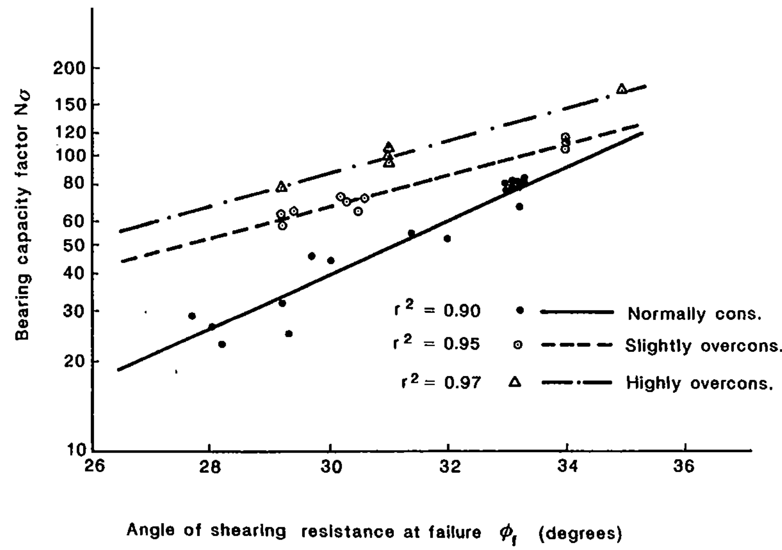
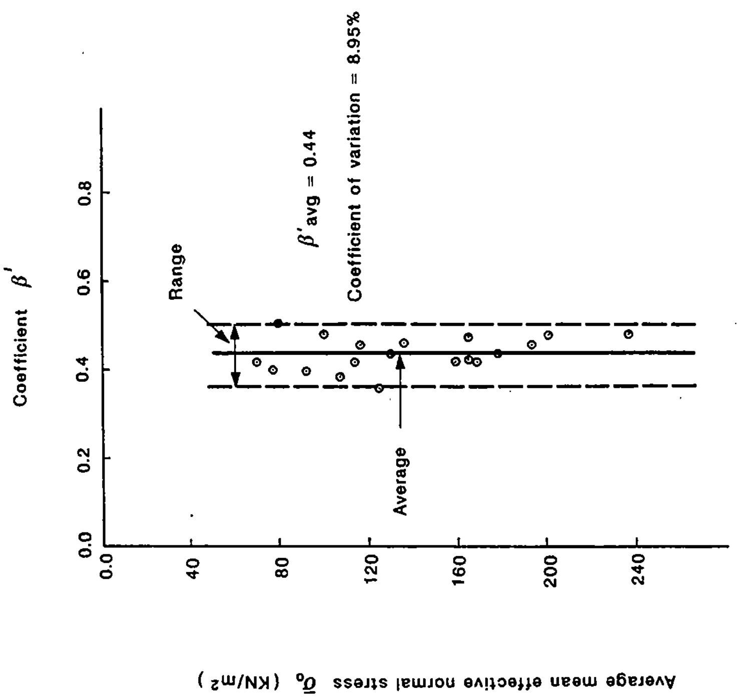
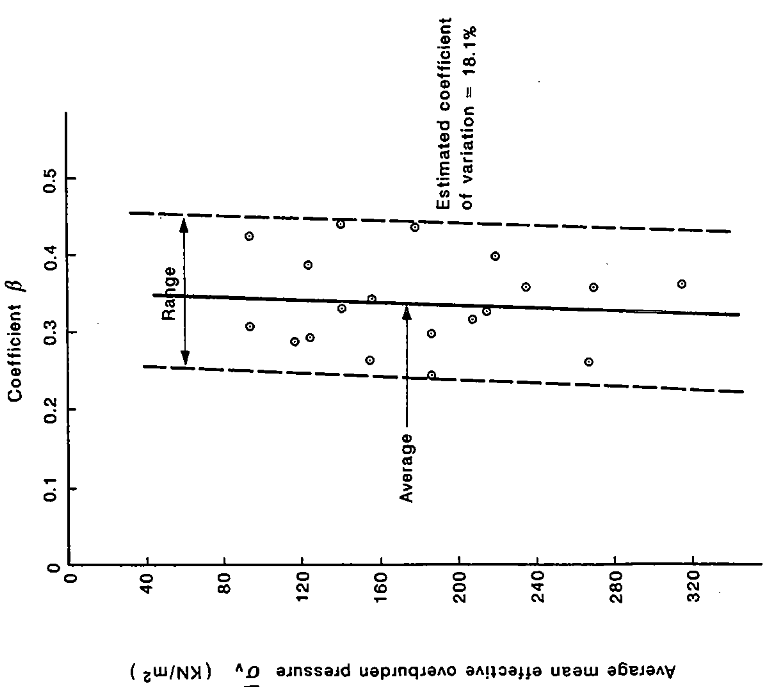
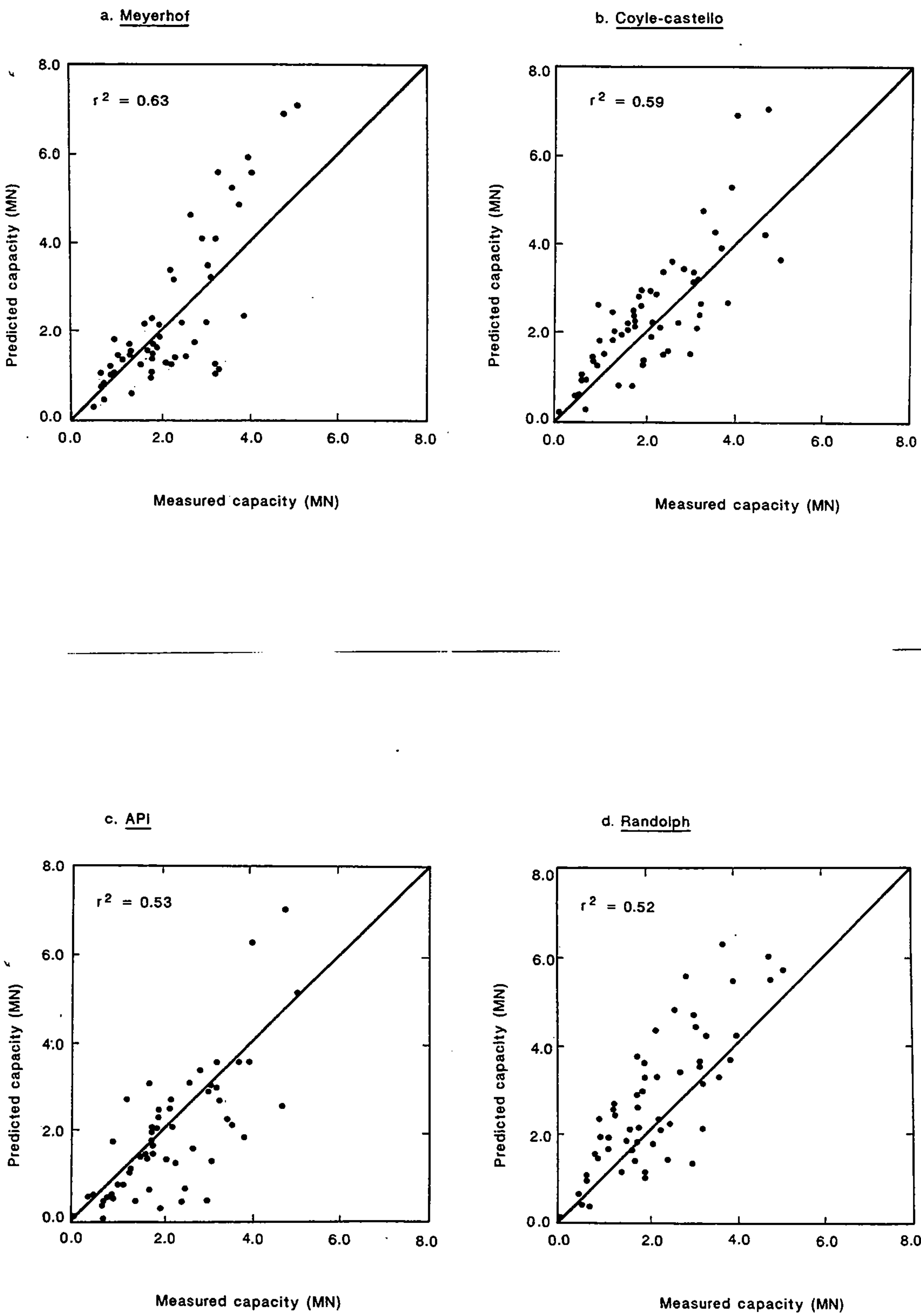

Normally consolidated
Slightly overconsolidated
Highly overconsolidated
Bearing capacity factor N_q
400
300
250
200
150
120
100
80
60
50
40
30
20
10
30
31
32
33
34
35
36
37
38
39
40
41
Initial angle of shearing resistance ϕ
at pile tip (degrees)

Fig 5

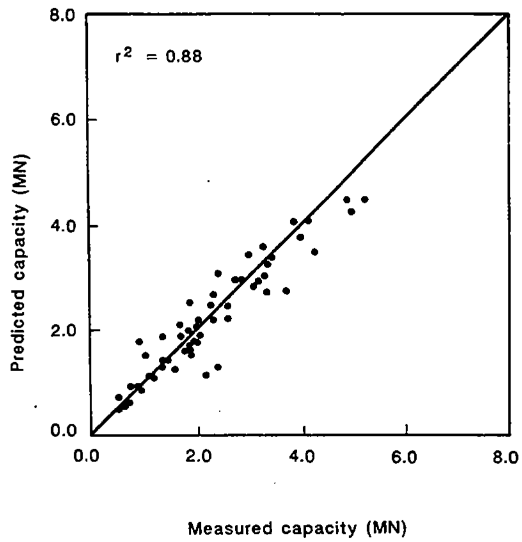

Bearing capacity factor N_σ
200
150
120
100
80
60
50
40
30
20
10
26
28
30
32
34
36
$r^2 = 0.90$ Normally cons.
$r^2 = 0.95$ Slightly overcons.
$r^2 = 0.97$ Highly overcons.
Angle of shearing resistance at failure ϕ_f (degrees)


Coefficient β'
0.0
0.2
0.4
0.6
0.8
Range
β' avg = 0.44
Coefficient of variation = 8.95%
Average
0.0
40
80
120
160
200
240
Average mean effective normal stress σ̄o (KN/m²)


Coefficient β
0
0.1
0.2
0.3
0.4
0.5
Range
Estimated coefficient
of variation = 18.1%
Average
0
40
80
120
160
200
240
280
320
Average mean effective overburden pressure σ̄v (KN/m²)


a. Meyerhof
r2 = 0.63
b. Coyle-castello
r2 = 0.59
c. API
r2 = 0.53
d. Randolph
r2 = 0.52
Predicted capacity (MN)
Measured capacity (MN)
0.0
2.0
4.0
6.0
8.0

e. Suggested Method



NONLINEAR FINITE ELEMENT ANALYSIS OF PAVEMENTS USING MICROCOMPUTER

R. Panneer Selvam[1]
Robert P. Elliott[2]

INTRODUCTION

The computer program ILLI-PAVE (1982) is perhaps the most powerful and realistic tool presently available for the structural analysis of flexible pavements. This program, developed at the University of Illinois, models the nonlinear, stress dependent behavior of unbound granular base and subgrade soils. It also includes a stress adjustment feature based on the Mohr-Columb failure theory to compensate for stress states that exceed the strength of the materials.

