Proceeding of the 40th Highway Geology Symposium

STATE OF ALABAMA HIGHWAY DEPARTMENT

BIRMINGHAM, ALABAMA
MAY 17-19, 1989

PROCEEDING OF THE 40TH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

SYMPOSIUM VENUE

MAY 17-19, 1989

SPONSORED BY
ALABAMA HIGHWAY DEPARTMENT

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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 was organized and held its first meeting on February 16, 1950, in Richmond, Virginia. Since then, 40 consecutive annual meetings have been held in 27 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

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

The Wyoming Highway Department hosted the 1973 meeting in Sheridan. From there it moved to Raleigh, North Carolina in 1974, back west to Coeur d'Alene, Idaho in 1975, Orlando, Florida in 1976. Rapid City, South Dakota in 1977, and then back to Maryland in 1978, this time in Annapolis. Portland, Oregon was the site of the 1979 meeting, Austin, Texas in 1980, and Gatlinburg, Tennessee in 1981. The 1982 meeting was held in Vail, Colorado, and in Stone Mountain, Georgia in 1983. The 35th meeting in 1984 was in San Jose, California, the 36th HGS was in Clarksville, Indiana, and the 37th meeting was held in Helena, Montana. Pittsburgh, Pennsylvania was the site of the 38th meeting, Park City, Utah hosted the 39th, and this year the 40th annual Highway Geology Symposium was held in Birmingham, Alabama.

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 form state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contributions to the symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. 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. Some of these committees are: By-laws, Public Relations, Awards Selection, and Publications. The lack of rigid requirements, routing, and the relatively 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 pro tem of the Steering Committee. Depending on interest and degree of participation, the temporary member may gain full membership to the Steering Committee.

The symposia are generally 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. 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 pincipally with mining activities; North Carolina provided stops at a quarry 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 interaction of fault activity, urban landslides, and coastal erosion with the planning, construction, and maintenance of transportation systems. In 1985 the one day trip illustrated new highway construction procedures in the greater Louisville area. The 1986 field trip in Montana provided valuable information concerning geology associated with road construction in mountainous conditions. The 1987 field trip in Pittsburgh emphasized highway construction in a northeastern urban area with challenging geological conditions and informative geotechnical solutions. The Utah field trip in 1988 emphasized local geology and geological hazards of highway and dam construction and rock fall, landslide, and snow avalanche problems. Local geology of the Birmingham area and geotechnical problems of bridge maintenance were some of the highlights of the 1989 field trip.

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 these proceedings are out of print, but copies of most of the last sixteen proceedings may be obtained from the treasurer of the Symposium, David Bingham, of the North Carloina Department of Transportation in Raleigh 27611. Costs generally range from $5.00 to $15.00, plus postage.

Highway Geology Symposium

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Paul Price	1970
K. B. Woods	1971
R. J. Edmonson	1972
C. S. Mullin	1974
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Henry Mathis	1982
David Royster	1982
Terry West	1983
Dave Bingham	1984
Vernon Bump	1986
C. W. "Bill" Lovell	1989

In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis Indiana designed a plaque to be presented to individuals who have made significant contributions to the Highway Geology Symposium over a period of years. The award, a 3.5" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

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TABLE OF CONTENTS

TITLE PAGE. i

HIGHWAY GEOLOGY SYMPOSIUM, History, Organization, and Function. ii

MEDALLION AWARD WINNERS v

STEERING COMMITTEE OFFICERS vi

STEERING COMMITTEE MEMBERS. vii

EMERITUS MEMBERS. x

STANDING COMMITTEES xi

CHAIRMAN OF LOCAL ARRANGEMENTS FOR UPCOMING SYMPOSIUMS. xii

TABLE OF CONTENTS xiv

PAPERS PRESENTED AT THE 40TH SYMPOSIUM

GENERAL OVERVIEW OF THE GEOLOGY AND NATURAL RESOURCES OF ALABAMA 1
Bennett L. Bearden, Geological Survey of Alabama

ENGINEERING GEOLOGY AND GEOTECHNICAL ENGINEERING FOR A PRELIMINARY ROUTE ALIGNMENT STUDY FOR 25 MILES OF ARIZONA STATE ROUTE 87. 16
Thomas H. Walker, Project Geologist, Sergent, Hauskins & Beckwith Geotechinical Engineers Inc.

SUITABILITY OF BOTTOM ASH FOR INDIANA HIGHWAY CONSTRUCTION 37
Wei-Hsing Huang and C. William Lovell, Purdue University

GEOGRID REINFORCEMENT FOR COCHRANE BRIDGE EMBANKMENT. 55
Larry Lockett, Alabama Highway Department and Robert M. Mattox, Tensar Construction Products, Inc.

UPDATE ON STABL...PCSTABL5M 64
E. Achilleos and C. William Lovell, Purdue University

DEMOLITION AND REMOVAL OF STRUCTURES PRIOR TO LAND RECLAMATION. 74
Terry R. West and Quentin A. Gordon, Purdue University

SLOPE FAILURES ON HIGHWAY EMBANKMENTS HIGH IN SHRINK-SWILL CLAYS: PREVENTION AND REPAIR 111
Scott F. Burns, William O. Hadley, Jack W. Mutchler, and Schaun M. Smith, Louisiana Tech University, Paul M. Griffin, Jr., Louisiana Transportation Research Center

DESIGN APPLICATIONS OF THE WELDED WIRE WALL 131
K. Jeff Nelson and John R. Selvage, Selvage, Heber, Nelson & Assoc.

INTEGRATED SLOPE STABILITY ANALYSIS USING MICROCOMPUTERS. 148
Sunil Sharma, University of Idaho

HONDURAS HIGHWAY GEOLOGY. 160
Carlo Hugo Rivera Morales, SECOPT Geological Advisor,
Departamento De Investigaciones Geologicas

CONTRACT SPECIFICATION OPTIONS FOR RETAINING WALL DESIGN AND CONSTRUCTION:
DISCUSSION OF CONSTRUCTION ALTERNATES AND CASE HISTORIES 172
John R. Wolosick, Chief Engineer, National Foundation Company, Inc.

FINITE ELEMENT ANALYSIS OF A RIB-REINFORCED STEEL CULVERT 198
Sunil Sharma and James H. Hardcastle, University of Idaho

FIRST WIRE ROPE NET ROCKFALL PROTECTIVE BARRIER INSTALLED AT THE
GRAND CANYON NATIONAL PARK 216
Robert A. Thommen, Jr., Brugg Cable Products, Inc.

SPECIAL TREATMENT OF MINE OPENINGS IN ROCK CUT SLOPES 223
E. M. Wright, Kentucky Department of Highways

RECENT HIGHWAY TUNNEL PROJECTS IN THE APPALACHIAN MOUNTAINS 241
Richard Humphries and Randy Sullivan, Golder Associates Inc.

ATTENDANCE LIST, 40TH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM. 255

EXHIBITORS AND SPONSORS 259

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GENERAL OVERVIEW OF THE
GEOLOGY AND NATURAL RESOURCES OF ALABAMA

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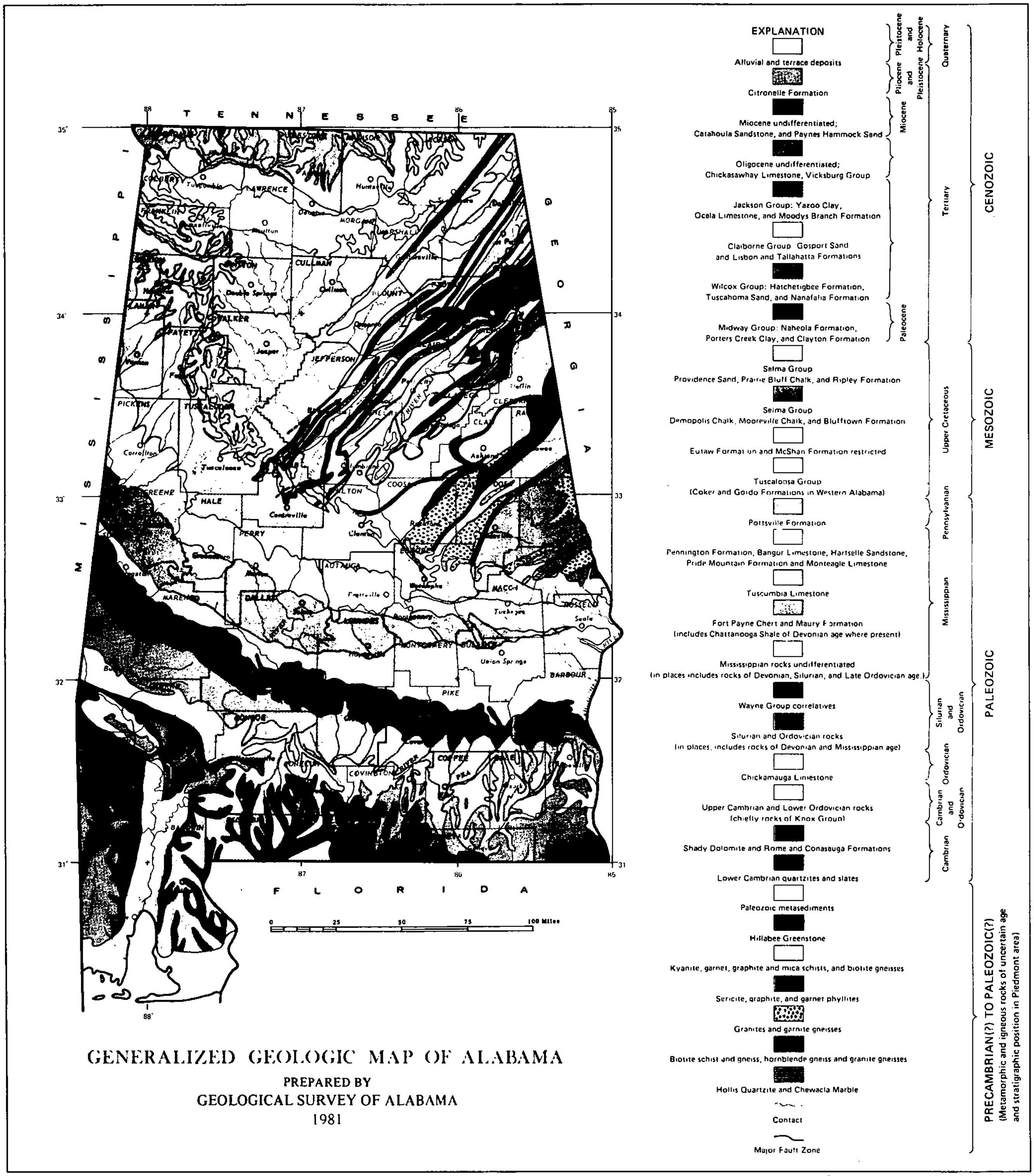
Alabama is blessed with geologic diversity (Figure 1) and as a result, a wealth of natural resources including abundant energy resources, diverse mineral deposits and plentiful supplies of good quality water. Alabama's geology and natural resources have played a key role in the economic development of the State. For example, the City of Birmingham owes its birth to geology and the red iron ore, limestone, and high quality coal that occur within its boundaries. As growth in Alabama continues, we will witness the construction of new highways, roads, bridges, dams; the building of housing and industrial facilities; and the development and expansion of towns and cities. The natural resources of the State are the basis for this growth and the viability of many key aspects of the southeastern United States socioeconomic system.