ILLI-PAVE is the culmination of the efforts of several researchers over many years. As a result it is long and quite complex. It is based on older FORTRAN compilers and uses data input routines that are quite cumbersome by todays standard. Its output is voluminous. This combination makes the program difficult to modify for further research work and the voluminous output hinders ease of use for more routine purposes.

The objective of this study was to formulate a program comparable to ILLI-PAVE using the same material models that: 1) would be more user friendly, 2) would have a simpler program structure for easier future modifications, 3) would use the more efficient isoparametric finite element, and 4) would be usable in a microcomputer. The program developed is called ARKPAV.

MATERIAL BEHAVIOR MODELING

A typical flexible pavement consists of three or more layers as depicted in Fig. 1. With the brief loading times associated with moving vehicle loads, the asphalt concrete surface layer behaves in a manner that is essentially elastic. This material can be reasonably modeled as linear elastic. However, the other two layers behave in a non-linear, stress dependent fashion that complicates the mathematical modeling.

[1] Associate Professor of Civil Engineering, BELL 4190, University of Arkansas, Fayetteville, AR 72701, USA.
[2] Professor and Head of Civil Engineering, BELL 4190, University of Arkansas, Fayetteville, AR 72701, USA.

The granular base material behaves in "stress stiffening" manner and the cohesive subgrade soil behaves in a "stress softening" manner. For granular soils

$$M_R = k(\theta)^n \quad (1)$$

in which M_R is the resilient modulus; $\theta = \sigma_1+\sigma_2+\sigma_3$; $\sigma_1, \sigma_2, \sigma_3$ = principal stresses; and k,n= material constants. For subgrade soils

$$M_R = K_2+K_3[K_1-(\sigma_1-\sigma_3)]; \text{ when } K_1 > \sigma_1-\sigma_3 \quad (2a)$$

$$M_R = K_2+K_4[(\sigma_1-\sigma_3)-K_1]; \text{ when } K_1 < \sigma_1-\sigma_3 \quad (2b)$$

in which K_1,K_2,K_3,and K_4 = material constants. Models for these behavior battens were incorporated into ILLI-PAVE by Duncan et al. (1968).

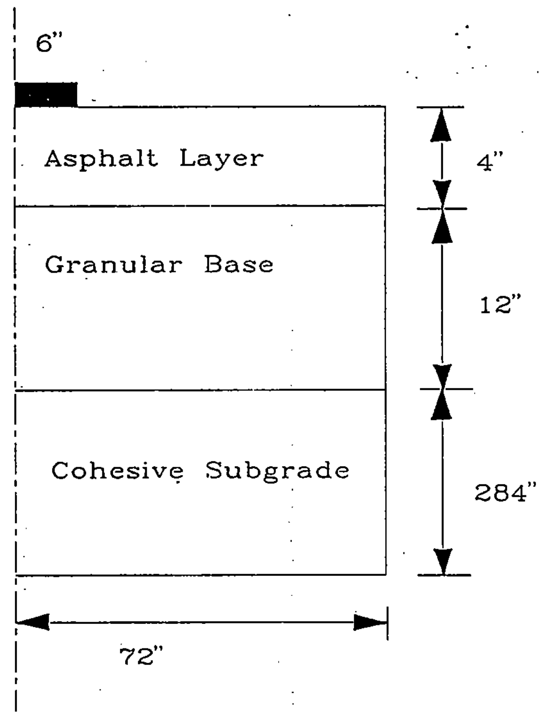


Fig. 1. Flexible Pavement Considered for Analysis.

For many pavement systems, these models alone were not sufficiently complete since the calculated stress states often exceeded the strength of the materials. This was particularly true for the unbound, granular base and subbase material as well as the upper portions of the subgrade. Raad and Figueroa (1980) improved the ILLI-PAVE model by incorporating a stress adjustment feature based on Mohr-Coulomb failure theory.

Another advantage of ARKPAV is that the programming is simpler and more straight forward. ILLI-PAVE was developed over many years with many researchers contributing to todays program. As a result, the programming is complex and not fully documented. Analysis of the ILLI-PAVE code for de-bugging when installing on a different machine or for the incorporation of additional modifications is quite time consuming. The simpler structure of ARKPAV will make the incorporation of any future modification much easier. With all of the modifications, the required storage for the uncompiled FORTRAN code (source code) for ARKPAV is about 25 k compared to 360 k for ILLI-PAVE.

COMPUTER MODELING

The flexible pavement having the nonlinear behavior mentioned in the previous section is analyzed for stresses and strains using finite element method. ILLI-PAVE models the three-dimensional pavement as an axisymmetric solid of revolution. The pavement region is divided into quadrilateral elements formed out of four triangular elements. The triangular element used in ILLI-PAVE was incorporated by Duncan et. al. in 1968. Currently isoparametric quadrilateral elements which can predict displacements and stresses with much better accuracy than triangular elements are available (Bathe 1982) in finite element literature.

ARKPAV was developed incorporating the nonlinear material models used in ILLI-PAVE but using the more accurate isoparametric quadrilateral element. ARKPAV is written in FORTRAN77 which is compatible with most computers. The input preparation is free format and very simple to prepare.

ILLI-PAVE prints the stresses and strains of each element and the displacements of each node for each iteration. The output is rather voluminous. In many cases, the pavement analyst is only concerned with the radial strain at the bottom of the asphalt concrete layer and the vertical strain at the top of subgrade. ARKPAV includes an option to print only this information.With these modifications ARKPAV size is about 25k compared to 360k for ILLI-PAVE.

SOLUTION PROCEDURE

In the ILLI-PAVE program the resulting symmetric simultaneous equations from finite element method are stored in a banded matrix form considering only the upper triangular part. These are solved by a direct solution procedure similar to Cholesky decomposition. Instead of solving the banded matrix in core, the matrix is divided into blocks and only two blocks are considered at one time in core. The rest of the blocks are stored in out of core devices. By this process large set of equations can be solved with the available storage. The minimum size of the block should be at least as large as the semiband width.

In a microcomputer, the time to transfer from disk to core takes a relatively long time. To eliminate the need for this transfer, a more efficient storage scheme is used in ARKPAV. This scheme stores only the non-zero elements of the lower triangular matrix in an array illustrated as A in Fig.2 (Axelsson and Barker, 1984; and Johnson, 1987). As illustrated, the values are stored in order by row. The column locations of the non-zero values are stored in another array labelled LC. A third array, LR, is used to identify the row locations. In array LR, the ith value identifies the beginning point in A of the ith row of the matrix. For example in Fig. 2., the third value in LR is 4 which indicates that the third row of the matrix starts from the fourth element of A. This storage scheme is independent of the bandwidth which greatly reduces storage needs. For pavement analysis using quadrilateral element maximum nine storage spaces are needed for each equation. For example, the storage needed for a 16x17 mesh in ARKPAV is about 20k of memory; for the same mesh using the banded matrix method used in ILLI-PAVE 157k of memory is required.

ARKPAV further reduces storage requirements by solving the equations using an iterative solution procedure. The direct procedure used by ILLI-PAVE requires double precision for acceptable accuracy. With the iterative procedure, single precision provide acceptable accuracy.

$$A = \begin{bmatrix} a11 & & & & \\ a12 & a22 & & \text{sym} & \\ 0 & a32 & a33 & & \\ a41 & 0 & a43 & a44 & \\ a51 & 0 & a53 & 0 & a55 \end{bmatrix}$$

	1	2	3	4	5	6	7	8	9	10	11	
A	[a11	a12	a22	a32	a33	a41	a43	a44	a51	a53	a55]	
LC	[1	1	2	2	3	1	3	4	1	3	5]	
LR	[1	2		4		6			9			12]

Fig. 2 Illustration of the Compact Storage Scheme

The equations are solved by preconditioned conjugate gradient iterative procedure. There are different preconditioning procedures are available in the literature (Axelsson and Barker, 1984; Mitchell and Griffiths, 1980; and Oden and Carey, 1982). In this work Jacobi conjugate gradient(JCG) is used; which is simple to implement without extra storage and is computationally efficient. The following is the

JCG algorithm for simultaneous equations of the type Ax=b (where A is a symmetric matrix and x and b are unknown and known vectors):

x_0 = starting guess at solution
$r_1 = b\text{-}Ax_0$
$p_0 = 0$ and $r_0\, M^{-1}\, r_0 = 10^{20}$
For k=0,1,2,..do until required convergence
$\beta \ = r^t_{k+1}\, M^{-1}\, r_{k+1} / (r^t_k\ M^{-1}\ r_k)$
$p_{k+1} = M^{-1}\, r_{k+1} + \beta p_k$
$\alpha \ = r^t_{k+1}\, M^{-1}\, r_{k+1} / (p^t_{k+1}\ A\ p_{k+1})$
$x_{k+1} = x_k + \alpha p_{k+1}$
$r_{k+2} = r_{k+1} - \alpha A p_{k+1}$

In this r_k,x_k,p_k,b are vectors and α and β are scalars. Here M is a diagonal matrix whose diagonals are diagonal of A. The above iteration is performed until the required convergence is reached. This step of solving Ax=b until required convergence is called outer iteration. This outer iteration is done until the updated resilient modulus of the materials do not have much difference from the previous outer iteration. When performing the nonlinear outer iteration, the solution of Ax=b for the first iteration takes little bit more iteration because of assumed initial x is for linear solution. The second and subsequent outer iterations take less number of iteration in JCG because x is very close to the actual value. Usually four outer iteration are enough for convergence. In a 486/33mhz machine ARKPAV takes about 7 minutes to solve a sample problem of 16x17 mesh.

RESULTS

A nonlinear finite element analyses was performed using a mesh size of 16x17 for a typical pavement shown in Fig. 1 to compare different design parameters from ILLI-PAVE to ARKPAV. The failure modulus values for asphalt, base and subgrade are taken as 500000, 4000 and 3000 psi respectively. The layer thickness are 4, 12, and 284 inches for asphalt, base and subgrade layers respectively. The earth pressure at rest are 0.67, 0.6, and 0.82; the Poisson's ratios are 0.4, 0.38 and 0.45; and the densities are 145, 135 and 120 pcf respectively for asphalt, base and subgrade. The material coefficient k and the exponent n for the granular material are 5000 psi and 0.5 respectively. The maximum allowable stress ratio and the minimum horizontal compressive stress (psi) that the granular material is permitted to reach before tensile failure are 4.8 and 0.01 respectively. For the cohesive subgrade the deviator stress(K_1 in Eq.2) and the corresponding modulus value at the turning point of the deviator stress resilient modulus relation(K_2 in Eq. 2) are 6 and 6000 psi respectively. The slopes of the left and right portions of the deviator resilient modulus relation are 1000 and -200 respectively. The lower and upper limiting values for deviator stress in the deviator stress resilient modulus relation K_3 and K_4 in Eq. 2 are 2 and 21 psi respectively. The shear strength beyond which failure occurs is considered as 15 psi. The tire pressure is assumed to be 80 psi and applied on a radius of 6 inches. This is the data set used in ILLI-PAVE (1982) users manual. The nonlinear relationships are well explained in Raj (1991), ILLI-PAVE(1982).

Using the above input parameters the deflection at the top of asphalt, radial strain at the bottom of asphalt and vertical strain at the top of subgrade are considered for comparison. These parameters are reported in Table 1. The displacement and strains are almost the same value from ARKPAV and ILLI-PAVE. The displacement and radial strain from ARKPAV are slightly higher than ILLI-PAVE. Further details on mesh refinements and its effects on the improvement on the displacement and strains are reported in Raj(1991).

Program	ARKPAV	ILLI-PAVE
Vertical Deflection	0.02380 in.	0.02303 in.
Radial Strain on Base	0.3040×10^{-3}	0.2804×10^{-3}
Vertical Strain on Subgrade	0.5781×10^{-3}	0.5813×10^{-3}

Table 1. Comparison of Results from ARKPAV and ILLI-PAVE.

CONCLUSIONS

A nonlinear pavement analysis program ARKPAV which is similar to ILLI-PAVE material model is developed. ARKPAV uses efficient isoparametric quadrilateral element. The program is made usable in microcomputer by storing them in a compact scheme. For a 16x17 mesh about 8 times storage saving is achieved using ARKPAV than ILLI-PAVE. The saving in storage increases with increase in the mesh size. The equations are solved using Jacobi conjugate iterative procedure which is comparable in speed with direct solution procedures. In IBM486 equivalent machines a 16x 17 mesh takes about 7 minutes. The results from ARKPAV and ILLI-PAVE for a sample problem is in good agreement. This program is easy to modify for future use. It is available for anyone to uses it for a reasonable cost.

REFERENCES

Axelsson, O., and Barker, V.A.,(1984),**Finite Element Solution of Boundary Value Problems**: Theory and Computation, Academic Press, Inc. New York.

Bathe, K.J.,(1982),**Finite Element Procedures in Engineering Analysis**, Prentice Hall.

Duncan,J.M, Monismith, C.L.,and Wilson, E.L.,(1968), "Finite Element Analysis of Pavements," **Highway Research Record** No. 228, Highway Research Board, Washington, D.C., pp.18-33.

ILLI-PAVE Users Manual (1982), Transportation Facilities Group, Department of Civil Engineering, University of Illinois at Urbana- Champaign, May

Johnson, C.,(1987),**Numerical Solution of Partial Differential Equations by the Finite Element Method**, Cambridge University Press, New York.

Mitchell, A.R., and Griffiths, D.F., (1980),**The Finite Difference Method in Partial Differential Equations**, John Wiley & sons, New York.

Carey, G.F.,and Oden, J.T., (1984), **Finite Elements: Computational Aspects**, Vol. 3, Prentice-Hall, New Jersey.

Radd, L., and Figueroa, J.L., (1980),"Load Response of Transportation Support Systems," **Transportation Engineering Journal**, ASCE, Vol. 106, No. TE1, pp. 111-128.

Raj, K., (1991),"Nonlinear Finite Element Analysis of Pavements," MSCE thesis, Department of Civil Engineering, University of Arkansas, Fayetteville, AR 72701, 77 pages, August.

Aggregate Suitability (Stripping) for Asphalt Pavements

Sam I. Thornton and Miller C. Ford, Jr.
University of Arkansas - Fayetteville

ABSTRACT

Five natural aggregates: limestone, sandstone, granite, chert gravel, and novaculite, were examined for use in asphalt pavements. In all, eighteen samples from the aggregates were tested. Of the five, only the limestone was hydrophobic (basic).

Based on "film stripping" and "immersion-compression" tests, the limestone aggregate performed best. In addition, limestone samples with insolubles like silica dioxide performed less well than samples without insolubles.

Skid resistance, however, was better on samples with insolubles. Hardness of the included insolubles was the reason for the better skid resistance.

INTRODUCTION

In a study (Ford,1978) of asphalt pavement stripping, eighteen samples of five natural aggregates and one synthetic aggregate were selected as representative of the aggregates used in Arkansas asphalt pavements. Locations of the sources are widely spread over Arkansas. Seven of the samples were limestone, five were sandstone, three were gravel, one was novaculite, one was granite (syenite) and one was synthetic (expanded clay).

BACKGROUND

Stripping in asphalt pavement is caused by water,and accelerated by traffic. Aggregates are removed from the mixture by raveling. In extreme cases, the entire pavement mass disintegrates. Stripping is thought to occur when there is a loss of adhesion between the aggregate and asphalt cement.

Asphalt mixtures may be divided into two types of asphalt-aggregate systems for stripping evaluation, seal coats and compacted asphalt mixtures. Different stripping tests are required for each system.

SEAL COATS

Seal coat construction has two stripping tests, called film stripping tests, devised to evaluate single size aggregate particles coated with asphalt cement. One of the tests is static and the other is dynamic.

The static immersion stripping (SIS) test is based on the work of Hubbard (1936): the test procedure used is similar to the ASTM D 1664 method. In the SIS tests no stripping of the aggregate was observed with a 77F water temperature (ASTM D1664). When the test temperature of the water was raised to 140F and after 18 hours of soaking, a considerable amount of stripping occurred. The amount of stripping was estimated visually using a comparison chart, then the amount of stripping was measured by the Surface Reaction Test of Ford et al (1974).

A Dynamic Immersion Stripping (DIS) test, based on the work of Nicholson (1932), was performed using a Soiltest apparatus. The asphalt coated aggregate was placed in a glass jar, covered with water and rotated in the Soiltest apparatus. This action caused the coated aggregate to fall from one end of the jar through the water to the other end during each revolution. A test temperature of 77F was maintained during the 4 hour tumbling period. The amount of stripping was evaluated visually and also by the Surface Reaction Test.

Film Stripping Tests Results

Film Stripping test results are given in Table I. More stripping usually occurred in the SIS at 140F, than in the DIS at 77F. This was confirmed by Visual evaluation and the Surface Reaction Test. The average SIS retained coating was 56 percent, whereas the DIS tests had a 70 percent retained coating.

Test results reflect the different test methods used to induce the stripping effects of water. In the DIS test, the coated aggregate particles are abraded against one another and the test jar as they revolve back and forth in the water. This action simulates the effects of traffic on the wet pavement surface. The SIS (140F) test simulates the effects of water being heated on or in the pavement during the summer months. There is no agreement among researchers about which method is more indicative of stripping.

An average of the results of the two stripping tests would rank the aggregates, with the average percentage retained coating, as: limestone (85), novaculite (66), synthetic (65), syenite (64), gravel (64) and sandstone (34). These results are in general agreement with the information obtained from the literature.

One of the more significant correlations was established between film stripping and insoluble residue of the aggregate. The curve relating insoluble residue and average retained coating is shown in Figure 1. The best fitted equation shown had a coefficient of correlation of 0.72 and indicates a decrease in stripping resistance with increasing amounts of insoluble residue.

COMPACTED ASPHALT MIXTURES

The compacted asphalt mixtures such as asphalt concrete are evaluated for their stripping tendency by strength tests, swell tests, abrasion tests, or the non-destructive resilient modulus test.

TABLE I

Laboratory Test Results - Aggregate Shape and Film Stripping

No.	Aggregate	Static Immersion Visual Eval. %		Dynamic Immersion Visual Eval. %	Surface Reaction Ret. Coating - %		Particle Shape	
		77F	140F	77F	SIS 140F	DIS 77F	Flakiness Index	Elongation Index
1	Johnson ls	100	75	80	90	98	25	14
2	Valley Springs ls	100	90	85	96	99	25	13
3	Cabot ss	100	40	80	44	49	42	30
4	Van Buren ss	100	50	75	26	37	33	18
5	Jenny Lind ss	100	60	60	46	21	37	18
6	Texarkana gvl	100	70	60	47	-	28	24
7	Murfreesboro gvl	100	30	90	57	81	25	25
8	Big Rock ns	100	60	80	49	79	38	18
9	Malvern nov	100	70	75	48	84	31	24
10	England syn	100	40	85	38	92	20	6
11	Twin Lakes ls	100	70	80	62	72	24	12
12	Kentucky ls	100	80	85	95	99	24	23
13	Rocky Point ls	100	90	90	93	98	31	24
14	Black Rock ls	100	30	85	54	46	39	29
15	West Fork ls	100	70	90	94	98	21	10
16	Russellville ss	100	20	85	52	32	36	9
17	Bald Knob ss	100	20	90	18	14	33	33
18	Hampton gvl	100	70	90	80	72	42	12