Alabama's multitude of natural resources include oil and gas, coal, limestone, chalk, sand, gravel, building stone, and clay. In the United States, Alabama ranks 15th in the production of liquid hydrocarbons, 14th in the production of natural gas and 11th in coal production. Alabama ranks 19th in the value of produced nonfuel minerals for 1988. Alabama ranks nationally in the production of recovered sulfur. Salt deposits underlie much of southwest Alabama. Alabama is a leader in the development and production of coalbed gas resources, an energy source which appears to be a vital part of the State's future energy supply.

In addition to the better known energy mineral resources, other resources include lignite, tar sand, oil shale, tripoli, graphite, mica, bauxite, sulfur, marble, and some strategic metallic minerals. Alabama ranked 2nd in the nation in production of bauxite and ferroalloys and 3rd in fire clay and bentonite for 1988.

Alabama's water supply is perhaps the most important natural resource in the State. Rainfall ranges from a low of 48 inches to a high of 68 inches in various parts of the state during an average



Modified from U.S. Geological Survey, 1932, Geologic map of the United States

Figure 1.--Geologic map of Alabama.

year. Many municipalities depend on ground water either as the primary or the sole water source. Surface water provides additional water for domestic consumption and for industrial use, transportation, power generation, and recreation.

Physiographically Alabama is divided into five regions which are differentiated on the basis of topographic relief, rock types, and geologic structure. From north to south these include the Interior Low Plateaus, Appalachian Plateaus, Valley and Ridge, Piedmont and Coastal Plain provinces (Figure 2). Generalized descriptions of each of the regions and the associated mineral resources are presented below. A generalized stratigraphic column of the regions within the state is depicted in figure 3.

Most of north Alabama is divided on the basis of physiography into two provinces, the Appalachian and Interior Low Plateaus (Figure 2). The Interior Low Plateaus to the north is primarily a limestone plateau of moderate relief. To the south and east is the Appalachian Plateaus, which comprises submaturely to maturely dissected sandstone and shale synclinal plateaus having moderate relief. In the eastern part of the Appalachian Plateaus are three linear anticlinal limestone valleys (Murphrees Valley, Wills Valley, and Sequatchie Valley) characterized by the presence of resistant sandstone ridges and moderate relief (Figure 4).

Coal mining is the largest mining industry in Alabama, and two of Alabama's four coal fields underlie the Appalachian Plateaus province. The Warrior coal field (Figure 5) to the south is the most productive of the four fields and contains 90 percent of the state's coal reserves in more than 20 coal beds. Both surface and subsurface mining are used to recover the coal. The Plateau coal field (Figure 5) to the north contains more than 25 coal beds; however, because of geologic complexity utilization of reserves has been limited.

Oil and gas (Figure 6) are currently being produced from several sandstone and limestone units in Ordovician, Devonian, Mississippian, and Pennsylvanian rocks of the Black Warrior basin. Most production is in Fayette and Lamar Counties. Gas is also being produced from coal beds in the Pottsville Formation (Pennsylvanian) in Tuscaloosa and Jefferson Counties.

Asphalt impregnated sandstone occurs in the Mississippian Hartselle Sandstone and Pride Mountain Formation in northwest Alabama. Reserves in the Hartselle have been estimated at about 3

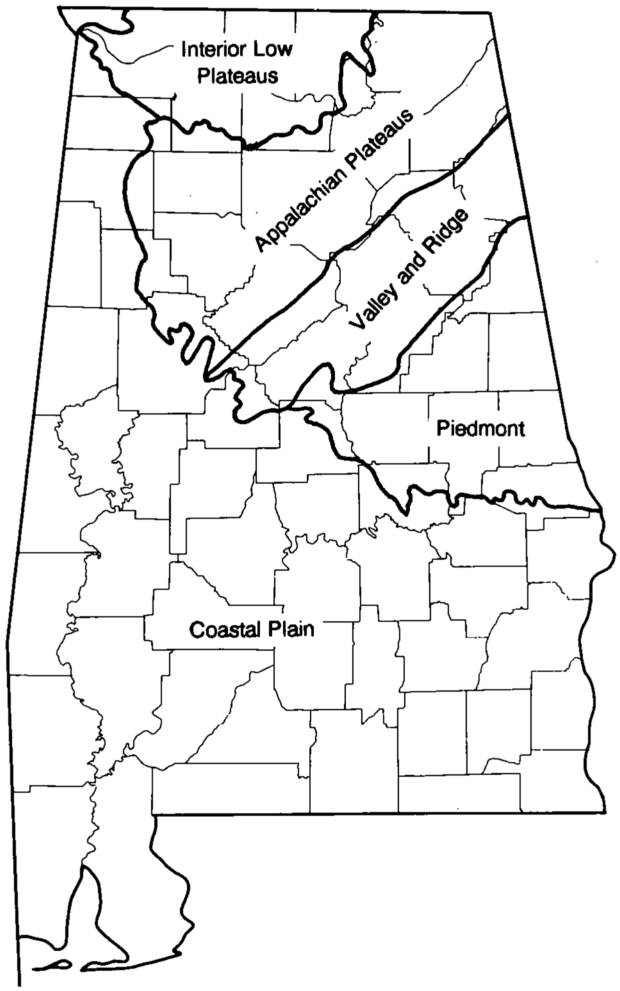


Figure 2.--Physiographic provinces of Alabama.