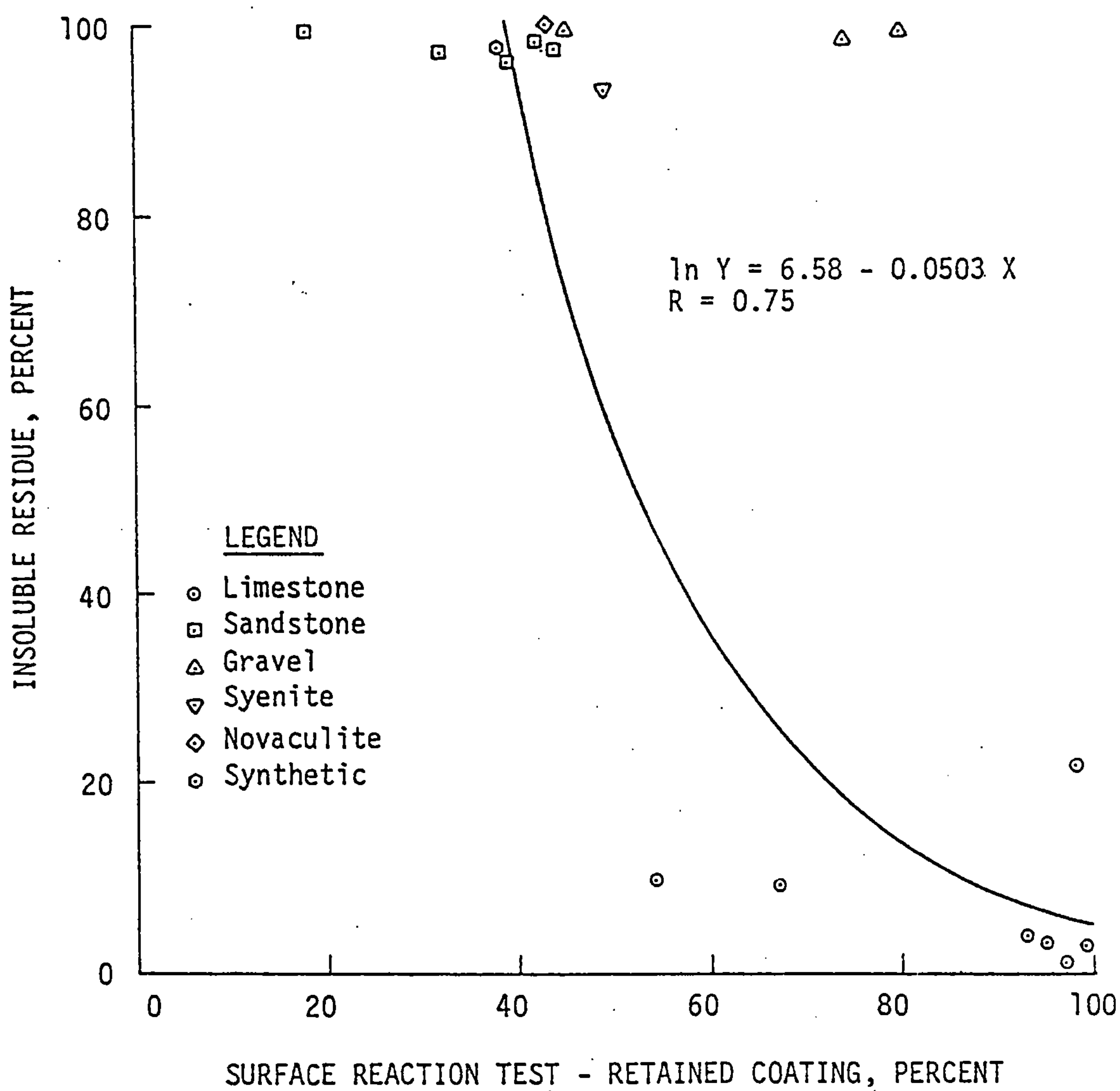


Figure 1 Relationship Between Insoluble Residue and SRT Retained Coating

Marshall specimens were used to determine the polishing characteristics of the coarse aggregates under investigation. The fine aggregate consisted of about 10 percent river sand and 33 percent screened limestone from the West Fork quarry. Thus, the 57 percent coarse aggregate of the mix was the variable to be evaluated for its stripping tendency.

Each aggregate combination was molded using a 50 blow Marshall procedure at optimum asphalt content. A total of 12 specimens were made to represent each aggregate source. Six specimens were used in the polishing tests and six specimens were tested for their stripping resistance.

The test method used for the Immersion Compression (I-C) test was like the "Effect of Water on Resistance to Plastic Flow of Bituminous Mixtures using the Marshall Apparatus" reported in the 1973 Annual Book of ASTM Standards, Part II. To ensure that water had an opportunity to cause stripping, the "wet" specimens were vacuum saturated at 77F prior to immersion in the 140F water bath for 24 hours. The vacuum saturation procedure was similar to The Asphalt Institute Method.

The broken six specimens from the 50 blow Marshall stability I-C test were then reheated to 255F and loosened by hand using a trowel, and remolded at 35 blows per side for a second immersion-compression test cycle. This lower compactive effort was used to obtain greater air voids.

Compacted Mixture Stripping Test Results

The asphalt content, bulk specific gravity, air voids and voids in the mineral aggregate for each test aggregate are shown in Table II. The voids analysis was based on the measured maximum specific gravity of the mix (Gr) per ASTM D 2041. Since the water absorbed by each "wet" specimen was measured the volume of air voids based on water absorption is also shown in Table II. The average air voids based on ASTM D 2041 was 2.7 percent, while the average air voids based on water absorption was 2.5 percent.

The stiffness of each Marshall specimen was determined using the Marshall stability and flow data. The Marshall modulus was the slope of the stress-strain curve from zero to the point of maximum load. The percentage retained modulus was calculated by dividing the "wet" modulus by the "dry" modulus.

Likewise the Immersion-Compression retained percentage was determined by dividing the "wet" stability by the "dry" stability. Several of the wet stabilities were greater than the dry stabilities giving I-C's greater than 100 percent. However, the wet Marshall modulus values were less than the dry ones which gave values of retained modulus's less than 100 percent.

The average Marshall modulus retained for all of the mixtures by type of aggregate was: limestone, 89; sandstone, 78: gravel, 75; syenite, 64; and novaculite, 82. The immersion-compression

TABLE II

Laboratory Test Results - Marshall Stability, Retained Modulus and Immersion-Comp.

No.	Aggregate Name	Blows per Side	Bulk Sp. Gr.	Air Voids %		VMA	Mix	Marshall Test (dry); Wet/Dry				AC	Film
				Gr Basis	Water Abs.	%	Insol Res.	Stab lb.	Flow .01"	Modulus % Ret.	I-C % Ret.	%	Strip. % Ret.
1	Valley Spgs. ls	50	2.443	0.8	0.7	13.5	7.6	2185	17	93	99	5.3	98
1R	"	35	2.411	2.1	1.4	14.6	"	2045	18	86	93	"	"
2	Twin Lakes ls	50	2.430	1.9	2.7	14.8	11.3	2565	17	96	99	5.4	67
2R	"	35	2.407	2.9	3.2	15.8	"	2330	15	77	90	"	"
3	Kentucky ls	50	2.448	1.7	0.9	13.9	7.8	2000	16	90	104	5.1	97
3R	"	35	2.435	2.1	1.2	14.3	"	2075	19	100	100	"	"
4	Rocky Point ls	50	2.425	1.0	1.1	13.4	8.4	2240	16	91	92	5.2	96
4R	"	35	2.410	1.6	1.3	13.9	"	2010	18	95	100	"	"
5	Black Rock ls	50	2.471	2.4	1.3	15.3	11.6	2065	18	86	89	5.3	50
5R	"	35	2.475	2.3	1.2	15.1	"	1920	21	85	112	"	"
6	West Fork ls	50	2.406	2.2	1.4	15.2	6.6	2170	10	82	110	5.5	96
6R	"	35	2.393	2.8	1.8	15.6	"	2325	16	91	94	"	"
7	Cabot ss	50	2.388	1.3	2.1	14.0	61.3	2405	17	92	100	5.4	46
7R	"	35	2.371	2.0	2.2	14.6	"	2150	16	92	104	"	"
8	Van Buren ss	50	2.369	2.5	3.1	14.8	60.6	2485	14	68	90	5.3	32
8R	"	35	2.361	2.9	2.6	15.2	"	2435	14	78	102	"	"

TABLE II (continued)

No.	Aggregate Name	Blows per Side	Bulk Sp. Gr.	Air Voids % Gr Basis	Air Voids % Water Abs.	VMA %	Mix Insol Res.	Marshall Test (dry); Stab lb.	Marshall Test (dry); Flow .01"	Wet/Dry Modulus % Ret.	Wet/Dry I-C % Ret.	AC %	Film Strip. % Ret.
9	Jenny Lind ss	50	2.325	4.2	4.8	16.0	62.2	2640	11	61	84	5.2	34
9R	"	35	2.309	4.8	5.5	16.6	"	2520	13	61	82	"	"
10	Russellville ss	50	2.396	2.5	2.1	14.7	61.6	2415	15	86	95	5.2	42
10R	"	35	2.382	2.8	2.0	15.0	"	2370	15	90	90	"	"
11	Bald Knob ss	50	2.376	2.5	1.5	14.8	62.8	2775	13	83	91	5.3	16
11R	"	35	2.345	3.8	2.8	15.8	"	2430	15	68	77	"	"
12	Texarkana gvl	50	2.357	2.8	2.9	15.9	63.0	1775	15	74	88	5.7	47
12R	"	35	2.361	2.7	2.5	15.8	"	1750	14	77	92	"	"
13	Murfreesboro gvl	50	2.326	2.7	4.3	15.4	63.0	2105	14	68	85	5.6	69
13R	"	35	2.313	3.1	4.2	15.8	"	1705	14	72	90	"	"
14	Hampton gvl	50	2.380	2.4	1.7	14.3	62.5	1855	15	87	96	5.1	76
14R	"	35	2.364	3.0	2.3	14.8	"	1985	13	72	89	"	"
15	Big Rock syenite	50	2.348	4.0	4.0	16.0	59.6	1975	11	56	85	5.2	65
15R	"	35	2.334	4.4	3.9	16.4	"	2065	15	72	83	"	"
16	Malvern nov	50	2.327	3.7	3.3	16.2	63.2	2005	15	87	92	5.5	66
16R	"	35	2.307	4.2	3.4	16.7	"	1985	15	77	92	"	"

retained strength averages were: limestone, 98; sandstone, 92; gravel, 90; syenite, 84; and novaculite, 92.

One of the most significant correlations of I-C retained strength was with air voids in the mixture (Figure 2). The best fitted curve for this data was:

$$\log \text{air voids} = 5.7 - 2.65 \log \text{I-C}$$

This curve indicates a retained strength of 70 percent at about 6 percent air voids.

For comparison, the relationship between I-C retained strength and Marshall modulus is shown in Figure 3. The best fitted curve for this data was:

$$\log \text{I-C} = 1.0 + 0.506 \log \text{Retained Marshall Modulus}$$

For a I-C retained strength of 70 percent the Marshall modulus retained was about 45 percent.

The relationship between air voids (Gr basis) and the Marshall modulus is shown in Figure 4. The data indicates a good correlation, with a decrease in Marshall modulus as air voids increase. The best fitted curve for this data was:

$$\text{Air Voids} = 8.07 - 0.0662 \text{ Retained Marshall Modulus}$$

The retained Marshall modulus is 45 percent at 5 percent air void. The loss in stiffness of the mixture may be a better indicator of stripping resistance than the I-C retained strength ratio.

REFERENCES

ASTM, 1973, "1973 Annual Book of ASTM Standards," Part II, American Society of Testing Materials

Ford, M.C., Manke, P.G., and O'Bannon, C.E. "Quantitative Evaluation of Stripping by the Surface Reaction Test," Transportation Research Board Record 515, 1974.

Ford, Miller C. Jr, 1978, "Asphalt Surface Durability and Skid Resistance," Final Report of HRC 38, AHTD, Box 2261, Little Rock, AR 72203

Hubbard, P., 1938, "Adhesion of Asphalt to Aggregates in the Presence of Water," Proceedings, Highway Research Board, Vol 18, Part 1

Nicholson, V., 1932, "Adhesion Tension in Asphalt Pavements, Its Significance and Methods Applicable in Its Determination," Proceedings, The Association of Asphalt Pavement Technologists, Vol 3

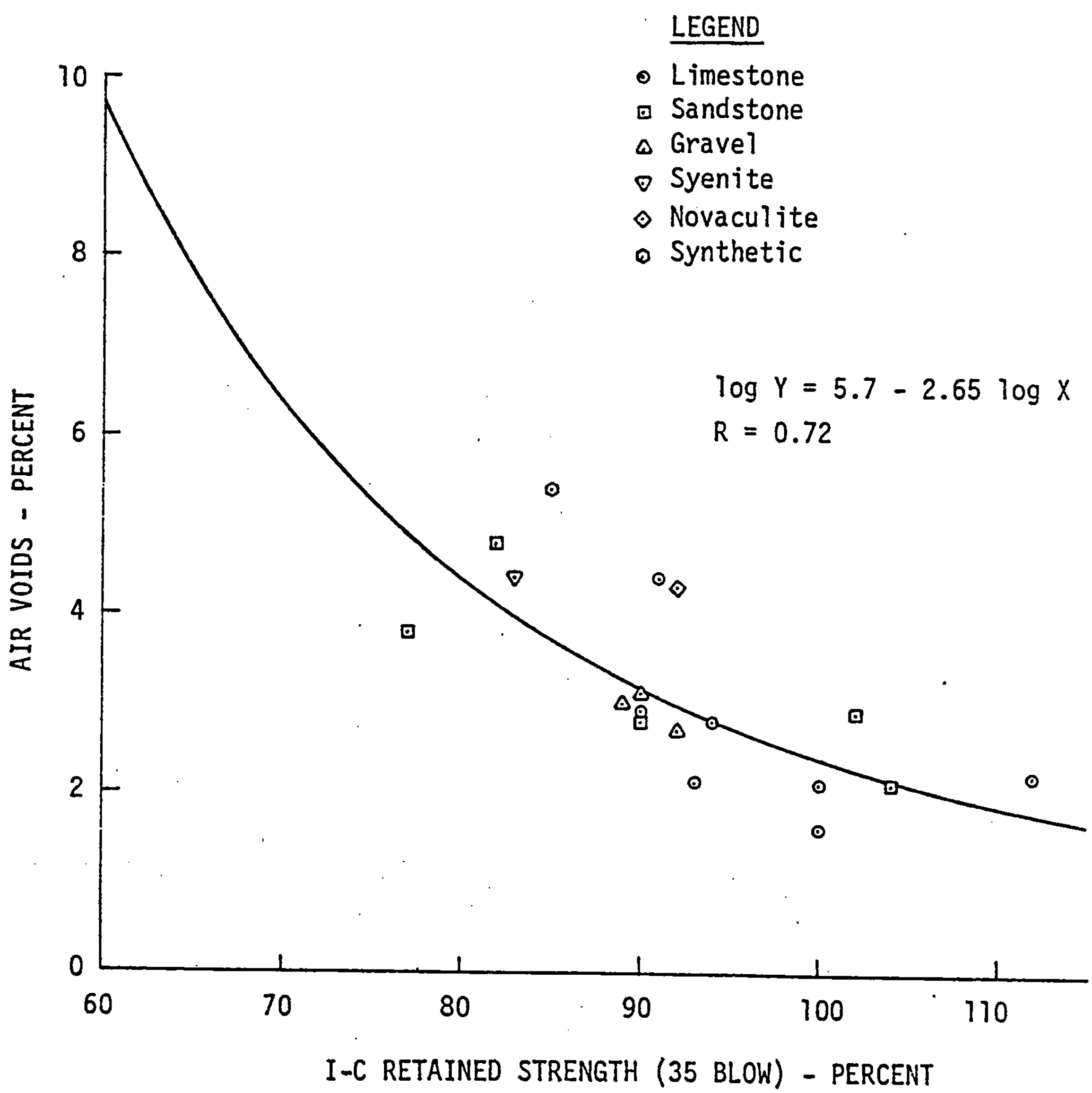


Figure 2 Relationship Between Air Voids and Immersion-Compression Retained Strength

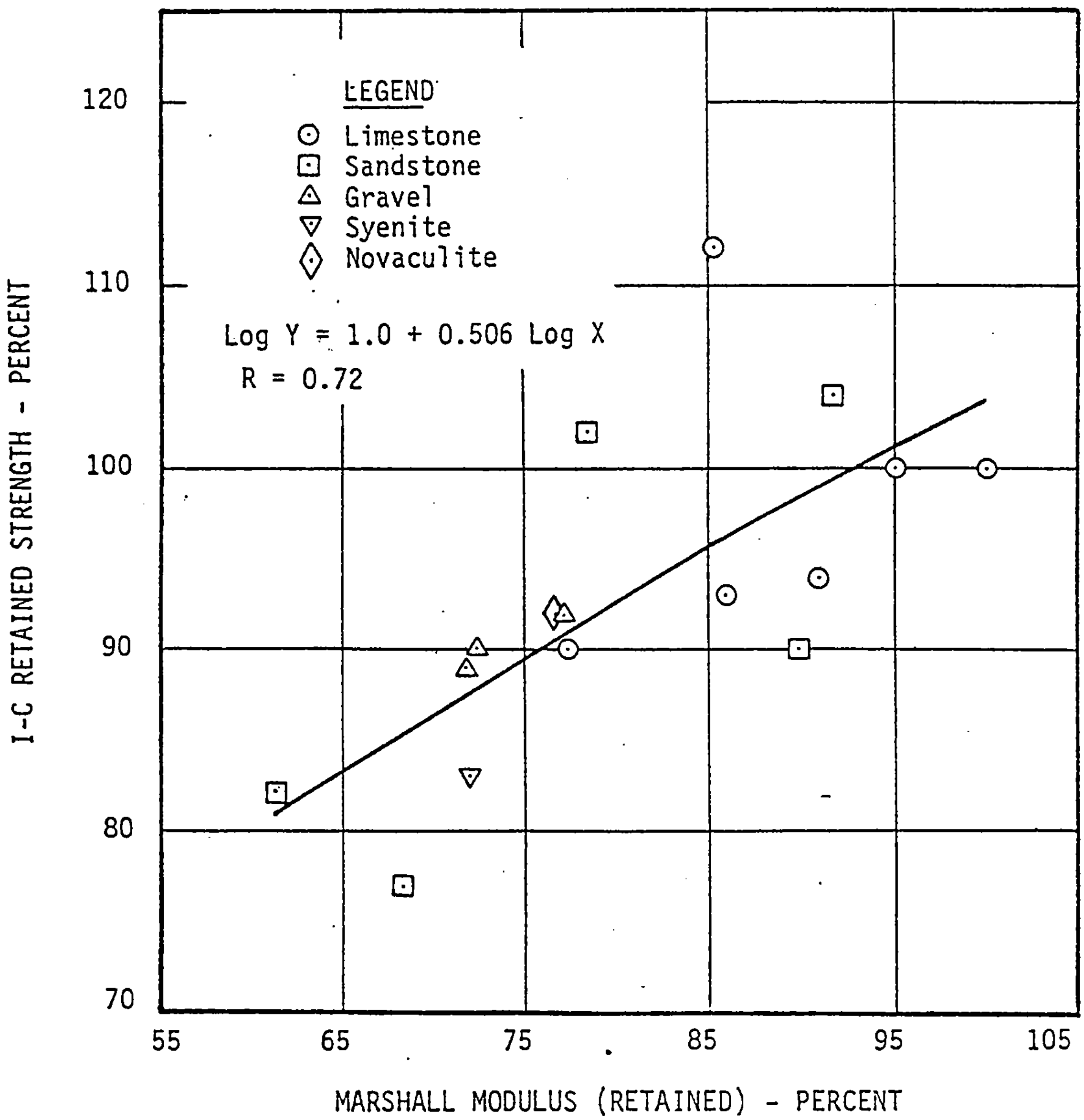


Figure 3 Relationship Between I-C Retained Strength and Marshall Modulus Retained Stiffness, 35 Blow Mix