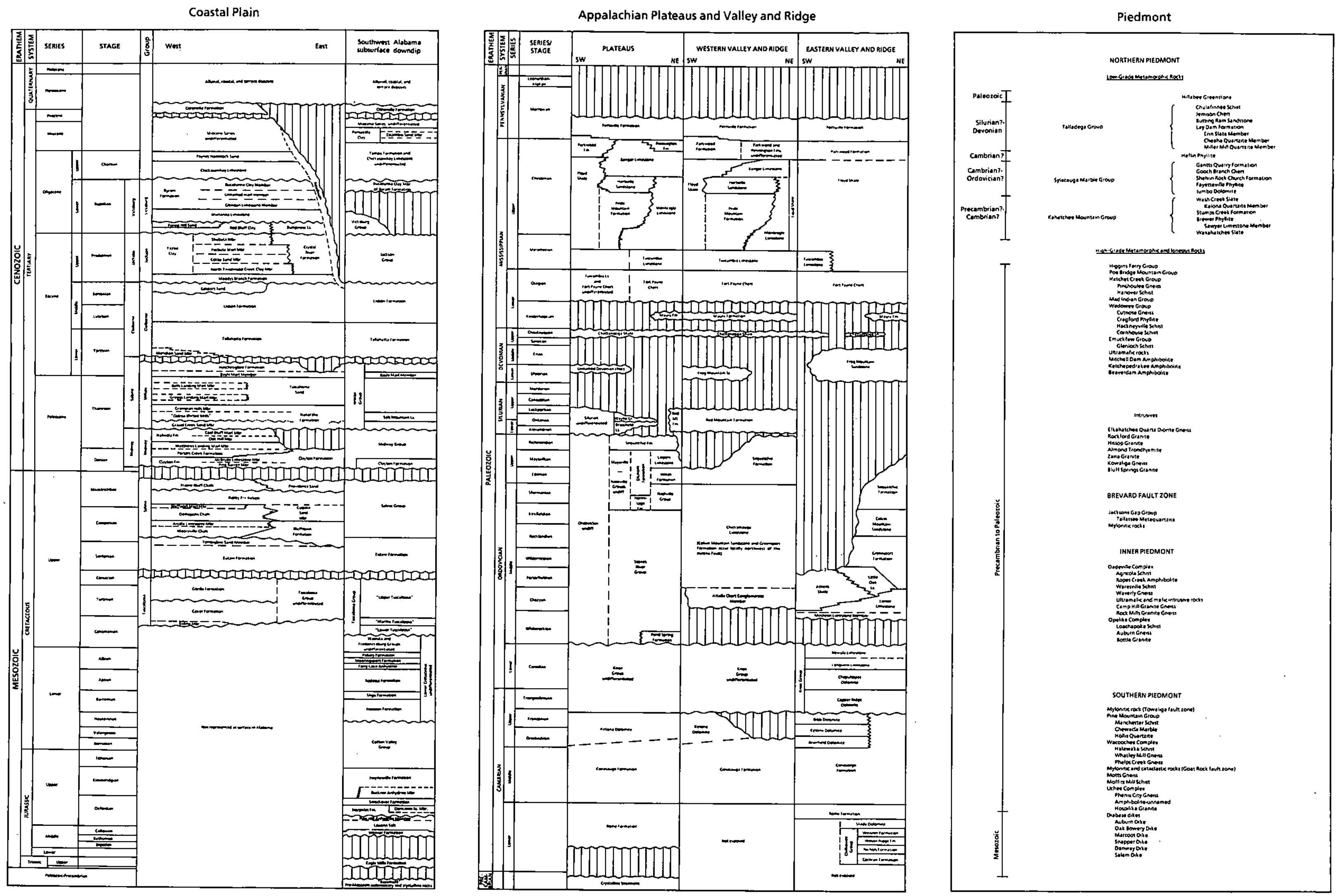


Figure 3.--Alabama stratigraphy

Dorothy E. Raymond and Charles W. Copeland

1988

Cartography by Bonita Brennan

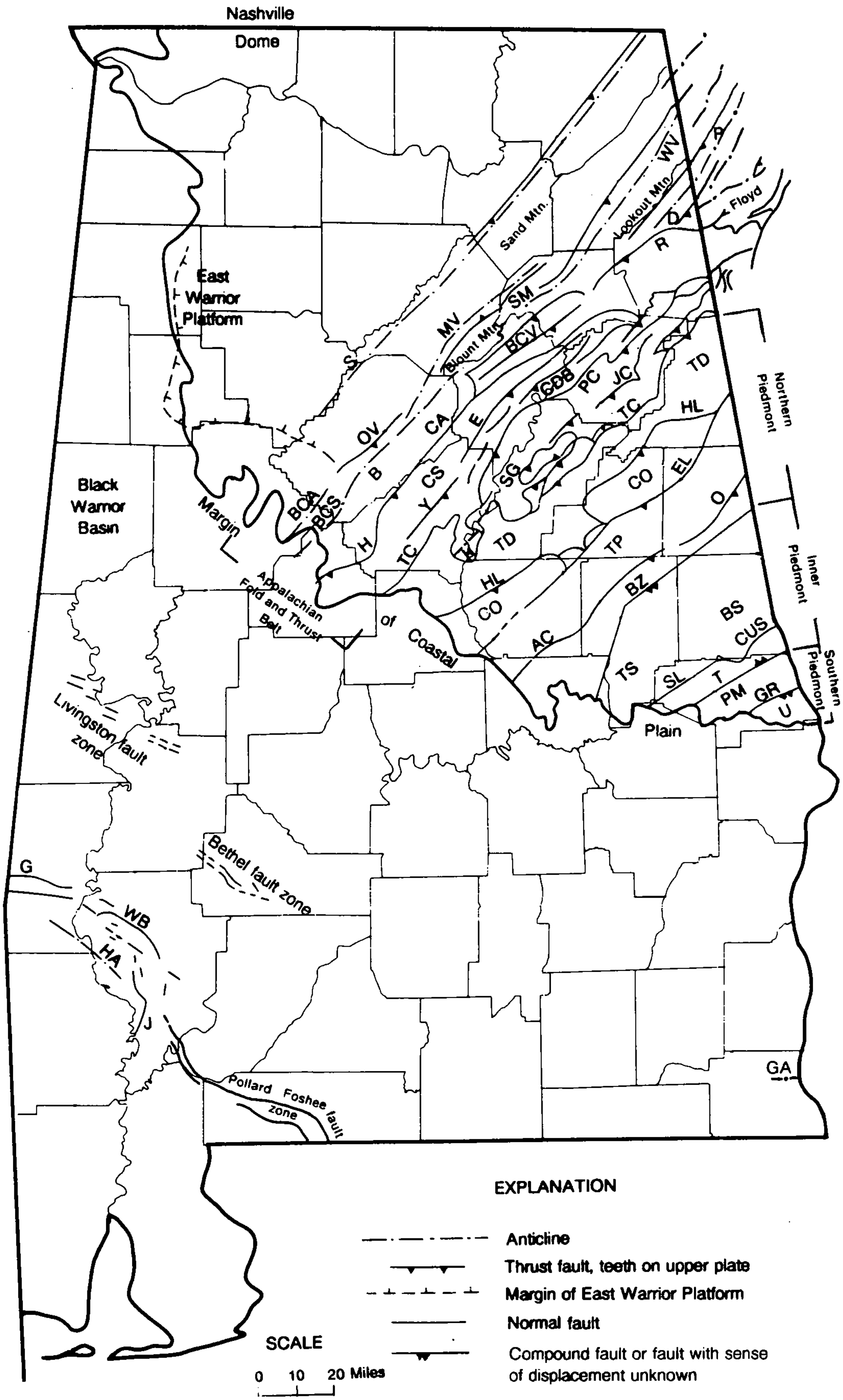


Figure 4.--Generalized structural geology map of Alabama.

EXPLANATION OF SYMBOLS FOR FIGURE 4

AC	Alexander City fault
B	Birmingham anticlinorium
BCA	Blue Creek anticline
BCS	Blue Creek syncline
BCV	Big Canoe Valley fault
BS	Boyds Creek synform
BZ	Brevard fault zone (includes Abanda and Katy Creek faults)
CA	Cahaba synclinorium
CDB	Coosa deformed belt
CO	Coosa block
CS	Coosa synclinorium
CUS	Cusseta synform
D	Dirtseller Mountain syncline
E	Eden fault
EL	Enitachopco line fault system
G	Gilbertown fault zone
GA	Gordon anticline
GR	Goat Rock fault zone
H	Helena fault
HA	Hatchetigbee anticline
HL	Hollins Line fault
J	Jackson fault
JC	Jacksonville fault complex
K	Kelley Mountain anticline
MV	Murphrees Valley anticline
O	Omaha fault
OV	Opossum Valley fault
P	Peavine anticline
PC	Pell City fault
PM	Pine Mountain block
R	Rome fault
S	Sequatchie anticline
SG	Sleeping Giants klippe
SL	Stonewall line
SM	Straight Mountain fault
T	Towaliga fault zone
TD	Talladega block
TF	Talladega-Cartersville fault
TP	Tallapoosa block
TS	Tallassee synform
U	Uchee block
WB	West Bend fault zone
WV	Wills Valley anticline
Y	Yellowleaf fault