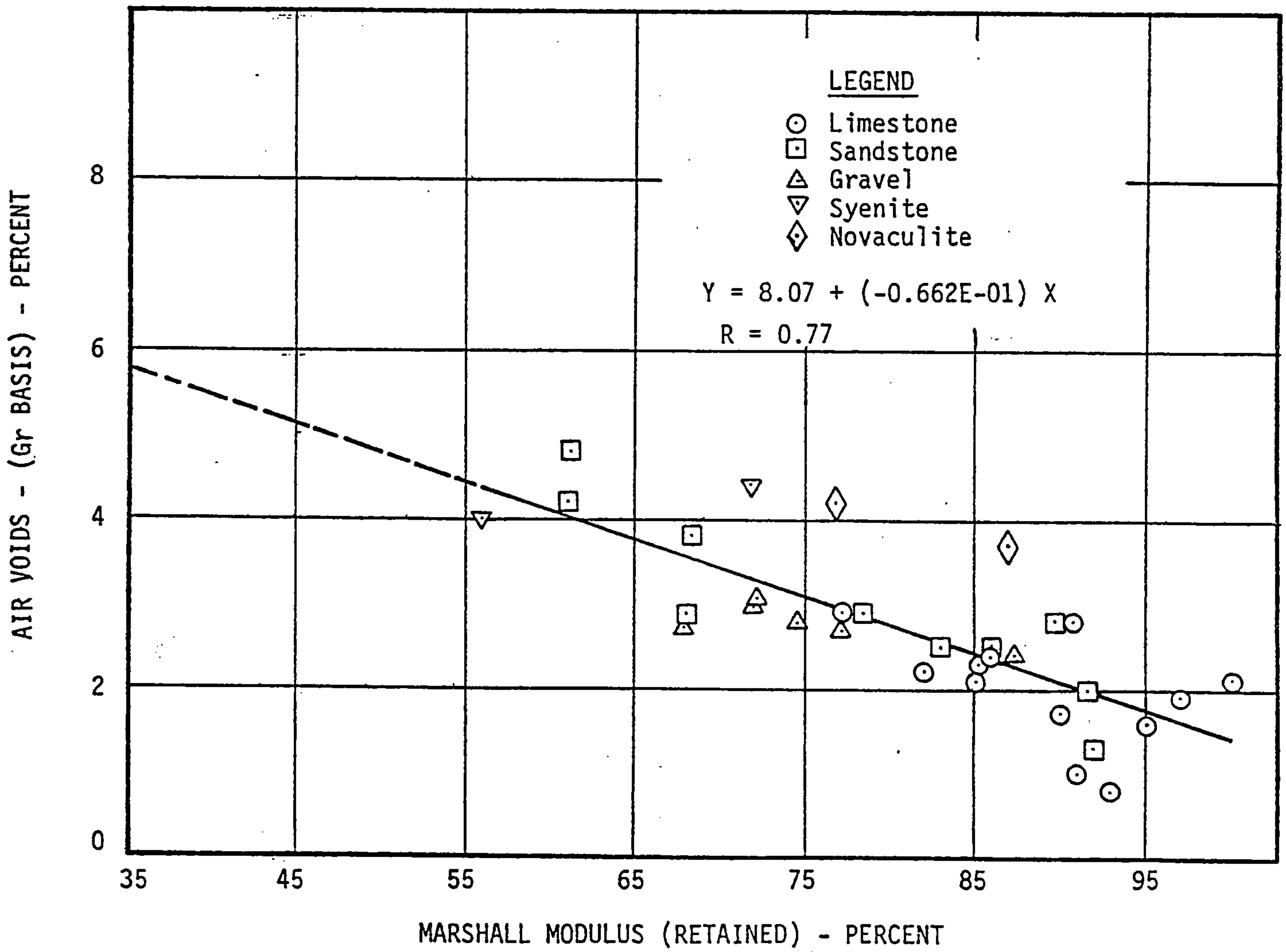


Figure 4 Relationship Between Air Voids (Gr Basis) and Marshall Modulus Retained Stiffness, 35 and 50 Blow Mixes

Christopher A. Ruppen, Geologist
Michael Baker Jr., Inc.
4301 Dutch Ridge Road
Beaver, PA 15009
(412) 495-7711

ABSTRACT

"WHEN DOES THE WORK END"

Although the actual procedures and formats vary from state to state, highway projects typically have established phases of the design process. The work starts with a general overview of a corridor or quadrant. Next this corridor is narrowed down to various alternatives and a rough alignment. The third phase of work is commonly a pre-final design phase which is followed by final design. The outcome of final design is a set of plans, specifications and estimates for construction.

Throughout the various phases of work and again depending upon the governing agency, there are various levels of involvement by geologists and geotechnical engineers in evaluating the alignment and supporting the design. Often, exhaustive efforts are made by geologists to investigate a particular alignment and support the design. These investigations commonly include a literature review, agency inquiries, field reconnaissance, test boring investigation and soil, rock and water laboratory analyses. These investigations are typically qualified in a report which would include recommendations, specifications and details which would be used for final design and construction.

Historically, the production of this final report is the culmination of the geologist's involvement with a project. The knowledge and intent gained from the geotechnical design investigations usually ends up being inferred or interpreted by the construction personnel who are neither geotechnical specialists nor familiar with the design effort.

A recent new highway construction project in Pennsylvania followed fast-track design phases. Geotechnical concerns investigated during the design include surface and deep mines, mine pools, landslide prone soils and soft embankment foundations. Extensive geotechnical investigations were completed with a major constituent being a boring program. From these investigations, substantial information was gathered relative to the project geology, stratigraphy, past mining and mine pool conditions. This information aided in the design of cut slopes and embankment and structure foundations.

This project differed from most in that when the project entered the construction phase, construction management was performed by the company which had participated in all phases of the design. Included on that construction management team was a geologist who was intimately familiar with the project geology and geotechnical design. As a result of this involvement by geotechnical personnel during construction, the value of a qualified field geologist being present during construction was demonstrated to apply geotechnical special provisions and construction details and solve day-to-day earthwork problems arising during the construction process. Similar involvement by geotechnical personnel should become a standard part of the construction process.

INTRODUCTION

Highways are almost always comprised of a combination of three types of construction: roadways are constructed through cut areas in hills or mountains, on engineered embankments crossing low areas, valleys and on the sides of slopes, or carried by bridges across water courses, sensitive environmental areas or other transportation systems. Whether excavating cuts, placing embankment or constructing bridge structures, all aspects of highway construction directly involve the use or handling of soil and bedrock.

Cuts are excavated through soil or bedrock and commonly through both. Embankments are constructed from soil and bedrock available from excavation on the site or from a borrow excavation. Bridge structures are constructed on both soil and bedrock. Therefore, site soil and bedrock, specifically the geology and understanding the site geology, are instrumental to construction of the roadway.

Highway geologists who are intimately familiar with a particular project's geology and geotechnical design typically end their role in the project at the culmination of the final design report. During the construction phase, this geologist or geotechnical engineer is not included in the construction decision making process to best utilize the available earth materials, apply geotechnical special provisions and details, and solve the day-to-day earthwork problems arising during construction.

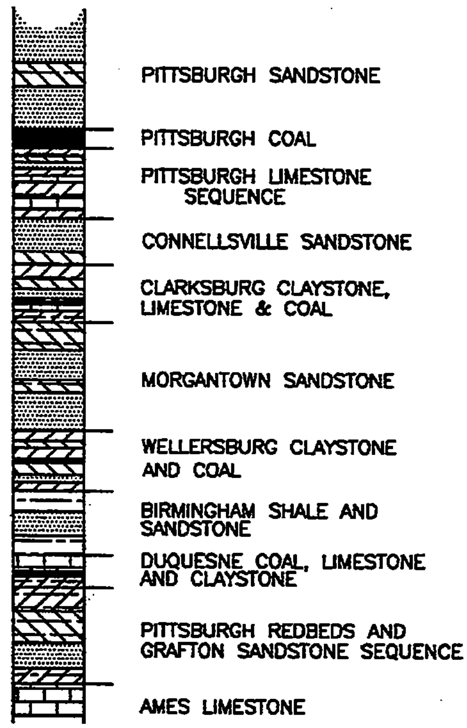
This discussion will use a recent highway project to illustrate the benefit of retaining the design geologist as part of the construction management team during the actual construction phase.

PROJECT DESCRIPTION

The project is a four-lane, limited access highway, totaling approximately twenty-one (21) miles of roadway. The project is located in Allegheny County near Pittsburgh, Pennsylvania and was designed and constructed under a fast-tracked process. Michael Baker Jr., Inc. of Beaver, Pennsylvania provided roadway, structure and geotechnical design services for preliminary and pre-final design phases of the project.

The project stratigraphically is positioned within the Pennsylvanian Age Pittsburgh and Casselman Formations of the Monongahela and Conemaugh Groups. These formations are sedimentary, relatively flat lying rocks comprised of alternating sequences of coal, limestone, claystone, shale, siltstone and sandstone. Economically important to the Pittsburgh area and adversely important to highway construction, the Pittsburgh coal is considered a significant unit within the sequence. Other limestone, sandstone and claystone units were also important in construction of the project.

LOCAL STRATIGRAPHY



Between 1987 and 1990, Baker undertook an extensive drilling program to evaluate the project geology and stratigraphy pertinent to the highway construction. Over 1000 borings were completed as part of these investigations which supplied invaluable information regarding site soil, bedrock, groundwater and mining conditions which was used for design and subsequent construction of the roadway. Specific information was gathered pertaining to:

- Embankment foundations (soft compressible soil)
- Site materials available for embankment construction
- Cut slope stability
- Mining conditions
- Mine pool levels
- Landslide-prone soil and bedrock

- Groundwater levels
- Structure foundations
- Borrow/waste areas

This project differed from most in that the company which had performed the design was also selected to provide the construction management. As the project entered the construction phase, the highway agency immediately identified the value of retaining the design geologists, whom were intimately familiar with the project geology and geotechnical design, as part of Baker's construction management team. Baker's Geotechnical Group supported the construction management team by placing a field geologist on the construction site and supporting this geologist with an office team of geologists and geotechnical engineers. This proved to be invaluable during the actual construction of the roadway.

DISCUSSION

The field geologist's role during construction generally fell into one of five general categories for discussion:

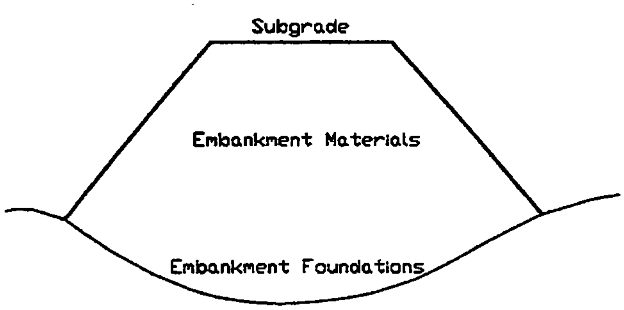
- Embankments
- Cuts
- Subgrade
- Structures
- Special Problems

Embankments

Embankments ranged in height to eighty feet. The embankment construction required the greatest involvement by the field geologist due to geotechnical special provisions and details which included strict requirements regarding the materials used to construct the embankments.