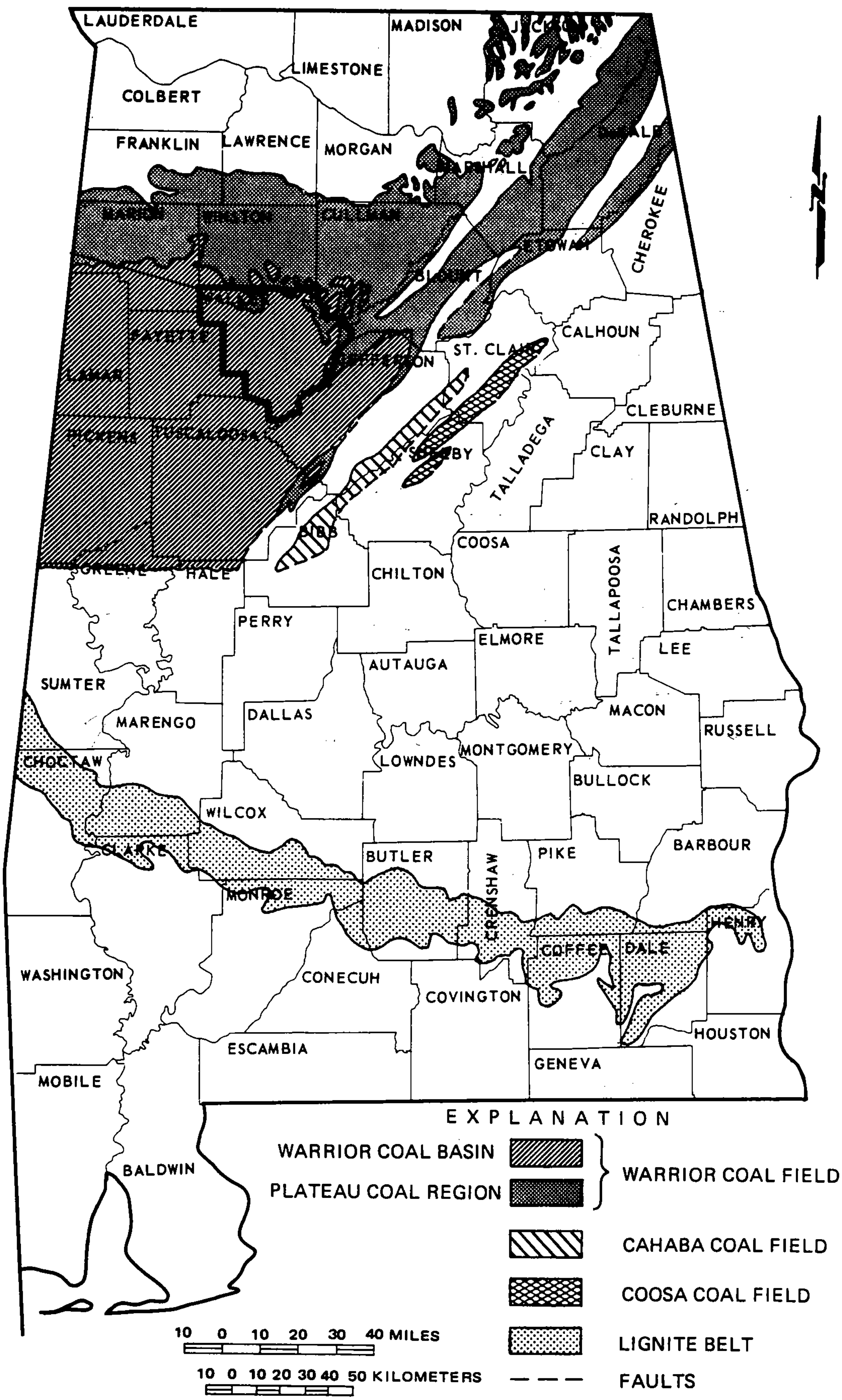


Figure 5.--Coal fields in Alabama

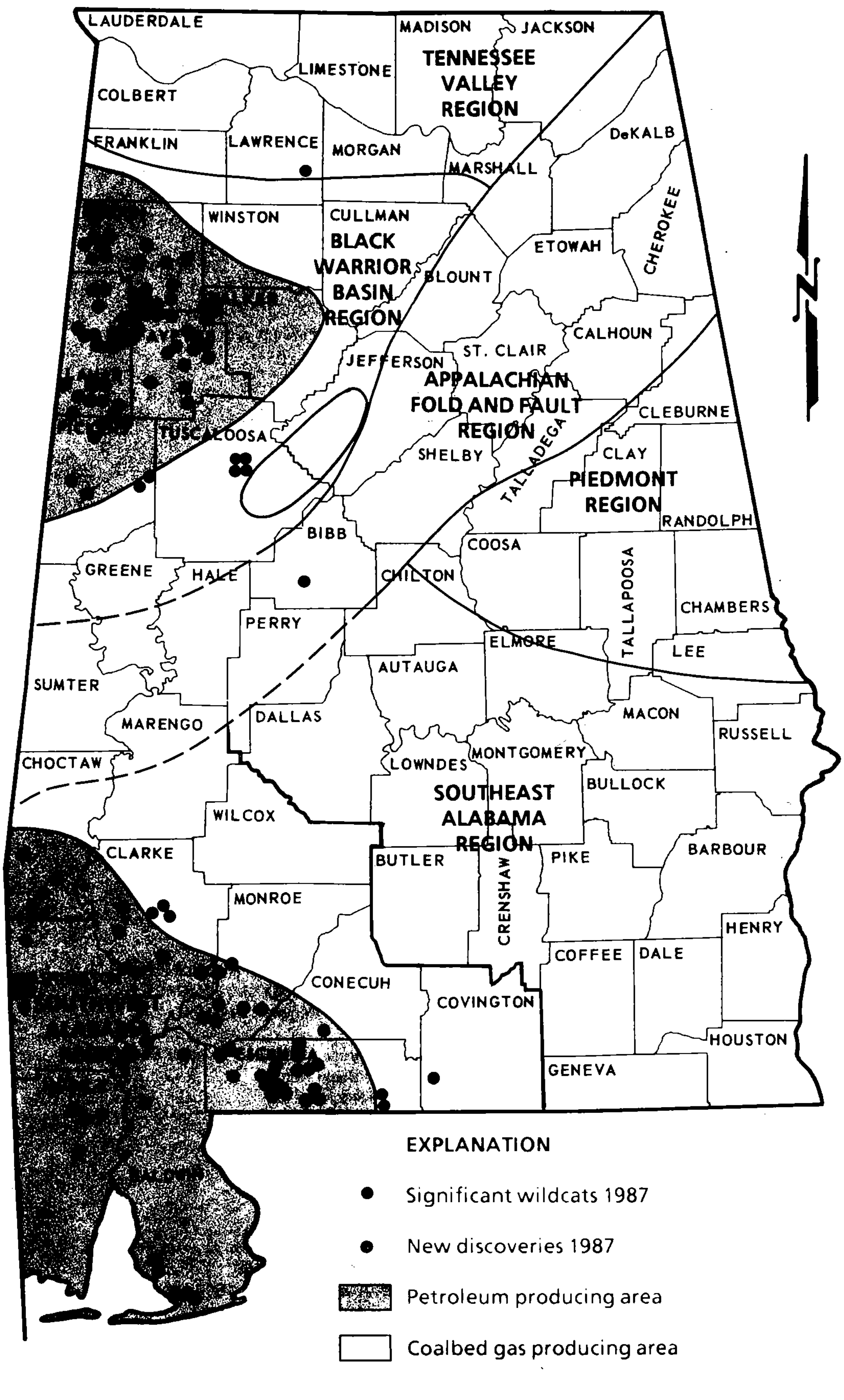


Figure 6.--Oil and gas regions in Alabama.

billion barrels of oil in place. To date, commercial development has been limited; however, improvement in extraction technology may lead to future development of this resource. Oil shale of the Chattanooga Shale also holds promise as a future source of energy for Alabama. Limestone is presently being quarried for aggregate and dimension stone from the Bangor, Monteagle, and Tuscumbia Limestones in the Plateaus. Other mineral resources in the Plateaus include asphaltic limestone, clay, bauxite, tripoli, iron ore, and sand and gravel.

The Valley and Ridge physiographic province in central and northeastern Alabama consists of a series of subparallel ridges and valleys trending generally northeast-southwest (Figure 2). This characteristic topography is developed on folded and thrust-faulted sedimentary rocks. The ridges are formed by sandstone and chert beds that are resistant to erosion; valleys are underlain by less resistant shale and carbonate rocks. The northwestern half of the province shows well-developed Valley and Ridge topography. The southeastern part of the province is characterized by a wide plain of varied relief containing irregularly spaced parallel ridges and valleys. In the extreme northeastern part of the province, mountainous terrain is developed on faulted and folded sandstone and quartzite.

The Valley and Ridge physiographic province in Alabama corresponds in part to the fold and thrust structural belt of the southern Appalachian orogen. The fold and thrust belt is defined as the belt of folded and thrust-faulted Paleozoic sedimentary rocks bounded by Paleozoic and Precambrian metamorphic rocks on the southeast (Piedmont) and by relatively undeformed Paleozoic sedimentary rocks of the Black Warrior foreland basin on the northwest.

The Valley and Ridge province contains two of Alabama's four coal fields: the Cahaba coal field (Cahaba synclinorium) (Figure 5) and the Coosa coal field (Coosa synclinorium) (Figure 5). The coal beds are primarily in the Pennsylvanian Pottsville Formation, although the Parkwood Formation contains a few, thin coal beds. The coal is generally of good quality and locally coal of coking quality is present in the Coosa field. Because of structural complexity the fields have not been extensively mined. The coal fields offer some potential for coalbed methane production.

Very few wells have been drilled for oil and gas in the Valley and Ridge province in Alabama. Since 1979 several deep holes have been drilled in the Valley and Ridge and its extension beneath the Gulf Coastal Plain; all are dry and abandoned. Potential source and reservoir rocks have been identified in the Middle and Upper Ordovician of the Valley and Ridge, but additional evaluation is needed to define the petroleum potential of this area.

The Valley and Ridge province contains a number of other mineral commodities. The largest non-energy mineral industry in the state is the quarrying of limestone. Limestone is presently mined from the Conasauga Formation, Longview Limestone, Newala Limestone, Lenoir Limestone, and Little Oak Limestone in the Valley and Ridge province. Other construction minerals produced from the Valley and Ridge during 1988 include dolomite, shale, clay, chert, and sand and gravel. Other mineral resources that are not presently commercially produced in the province include barite, red iron ore (hematite), brown iron ore (limonite), tripoli, manganese, lead, zinc, and bauxite.

The crystalline rocks of the Alabama Piedmont are subdivided into three lithotectonic provinces: Northern Piedmont, Inner Piedmont, and Southern Piedmont (Figure 4). Each province is bounded by a major regional fault, and each includes distinct lithostratigraphic and/or lithodemic units. Metamorphic grade generally increases across the Piedmont from low-grade greenschist facies on the northwest to high-grade migmatite facies on the southeast.

The crystalline rocks occur in the Piedmont Upland section of the Piedmont physiographic province of Alabama (Figure 4) which is divided into two districts: the Northern Piedmont Upland district and the Southern Piedmont Upland district. Elevations in the Northern Piedmont range from approximately 1,000 feet in the northwest to 500 feet in the south. Several prominent ridges subdivide the district's topography. The highest are the Rebecca and Talladega Mountains, including Cheaha Mountain (elevation 2,407 feet), which form a prominent system of mountains along the northwest side of the district. Elevations in the Southern Piedmont district are much more subdued, ranging from approximately 800 feet on the north to approximately 500 feet in the south. The major drainage system has incised the topography about 200 feet. The boundary between the two districts coincides with the approximate location of the Brevard fault zone (Figure 4).

A variety of metallic and nonmetallic minerals occur associated with the rocks of the Alabama Piedmont. Historically, only a few of these minerals have been successfully exploited. Most of the known mineralization occurs in the Coosa and Tallapoosa structural blocks of the Northern Piedmont although some rocks and minerals of economic use occur in other parts of the Piedmont.

Metallic mineralization in the Alabama Piedmont is concentrated in the Northern Piedmont. Gold and copper-pyrite have been produced in limited quantities from small scattered deposits since the 1840's. The principal gold-bearing areas are found in southern Cleburne, Clay and Tallapoosa Counties, although isolated occurrences of gold have been found in all counties. Tin was produced from a pegmatite-rich area of central Coosa County between 1940 and 1942. Lead, zinc, arsenic, columbium-tantalum, tungsten, molybdenum, vanadium and manganese also occur in the Northern Piedmont, but in smaller quantities than copper-pyrite, and are often associated with each other. Metallic mineralization in the Inner and Southern Piedmont parts of the Alabama Piedmont is poorly known. Pyrite, magnetite and chromite have been recognized in several small deposits associated with ultramafic rock but no production is known.

Nonmetallic or industrial minerals produced from rocks of the Alabama Piedmont include mica, feldspar, talc, anthophyllite asbestos, marble and beryl. Other nonmetallic minerals found in the Piedmont include uranium, kyanite, silica, kaolin and barite. Graphite production was one of the largest mineral industries in the Northern Piedmont for more than 40 years. Mica production spanned more than 100 years, and the principal mining activities were centered in Clay and Randolph Counties of the Northern Piedmont and Tallapoosa and Lee Counties of the Inner Piedmont. Feldspar and beryl were obtained as byproducts of mica production. Calcium products are produced from marble quarried from the Sylacauga Marble Group. Talc has been mined from metamorphosed dolomite in Talladega County, and talc and asbestos have been mined from ultramafic rocks in Tallapoosa County.

Construction materials represent the major mineral extraction activity in the Alabama Piedmont. Marble, crushed stone and sand and gravel are produced from a variety of sources. Dimension stone was produced from the Sylacauga Marble Group in Talladega County and crushed dolomitic and calcitic marble is produced from marble bodies in Talladega and Lee Counties. Crushed stone, principally granite and granitic gneiss, has been produced from a variety of different granitic bodies scattered throughout the Piedmont. Most production is intermittent, depending on local demand. Sand and gravel are produced principally from the flood plain of the Tallapoosa River in Cleburne County. Smaller deposits on secondary streams supply local markets.