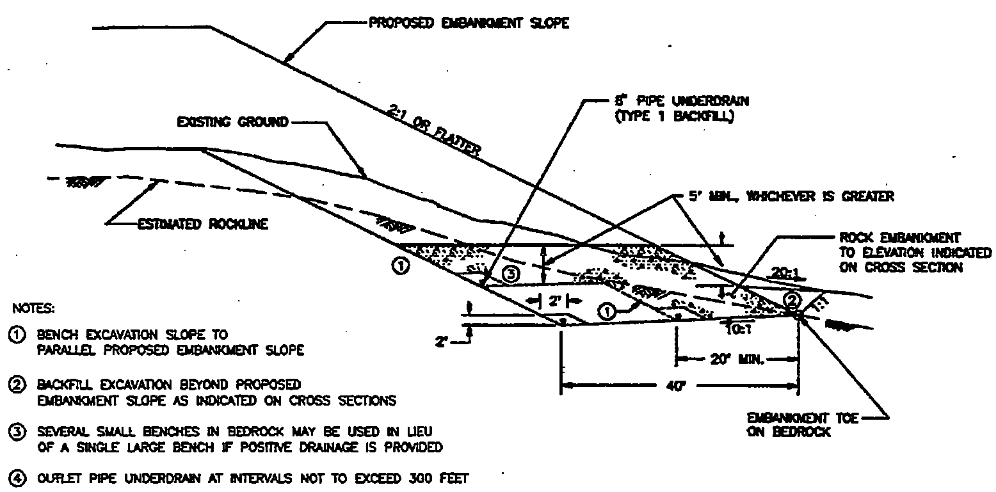
Ruppen, Christopher A.

In looking at the embankment for discussion, the embankment could be considered as three interconnected zones: embankment foundations, embankment materials and roadway subgrade.



Embankment foundation stability was increased by the undercutting of soft, compressible foundation soils and the construction of embankment toe benches. Undercutting occurred typically through alluvial or organic deposits, with limits and depths verified and adjusted as needed by the project field geologist prior to backfilling. Judgments were also made as to the use of undercutting as opposed to bridging/choking soft areas with rock embankment. The bridging capability of this rock was confirmed by constructing test embankments which evaluated their performance in stabilizing the areas.

Toe benches were constructed under numerous embankments as dictated by stability analyses.

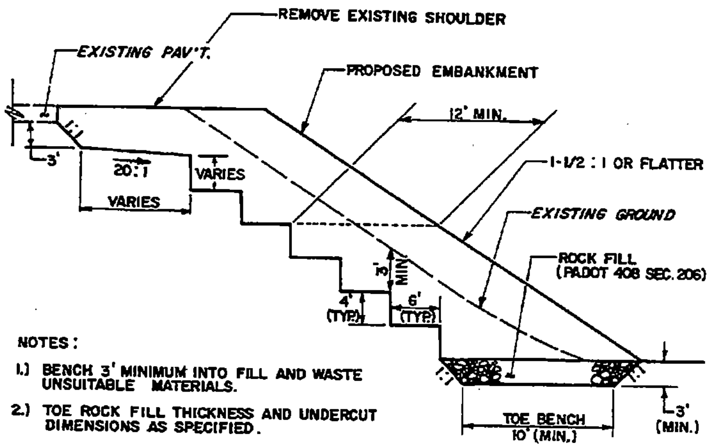


TOE BENCH DETAIL

NOT TO SCALE

Toe benches were excavated to bedrock at the outside edge of the bench (embankment toe area). Once bedrock was encountered on the outside edge of the bench, the remainder of the bench was excavated and configured along this line to meet the detail template. Bench configurations and positions were adjusted by the field geologist typically related to bedrock slope both along the bench and across the bench, bench foundation bedrock type and groundwater conditions. The final excavated toe bench configuration and foundation was reviewed and approved prior to embankment construction.

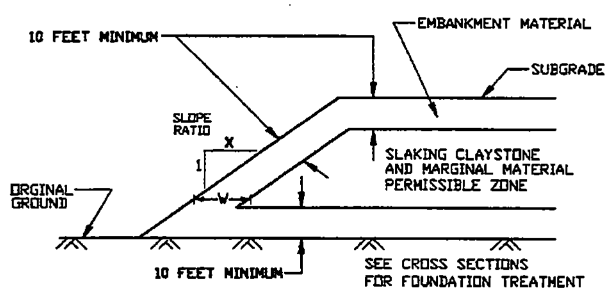
A variation of the toe bench was the case where a "sliver" embankment was added onto an existing embankment as part of roadway widening. Here, a widened embankment bench detail was utilized to key the new embankment into both bedrock and the existing embankment. Review and approval by the field geologist was again provided to ensure proper construction.



TYPICAL EMBANKMENT WIDENING FILL BENCH
FOR FILLS GREATER THAN 15 FEET HIGH
NO SCALE

Review of materials available for use in the embankment accounted for the greatest portion of the project field geologist's time. Starting with the embankment foundation, the contract documents defined three different qualities of rock to be used under different embankment requirements. Embankments steeper than two horizontal to one vertical and undercut and toe bench areas (which would be transmitting intercepted groundwater) required the highest quality rock. This was competent rock which met the requirements of a sodium sulfate soundness test. The next highest quality of rock embankment would be used in most other areas to elevations depicted on the cross sections. This was rock which met the requirements of a physical breakdown test which was conducted on-site. Basically, this rock was required to be either sandstone or limestone which withstood crushing under standard compaction equipment. Small test embankments were constructed near the area of excavation to evaluate the performance of this rock as placed. The third type of rock was placed on top of the other two rock types. This was rock which required ripping or blasting during excavation (siltstone and shale or limestone and sandstone), but broke down under compaction equipment. Slaking claystone (indurated clay) was excluded by the special provisions as rock embankment.

Claystone was governed by a special provision which defined an embankment zone in which slaking claystone could be placed.



X	W-MINIMUM WIDTH (FEET)
1.5	18
2	22
3	32
4	41
5	51
6	61

TYPICAL SECTION
PLACEMENT OF CLAYSTONE AND MARGINAL MATERIAL
IN EMBANKMENTS
N.T.S.

Claystone was subjected to a slake durability test. Slaking claystone was then required to be placed within a zone which was not within ten feet of original ground, the embankment slopes or subgrade. The use of slaking claystone as embankment outside of this zone was excluded.

The field geologist had continuous interaction with other project personnel with regard to the identification, availability and allowable use of these materials for embankment construction. Intimate familiarity of the geologist with the project geology and stratigraphy due to work during the design phases allowed for better utilization of excavated materials during construction. The geologist provided foresight which aided excavation schedules relative to embankment requirements at any particular time. In general, an ample amount of rock was available to meet the three rock types and satisfy the project rock requirements. However, as typical, the rock which was required at the bottom of the embankment was not available at the top of the cut. Furthermore, claystone was readily available as embankment and tended to be more available once the embankment reached the height at which claystone was restricted from use. These were two of the common issues that the geologist handled on a daily basis.

Cuts

Adjustments needed to be made during construction of cuts to remediate problems. Problems on this particular project included several small (or localized) landslides, a highly jointed and weathered bedrock cut slope and surface/subsurface drainage. Landslides generally involved the combination of colluvium or weathered claystone with the addition of either surface or subsurface water. In one instance, a landslide was activated on top of a cutslope by mine subsidence promoted by blasting in the cut area. Referring back to the Local Stratigraphy figure, it can be seen that claystone units are typically associated with each limestone or coal unit. These limestone and coal units locally were the best aquifers.

Excavation through the Pittsburgh limestone and claystone unit was responsible for the greatest frequency of slope instabilities. Three to seven foot thick limestone units with developed joints and fractures confined within a less permeable claystone unit acted as significant local aquifers. When these limestones were intercepted within the cut areas, subsurface water was commonly discharged onto the cut slope. This introduction of water onto the cut slope tended to weather, soften and saturate the underlying claystone unit which leads to eventual slope instability.

The field geologist, aware of the local stratigraphy, was able to evaluate each individual landslide and determine contributing causes of the movement. By identifying the combining factors, remediation plans were developed quickly, and were long-term, esthetically pleasing, maintenance-free and stable solutions. Landslide remediations included:

- Removal of slide mass and flattening of final slope
- Removal of slide mass to bedrock, providing drainage and backfilling with rock embankment
- Benching of cut slope below the limestone units to collect and channel intercepted subsurface flow prior to contaminating underlying softer claystone

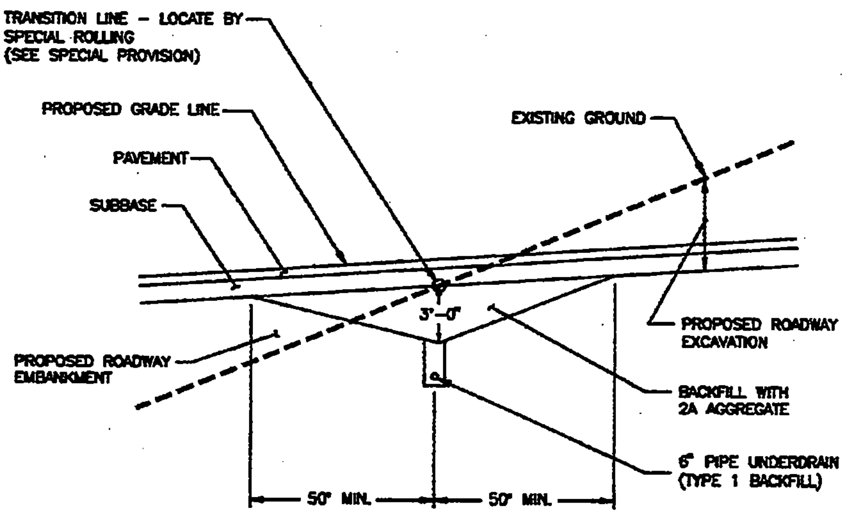
All was performed under the observation of the field geologist. Another area where landslides were activated was in contractor's temporary excavations, typically for haul roads, waste areas or borrow sites. Remediation of these movements typically was the responsibility of the contractor. Review of these repairs still needed to occur to protect the highway agency from maintenance problems.