The East Gulf Coastal Plain physiographic section of the Coastal Plain province (Figure 2) in Alabama is an area of Mesozoic and Cenozoic sediments occupying the southern part of the state and parts of the western tier of counties where mostly unconsolidated sediments of the Coastal Plain overlap consolidated rocks of the Plateaus, Valley and Ridge, and Piedmont provinces. In Alabama, the strike belts of the Coastal Plain sediments trend east-west in eastern and central parts of the state, trend northwest in western Alabama, and trend northward from Lamar County as part of the regional trend of stratigraphic units along the eastern flank of the Mississippi embayment.

In northwestern Alabama, Coastal Plain sediments cap hills and plateau remnants of Paleozoic rock and are usually from 50 to 1,000 feet thick. The sediments thicken rapidly to the south and near the coast are probably more than 24,000 feet thick where they overlie metamorphic or igneous rocks. In southeastern Alabama, Coastal Plain sediments are about 8,000 feet thick and overlie Mesozoic volcaniclastic rocks, metamorphic rocks, and a small area of unmetamorphosed Paleozoic rocks.

Most of the Coastal Plain is an area of low to moderate relief, and the topography is considerably more gentle than that of the bordering regions. However, stream valleys commonly have 200 to 400 feet of relief. Resistant beds in some of the Cretaceous formations form broadly arcuate cuestas (ridges) that rise from 50 to 200 feet above the surrounding prairie floors in parts of central Alabama. Further south, in parts of Choctaw, Clarke, and Monroe Counties are hills underlain by resistant parts of Eocene formations that interrupt the otherwise gentle slope of the land surface to the Gulf. Gently rolling landscapes and broad flat areas are common near the southeastern corner of Alabama where the formations are composed mainly of easily eroded sediments.

The Coastal Plain is a source of a wide variety of raw-material products and fossil fuels. All the materials necessary for construction are abundant in the area. Sand and gravel are mined in almost every county in south Alabama. Much of this sand and gravel is used in road construction and as aggregate within Alabama and is also transported to adjoining states. Calcareous rocks suitable for the manufacture of cement are present in marl of the Selma Group and the limestone formations of Eocene and Oligocene age in southwest Alabama. Limestone of the region has also been used widely for soil conditioning. Clay minerals such as bentonite, zeolite, high-alumina clay, and kaolin also occur in the Coastal Plain. Clay suitable for brick and tile manufacture occur in western Alabama and high-alumina clay, including kaolin, used for refractories occur in the southeastern part of the state. Bentonite used as bond for foundry sand and for absorbents is mined in Lowndes County. Expansive clay for aggregate and pet supplies occurs in the Porters Creek Formation in Choctaw, Marengo, Sumter and Wilcox Counties. Zeolite minerals are common constituents of the Clayton, Nanafalia and Tallahatta Formations. Brine for industrial chemicals is solution-mined from a shallow salt dome near McIntosh in Washington County.

Fossil fuels produced in south Alabama include oil, gas, and gas condensate. Crude oil is produced from formations of the Jurassic in Baldwin, Choctaw, Clarke, Escambia, Mobile, Monroe and Washington Counties; from Lower Cretaceous formations in Baldwin and Mobile Counties; and from the Upper Cretaceous formations in Baldwin, Choctaw, Clarke, and Escambia Counties. Natural gas is produced from formations of Jurassic age in Baldwin, Choctaw, Clarke, Escambia, Mobile, Monroe, and Washington Counties. In addition, large quantities of natural gas are being discovered in the Norphlet Formation in Mobile Bay and adjoining areas offshore. Deep wells in the Smackover and Norphlet Formations in Escambia, Mobile and Washington Counties yield large quantities of gas condensate. Sulfur, propane, butane, and natural gasoline are byproducts of the cleansing and extraction plants which process the oil, gas, and condensate from the fields producing from Jurassic formations. Sulfur is important in the production of many chemicals and the sulfur produced at the extraction plants adds considerably to the raw material wealth of the state. More than 4,125,000 long tons of sulfur had been produced through 1985.

A nearly continuous belt of lignite in formations of Paleocene age occurs between Sumter and Henry Counties. Demonstrated resources of lignite in beds from 30 to 60 inches thick occur at relatively shallow depths in the area. Preliminary studies indicate that considerable quantities of commercial grade peat occur near the coast in Baldwin and Mobile Counties and in the delta of the Mobile River.

Alabama's unique location at the intersection of five physiographic regions provides for both complex geology and an abundance of natural resources. Alabama's continued economic prosperity is dependent on the prudent development and management of these numerous natural resources. The Geological Survey of Alabama welcomes the opportunity to be a part of the team of State government agencies and private sector industries which play a role in the administration necessary to utilize and conserve our State's natural treasures so as to benefit Alabama's citizenry and future generations alike. We look forward to working in cooperation with our sister agencies, such as the Alabama Highway Department, in meeting the challenges associated with managing our vast natural resource base. We appreciate the opportunity to address the 40th Annual Highway Geology Symposium.

ACKNOWLEDGMENTS

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ENGINEERING GEOLOGY AND GEOTECHNICAL ENGINEERING FOR A PRELIMINARY ROUTE ALIGNMENT STUDY FOR 25 MILES OF ARIZONA STATE ROUTE 87

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Sergent, Hauskins & Beckwith Geotechnical Engineers, Inc.

ABSTRACT

Arizona State Route 87 (SR 87) is an important secondary highway linking metropolitan Phoenix with year-round recreational areas to the north. SR 87 in the study area is presently a two-lane highway with passing lanes on a few steep grades. In many areas, the existing roadway does not meet minimum standards for grades, horizontal curves and sight distances. Because of concerns about safety and traffic flow during peak usage periods, the Arizona Department of Transportation (ADOT) had a route alignment study performed for a proposed four-lane highway with a design speed of 65 miles per hour. Sergent, Hauskins & Beckwith Geotechnical Engineers, Inc. was the geotechnical engineering and engineering geology consultant for the study.

The study area lies almost entirely within the Transition Zone Physiographic Province of Arizona, an area of rugged mountains and steep valleys located between the Basin and Range and Colorado Plateau provinces. The alignment corridor traverses areas of unconsolidated to moderately consolidated sediments; Cenozoic volcanic rocks, primarily basalt flows and tuff deposits; and granitic rocks of older Precambrian age. Because of the rugged topography, deep cuts and fills will be required in many areas.

The performance of the existing roadway system was evaluated, and preliminary recommendations for slopes, earthwork factors and pavement design were developed for the proposed alternate alignments. The study also addressed the stability of embankment fills and deep cuts, foundations for bridges, and seismic risk relative to a suspected active fault crossing the existing roadway. In one area where the roadway elevation would have to be lowered by more than 100 feet, conventional deep-cut construction and twin tunnel bore alternatives were compared.

The rugged topography imposed constraints on the methods used in the field investigation. Many of the proposed alternative alignments were inaccessible to a truck-mounted drill rig. Refraction seismic traverses were utilized to

provide information on depth to bedrock and rippability of rock. Some of the seismic work, and much of the geologic mapping, had to be performed either on foot or on horseback.

INTRODUCTION

Arizona State Route 87 (SR 87) is an important secondary highway linking metropolitan Phoenix with year-round recreational areas to the north (see Figure 1). SR 87 in the study area is presently a two-lane highway with passing lanes on a few steep grades. In many areas, the existing roadway does not meet minimum standards for grades, horizontal curves and sight distances. Because of concerns about safety and traffic flow during peak usage periods, the Arizona Department of Transportation (ADOT) contracted for preliminary studies to evaluate the cost of upgrading SR 87 to a four-lane highway with a design speed of 65 miles per hour (mph).

In Spring of 1987, the firm of Sergent, Hauskins & Beckwith Geotechnical Engineers, Inc. (SHB), of Phoenix, Arizona was engaged to perform a preliminary geotechnical engineering and engineering geology investigation along approximately 25 miles of the existing SR 87 alignment. The initial contract included reconnaissance geologic mapping, a condition survey of the existing roadway system, and recommendations for improvements.

Before the initial investigation was completed, however, the scope of the project was changed. The Arizona Department of Transportation (ADOT) decided that a more detailed location study should be performed for the 25 miles of SR 87. The study was to include evaluation of the existing alignment as

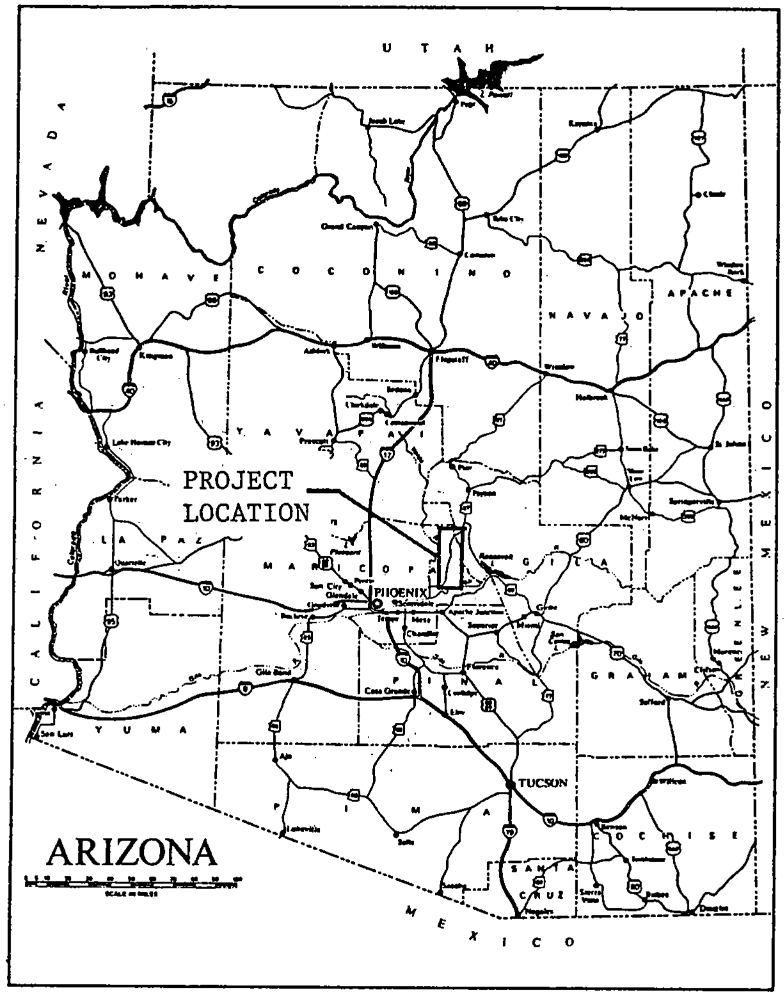

FIGURE 1 - LOCATION MAP

well as a number of alternate alignments. SHB's involvement in the study was broadened to include geotechnical engineering and engineering geology investigations in sufficient detail to provide preliminary recommendations for cost estimates to be used in comparison of the proposed alternate alignments.