Another problem which developed related to cutslopes was the presence of a highly jointed sandstone unit at the end of a moderately high cutslope. The well developed joints may very likely have been related to valley stress relief. Blasting isolated large blocks of loose yet inplace sandstone along the joints which would pose future rockfall concern even though a depressed rockfall zone was included as part of the design. Mapping of the joint pattern was performed by the geologist at an attempt to define the limits of the impact. The large loose blocks were removed, the cutslope face was offset further from the roadway (which increased the rockfall zone width) and the cutslope was flattened to minimize the occurrence, size and impact of future rock falls.

Ruppen, Christopher A.

Subgrade

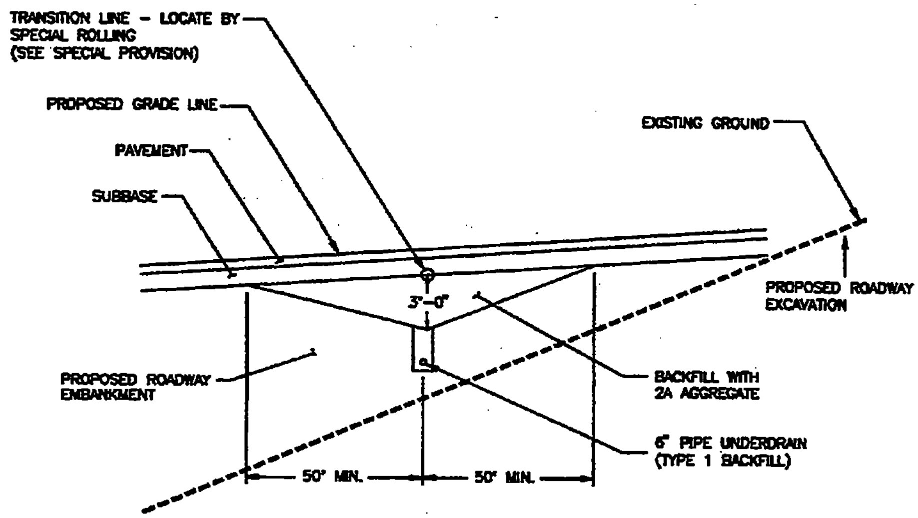
The portion of subgrade requiring the greatest involvement by the field geologist was the transition area between the cut and the embankment. A transition bench detail and special provisions outlined the location and construction of this bench.



TRANSITION BENCH DETAIL

NOT TO SCALE

This detail differed from traditional transition zones in that it applied to both lateral and longitudinal transitions. The bench was to be constructed at the actual transition line which could cross subgrade at any skew. The location of the bench was determined by proof rolling subgrade at the transition plan location and delineating cracking and pumping associated with subgrade instability. Proper position of the transition bench was essential in order for the bench to function. By improperly placing the bench on the embankment side of the transition, the bench underdrain could be suspended in embankment material and likely transmit little subsurface flow since embankment material in this zone was typically clayey soil.



IMPROPERLY POSITIONED TRANSITION BENCH

NOT TO SCALE

This could allow subsurface water to enter under the roadway from the cut side of the bench, soften the original ground and wedge of embankment material overlapping original ground. This softening of subgrade and the material underlying subgrade could potentially promote loss of subbase and eventual pavement failures. When properly positioned, the bench was designed to intercept and transmit subsurface flow prior to contaminating subgrade. Review of the proof rolling operation and the bench excavation by the field geologist occurred to identify proper position and template excavation in an effort to achieve maximum functionability of the bench.

Structures

The field geologist was also involved with structure foundations in that the various foundation types typically were bearing on or within bedrock. Box culverts, large diameter pipes and bridge pier and abutment footings typically were founded on or within bedrock. The verification of the bedrock type and condition at the foundation bearing level prior to construction of the footing was part of the role of the field geologist. Reviewing caisson (abutment and walls) and pre-drilled H-pile (walls) drilling occurred to ensure that required bedrock socket lengths were achieved. Driven H-pile lengths were compared to actual design borings to confirm pile tip bearing level and refusal depth. Rock embankment requirements were modified to allow for unrestricted pile driving through embankment areas. All of this occurred with review by the field geologist.

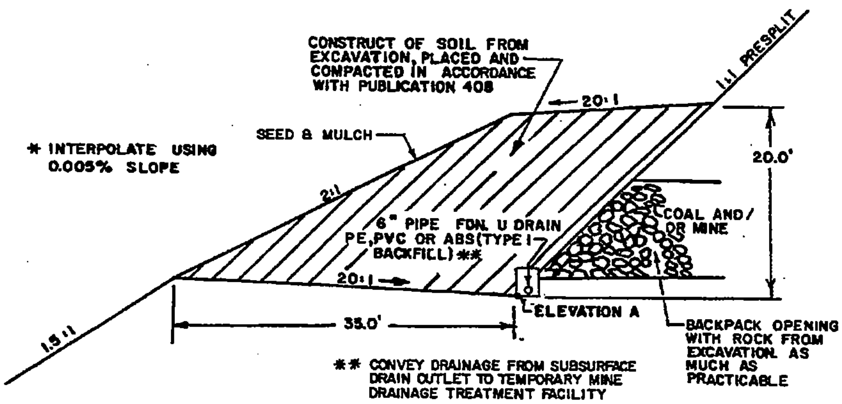
Special Problems

The most significant special problem encountered during design and construction involved the presence of the Pittsburgh coal within the project. The coal measured as thick as thirteen feet (including two roof coals) and was extensively mined by both surface and deep mine methods. During design, an actively burning mine fire was identified which lead to a subsequent line shift to avoid encroaching into the burning mine. Roadway grades were also shifted to minimize encroachment into old strip mine areas which had been utilized as landfills.

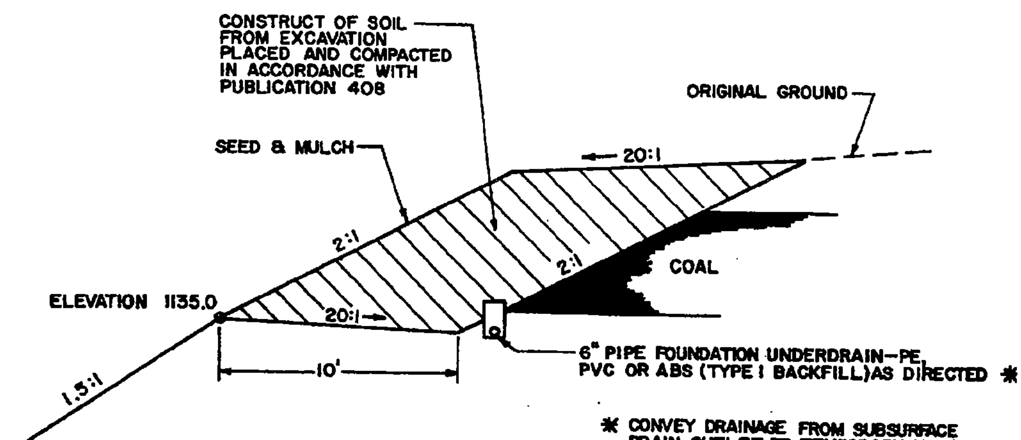
The roadway line and grade was established such that the coal would be present above grade in all instances, with one exception. Relocation of a sideroad could not be situated below the mine, so the mine was undercut and removed from under the future sideroad. With roadway grades established below the coal and associated mines, this required the development of the cuts through the mining areas. Three larger isolated coal areas were intercepted by mainline. Piezometers installed on the floor of the mines during the design phase identified mine pools for each mine. One of the three mines was controlled by a localized structural fold on the base of the coal which placed the lowest portion of the mine near the centerline of the mainline roadway. At this location, a mine pool was identified and as much as ten feet of water was measured in piezometers during design.

The contractor was required to dewater the mine by pumping prior to development of the cut. Since this water did not meet the environmental agency's standards for discharging into the watershed, temporary mine drainage treatment facilities were constructed which utilized soda ash and potassium permanganate briquettes combined with aeration to treat the water. Involvement by the field geologist occurred to aid in identifying optimum pumping locations to start the mine pool drawdown. This was essential because of the project schedule and the restraints which allowed for no cut development until the mine was dewatered. No cut development in turn meant no material availability for embankment construction. A suitable pumping location was developed which lead to the subsequent pumping and treatment of several million gallons of mine water prior to development of the cut.

Once the mine was dewatered, excavation in the cuts began which also allowed for start of embankment construction. Cuts were lowered to the base of the coal at which time a bench was excavated for construction of mine/coal seals. Various configurations of seals were constructed depending upon the proximity of the coal/mine to the top of the cut.



TYPICAL SECTION
MINE SEAL



TYPICAL SECTION
COAL SEAL

Mine openings or headings were cleaned out and backpacked with rock. Underdrains were placed on the inside of the bench and at the toe of the coal/mine to intercept any seepage from the mine. This would ideally inhibit future build up of a new mine pool which could force a failure in the seal. A mine seal embankment was then constructed from soil on the bench and against the coal/mine face. Application of these details along with the associated special provisions required significant interaction among construction inspection staff, contractor personnel and the field geologist.

Another item related to the mining, and a very simple item if one was familiar with the project geology, was the acceptable location of burn pits. Contractors would typically burn the debris from the clearing and grubbing operations in large pits. Fires would be ignited and kept smokeless through the use of large blowers. In two occasions, pits were excavated into the roof of the coal seam and ignited. Upon review by the field geologist, inspection and contractor personnel were alerted to the potential hazards of developing a new mine fire. As a result, pits were abandoned and reestablished below the coal crop line.

Another special problem which required involvement by the field geologist was blasting. Both production blasting and presplit blasting occurred on this project, with a greater involvement and emphasis placed on the presplit. For presplit blasting, the specifications required that a test pattern be performed to determine optimum spacing, size and loading of holes. The field geologist was involved in the review of the test patterns to identify the combination which would produce the most stable slope. The actual presplitting operation and presplit face were reviewed to recommend modifications to the pattern as warranted by varying bedrock conditions.

Other special problems which required involvement by the field geologist were:

- Channel relocations
- Deep mine grouting to minimize impact of blasting

- Erosion and sedimentation devices
- Sedimentation and detention pond foundations
- High mast lighting pole foundations
- Sign structure foundations

These are listed to further highlight the value of providing personnel intimate with the design and the project geology as part of the construction management team.

CONCLUSION

Through this project, the value of retaining qualified geotechnical personnel as part of the construction management team was clearly demonstrated. Baker's geologists were able to clarify geotechnical details and special provisions which aided in expediting the project and minimize delays while increasing the overall project quality. Similar participation by geotechnical personnel in the construction process should become standard in the highway development and construction process.