PROJECT DESCRIPTION

The existing two-lane roadway, with passing lanes on some difficult grades, is to be upgraded to provide two lanes in each direction. This upgrade will meet minimum standards for vertical grades (6 percent) and horizontal curves (3 degrees 45 minutes). In all areas, the existing alignment was included as as one of the alternatives to be studied. In most areas, several new alternate alignments were also investigated.

The 25-mile roughly north-south corridor was divided into six major study sections, based on similarity of conditions and anticipated construction phasing. Approximately 44 different alternate alignment segments were studied to various levels during the course of the project. The roadway elevation varies from about 2081 feet at Rock Creek near the south end to about 4350 feet at the Slate Creek Summit near the north end. The alignment crosses several major drainages; depending on which combination of alternate alignments is selected, as many as fifteen new bridges may be required.

As the investigation progressed, some alternate alignments were eliminated from further consideration because of excessive cost, anticipated inconvenience to motorists during construction (road closures, lengthy detours, and long-term reduced speed limits in construction zones), and environmental or other social considerations. Others were redesigned to some extent on the basis of additional information acquired during the field investigations. Still other alternate alignments were added after the study was well under way. Figure 2 shows the system of alternate alignments under consideration at the time of the final (?) report submittal in May, 1989.

As can be seen in Figure 2, most of the southern part of the study corridor involves a narrow strip along the existing roadway. At the north end of the project, however, a major relocation of SR 87 is being considered. For several miles in this area, the existing alignment follows Sycamore Creek, a major intermittent stream and an important riparian habitat. Maintenance problems have occurred in this area due to flooding of the roadway and saturation of the subgrade soils. Further north, the existing alignment passes through extensive clay deposits on the lower slopes of Iron Dike Mountain. ADOT personnel report recurring problems with slope stability and pavement distress in this area. Construction of a four-lane highway roughly following the existing alignment would necessitate deep fills and major cuts in rock and clay, and would require construction of retaining walls, bridges and other structures. Two new alignment corridors are also being considered in this area. The Kitty Joe Canyon alignment is located to the east and the West Sycamore alignment to the west of the existing alignment.

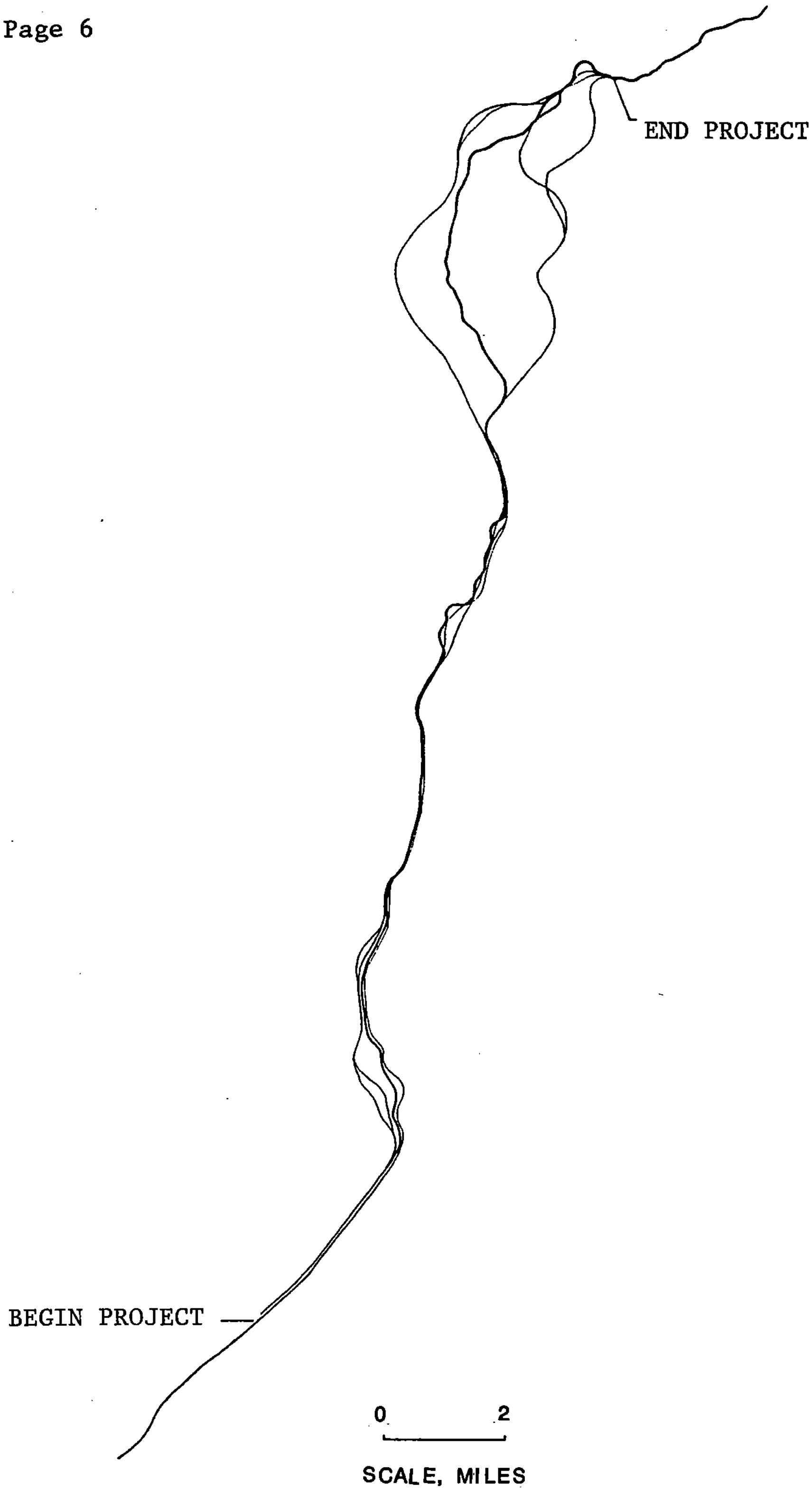

FIGURE 2 - ALTERNATE ALIGNMENTS

With one exception, all of the alternative alignments at the north end of the project converge near the Maricopa-Gila County Line. In order to maintain minimum geometric standards for the design speed of 65 mph, the roadway elevation at the Slate Creek Summit will have to be lowered by as much as 110 feet, depending on which alignment alternative is selected. Options studied in this area included conventional open cut construction, which would result in a deep, wide cut, and construction of twin 42-foot wide by 27-foot high tunnels.

INVESTIGATION

The object of our investigation was to evaluate the physical properties of the subsoils and rock along the proposed alternative alignments to provide preliminary evaluations and recommendations for the various alternatives for the new roadway system. Preliminary recommendations were provided for excavations, cut and fill slopes, embankment construction, design of foundations for structures, potential sources of fill material along the alignment and other earthwork elements of the project.

The project area is rugged, remote, sparsely populated and, with the exception of a few mercury mines near the north end, not known to include any mineral deposits of economic interest. As a result, detailed geologic studies of the area were not available in the literature. Adequate topographic mapping was not even available for some parts of the study area; additional surveying had to be performed along some proposed alignments in order to permit investigation to the same level of confidence for all alternates.

Some aerial photography of the project area was available, at varying scales and in some cases with stereographic coverage. As part of our investigation, low-sun-angle black-and-white aerial photography was performed in two areas: one near the south end of the project where a suspected active fault crosses the existing highway, and the other near the north end of the project where the roadway elevation may have to be lowered by as much as 110 feet.

The first task of the field investigation was to perform basic, reconnaissance-level geologic mapping along all the proposed alternate alignments. On the basis of this information, areas were identified which would require more detailed engineering geology and geotechnical engineering studies.

In the south part of the study area, the proposed alternate alignments generally follow the existing roadway, and it was possible to use our evaluation of the performance of the existing roadway system in developing recommendations for alternate alignments. In the north part of the project, however, many of the proposed alternate alignments are far from the existing roadway, and many also involve major cuts and fills in difficult geotechnical conditions. In this area we were able to rely far less on the performance of the existing roadway in developing recommendations for new alternate alignments.

Unfortunately, this north part of the study area is also very rugged, and outside of the existing SR 87 corridor there are only a few fairly primitive Forest Service roads. As a result, it was not possible to mobilize a drill rig to many areas where borings would otherwise have been performed. A total of five borings were drilled for this study, located where areas of interest were near existing roads. In other areas where additional subsurface information was required, seismic refraction traverses were performed, in many cases involving access by four-wheel-drive vehicle, on foot or on horseback.

GEOLOGY AND GEOTECHNICAL PROFILE

Regional Setting

The project lies almost entirely within the Transition Zone or Central Mountain Region of Arizona, between the Basin and Range Physiographic Province to the south and west, and the Colorado Plateau Physiographic Province to the north and east (Peirce, 1984 and 1985)*. The southernmost 3 miles of the project lies within the Basin and Range Province. (See Figure 3).

The Transition Zone (TZ) of Arizona is characterized by high topographic relief, with steep mountain ranges dissected by deep valleys. The rocks of the TZ consist predominantly of Precambrian metamorphic, deformed sedimentary, and intrusive and extrusive igneous rocks. Erosional remnants of Paleozoic sedimentary rocks are also present. Mesozoic rocks are generally absent in this province. Cenozoic rocks include

*References are listed at end of text.

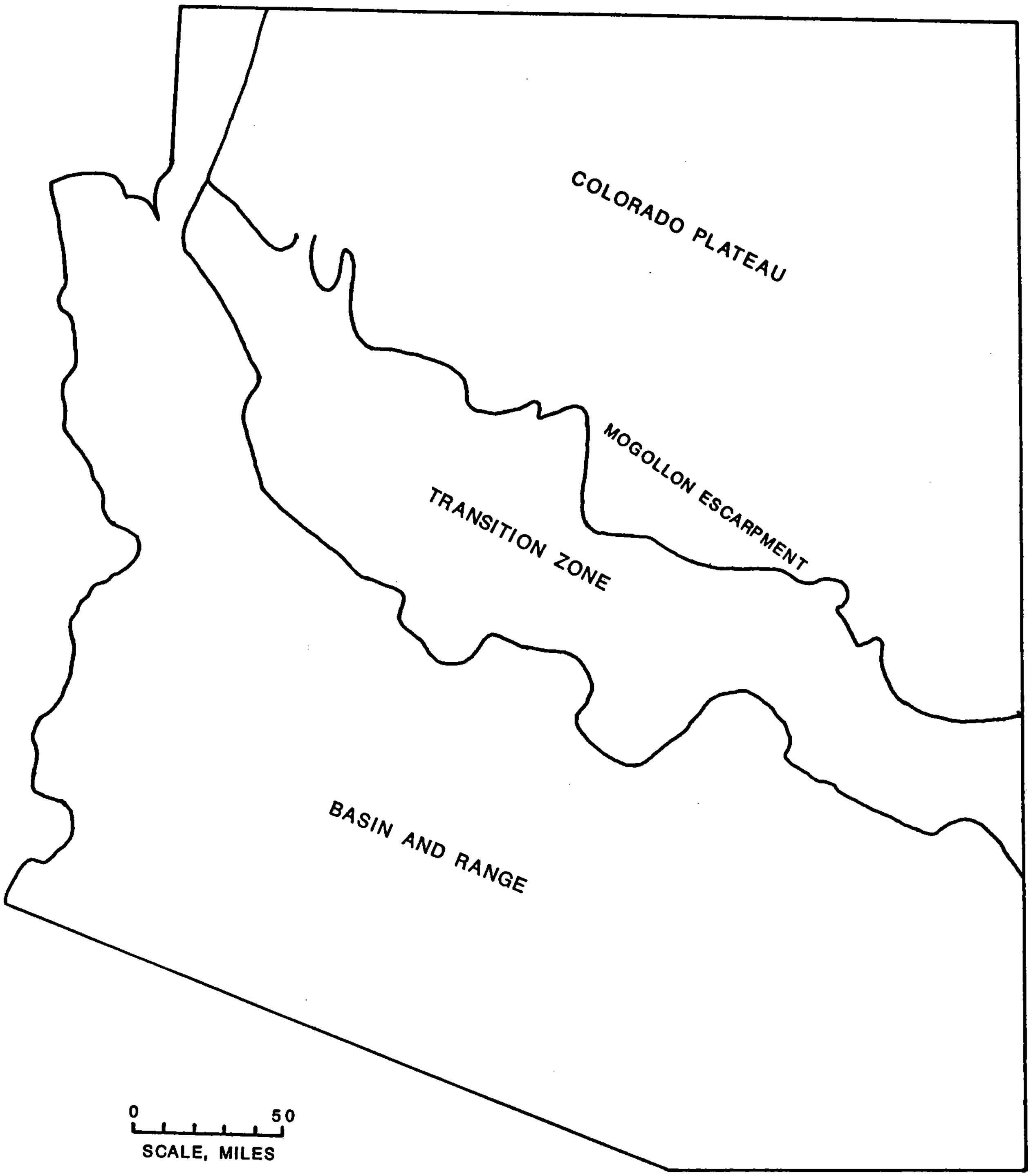

FIGURE 3 - PHYSIOGRAPHIC PROVINCES OF ARIZONA

basalt flows and dikes and related volcanics, and sedimentary deposits of Tertiary to Quaternary age.

Geology & Geohydrology of Alignment

The study area along the SR 87 corridor extends for a total of approximately 25 miles, beginning at an elevation of about 2200 in unconsolidated alluvial fan and pediment sediments at the south end. The corridor rises to an elevation of about 4350 in Cenozoic volcanic rocks of the Mazatzal Mountains at the Slate Creek Summit near the north end of the project. Bedrock in the project area includes granitic rocks of older Precambrian age, crosscut by numerous dikes and basalt flows of Quaternary and Tertiary age, locally including some gravels and considerable amounts of other volcanic rocks. Sediments of Quaternary age, mostly unconsolidated, predominate at the lower elevations near the south end of the project. These types of sediments also occur further north in present-day stream channels and intermontane basins, and on terraces and piedmont slopes. Alluvial deposits of Tertiary to older Quaternary age, unconsolidated to moderately consolidated, occur in scattered areas primarily as basin fills or on piedmont slopes (Wilson and others, 1957; Royse and others, 1971; Arizona Department of Transportation, 1977; ASU Department of Geology, 1978).

In the northern part of the project, where bedrock (basalt and associated volcanics, granite, and consolidated sediments) predominates at or near the surface, relief is great and slopes are steep. Stream gradients are correspondingly steep. For example, Sycamore Creek in the vicinity of Sunflower has a gradient of about 120 feet per mile (Thomsen and Schuman, 1968). Because the rocks are relatively impermeable, appreciable groundwater in storage only occurs in thin deposits of unconsolidated alluvium associated with drainages.

In the southern alluvial area, there is generally low relief and slopes are gentle. Numerous drainages dissect the landscape. Sycamore Creek (west of the existing SR 87 alignment) has a gradient of only about 30 feet per mile (Thomsen and Schuman, 1968). The alluvium has a relatively high permeability and, because of its thickness, can store large volumes of water received primarily as recharge from Sycamore Creek and its tributaries. The depth to the groundwater table in this area varies from a few feet to 50 feet or more, depending on the distance from contributing streams, thickness of alluvial cover over bedrock and precipitation patterns.

Geologic Structure

Numerous high-angle normal and reverse faults are visible in road cuts along the existing alignment. These structures appear as lineaments on aerial photographs and have been reported by previous investigators (Royse and others, 1971; ASU Department of Geology, 1978; Fugro, 1981; Menges and Pearthree, 1983; Scarborough and others, 1983).

The Sugarloaf Peak Fault Zone is a north to northwest trending arcuate normal fault, about 6 miles long, dipping steeply to the northeast (Fugro, 1981). This fault crosses the existing alignment at about MP 206.9. Estimates of the date of latest movement on this fault range from Pleistocene (Fugro, 1981) to late to mid-Holocene (5,000 years ago) (Menges and Pearthree, 1983; Anderson and others, 1986). No movement on this system has been reported in historic time and investigations during the field mapping program completed as part of this study revealed no evidence of recent movement. The low-sun-angle photography work done in the study indicates the fault is probably not active in the engineering sense.

Jointing in the granitic and dike rocks is highly variable. Wilson (1939) reports regional trends for the granites of the Mazatzal Mountains of northeast-striking and northwest-striking joints, both steeply dipping. These regional trends were noted during field reconnaissance. Other orientations, however, were also noted and rock slope stability studies will be required for major road cuts. Strong regional joint orientation trends were not apparent from an evaluation of the aerial photographs.

COMPARISON OF TUNNEL & OPEN-CUT CONSTRUCTION IN THE COUNTY LINE AREA

In order to conform to geometric design criteria for the proposed design speed of 65 mph, the future roadway near the County Line will have to be a maximum of approximately 110 feet lower than the existing roadway.

Two approaches were evaluated for construction of the future roadway at the 65 mph design grade:

- Conventional open cut construction. It is estimated that this strategy would result in slope heights in excess of 500 feet, and a maximum cut width, from slope crest to slope crest, of nearly 1,800 feet.
- Construction of twin, 42-foot wide by 27-foot high tunnels.

These alternative approaches are discussed in more detail in the following subsections.

A third possibility, that of not meeting the design standards in this area and maintaining the roadway at or near the present roadway elevation, has also been discussed as an interim measure to improve safety.

Tunnel Option

For assistance in preliminary design studies and cost estimating, a subconsultant on tunneling was engaged for this project. As a result of these studies, twin 42-foot wide by 27-foot high tunnel bores, one approximately 1,240 feet long and one approximately 1,310 feet long, were recommended.

Based on preliminary studies, the total cost of constructing two 42-foot wide by 27-foot high tunnels, one 1,300 feet long and the other 1,600 feet long (the tunnel lengths originally proposed), was estimated to be $13,300,000.00 or approximately $4,600.00 per lineal foot. This amount is based on an estimated total project cost of $11,100,000.00, plus a 20 percent allowance for contingencies. This estimate includes installation of the highway lighting system, but does not include the cost of constructing power transmission lines to the tunnel area. For the revised tunnel lengths now proposed, the total cost, based on the unit cost of $4,600.00 per lineal foot as developed above would be approximately $11,730,000.00.

The cost estimates developed were for nonventilated tunnels. Based on preliminary studies, it was our opinion that nonventilated tunnels would be technically feasible for the conditions anticipated for this project. There are several similar tunnels which are currently performing properly without mechanically assisted ventilation systems.

If a ventilation system should be required, it is estimated that construction costs would increase by approximately 50 percent. These additional capital costs include the ventilation system and a larger tunnel cross section to provide airflow plenum space and maintain minimum overhead clearance above the traffic lanes. A maintenance building for personnel and supplies would have to be constructed outside the tunnel. Operating and maintenance costs during the life of the tunnel would also be increased.

Open Cut Option

For preliminary planning and cost estimates, we recommended that composite slopes of 2:1 be used because of the extensive near-surface clay deposits in the area. Roughly horizontal benches 20 feet wide would be required at 60-foot vertical intervals. With a composite slope of 2:1, inter-bench slopes would be approximately 1 2/3:1. A typical cross-section of the open cut in this area is shown in Figure 4.

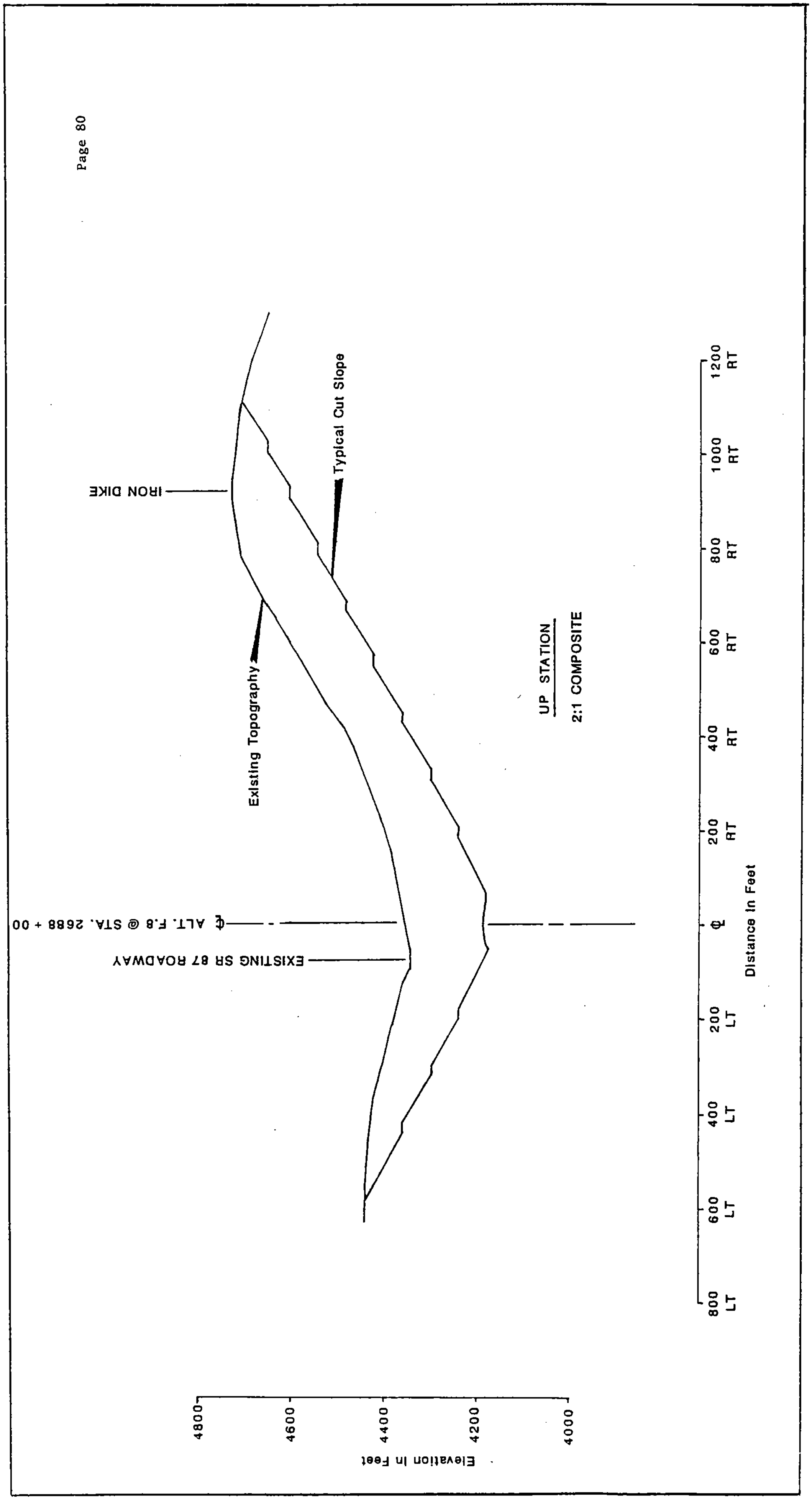

FIGURE 4 - TYPICAL CROSS-SECTION OF OPEN CUT AT COUNTY LINE

Actual cut slopes in final design would vary in the different materials encountered, which include unconsolidated soil deposits, sedimentary rock units, tuffs, and predominantly basaltic to andesitic volcanic rocks. Localized slides and isolated rock falls may occur during construction, requiring further adjustment of slopes. Considering the complex geology and history of slides in the area, in our judgment, cost estimates based on the 2:1 composite slope will include a reasonable contingency for slide repair.

Excavation methods for this cut would also vary widely in different materials. In unconsolidated sediments, excavation will be possible in most cases using scrapers with some light dozer ripping. Much of the sedimentary rocks and tuffs will probably be rippable, based on seismic velocities and laboratory point load test results. However, much of the excavation in the tuffs and sedimentary rock units, and probably most of the excavation in the basaltic to andesitic volcanic rocks, will require blasting. Based on the limited amount of subsurface information available from studies to date, it is not possible to determine more specifically the limits of the areas where different excavation methods will be applicable. If this option is selected for use in final design, detailed geotechnical and rock mechanics studies will be required.

CONCLUSION

This project was interesting and challenging on several levels. First, the large project area and large number of alternate alignments to be investigated, in conjunction with the lack of previous detailed studies, made it necessary to squeeze as much information as possible from the preliminary field investigation program. Second, the variety of rock and soil materials encountered, ranging in age from Precambrian to very recent, made generalization difficult, requiring instead the division of the project area into a large number of smaller segments for study purposes. Finally, the rugged terrain and the lack of roads made it necessary to perform much of the field investigation with four-wheel-drive vehicles, on foot or on horseback.

REFERENCES

Anderson, L.W., Piety, L.A. and Hansen, R.A., 1986, "Seismotectonic Investigation, Stewart Mountain Dam, Salt River Project, Arizona," U.S. Bureau of Reclamation, Denver, Colorado.

Arizona Department of Transportation, Materials Services, 1977,"A Materials Inventory of Maricopa County."

ASU Department of Geology, 1978, "Road Log from Tempe to Payson and Diamond Point Via Highways 87 and 260;" in Burt, D.M. and Pewe, T.L., Guidebook to the Geology of Central Arizona, Arizona Bureau of Geology and Mineral Technology, Special Paper No. 2, pp. 143-150.

Fugro, Inc., 1981, "Seismotectonic Study, Roosevelt Dam, Arizona," Report Prepared for U.S. Department of Interior, Bureau of Reclamation, Denver, Colorado.

Menges, C.M. and Pearthree, P.A., 1983, "Map of Neotectonic (Latest Pliocene-Quaternary) Deformation in Arizona," Open-File Report 83-22, U.S. Geological Survey and Arizona Bureau of Geology and Mineral Technology.

Peirce, H.W., 1984, "The Mogollon Escarpment," Fieldnotes, Arizona Bureau of Geology and Mineral Technology, Vol. 14, No. 2, pp. 8-11.

Peirce, H.W., 1985, "Arizona's Backbone: The Transition Zone," Fieldnotes, Arizona Bureau of Geology and Mineral Technology, Vol. 15, No. 3, pp. 1-6.

Royse, C.F., Sheridan, M.F. and Peirce, H.W., 1971, "Geologic Guidebook 4 - Highways of Arizona: Arizona Highways 87, 88 and 188," Arizona Bureau of Mines, Bulletin 184.

Scarborough, R.B., Menges, C.M. and Pearthree, P.A., 1983, "Map of Basin and Range (Post-15 m.y.) Exposed Faults, Grabens and Basalt-Dominated Volcanism in Arizona," Arizona Bureau of Geology and Mineral Technology, Open-File Report 83-21.

Thomsen, B.W. and Schumann, H.H., 1968, "Water Resources of the Sycamore Creek Watershed, Maricopa County, Arizona," U.S. Geological Survey Water-Supply Paper 1861.

REFERENCES (CONT.)

Wilson, E.D., 1939, "Pre-Cambrian Mazatzal Revolution in Central Arizona," Bulletin Geological Society of America, Vol. 50, pp. 1113-1164.

Wilson, E.D., Moore, R.T. and Peirce, H.W., 1957, "Geologic Map of Maricopa County, Arizona," Arizona Bureau of Geology and Mineral Technology, Scale 1:375,000.

SUITABILITY OF BOTTOM ASH FOR INDIANA HIGHWAY CONSTRUCTION

by

Wei-Hsing Huang* and C. William Lovell*

INTRODUCTION

The utilization of power plant ash as a construction material has received increasing attention because it not only solves a solid waste problem but also provides an alternative construction material. As supplies of natural construction materials diminish rapidly, the need for such alternative materials in construction becomes more evident. Also, power plant ash is considered by many as a waste product; as such it costs considerably less than conventional construction materials for which it may be adequately substituted.

There are essentially two types of ash produced from the combustion of coal in a furnace: bottom ash and fly ash. Bottom ash is the slag which builds up on the heat-absorbing surfaces of the furnace, and which subsequently falls through the furnace bottom to the ash hopper below. Fly ash is entrained in the flue gases that leave the furnace and is collected by the ash precipitator.

While there is considerable information accumulated on the properties of fly ash and its applications, very little has been developed in the way of productive use of bottom ash, primarily because of the lack of information on properties of this material. The purpose of this paper is to provide the physical and chemical characteristics and engineering properties of bottom ash, based on laboratory investigations conducted on Indiana bottom ashes. In addition, the potential environmental effects of bottom ash utilization are also evaluated.

PRODUCTION AND DISPOSAL OF BOTTOM ASH

Ash characteristics are affected as much by the type of coal burned as they are by the type of boiler. Today's large utility boilers burn pulverized coal and

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have dry-bottom furnaces, which produce dry bottom ash. In a dry bottom furnace, bottom ash represents about 20% by weight of the ash originally in the coal. The remaining 80% is collected as fly ash. Most newer power plants are of the dry bottom type and, as a result, dry bottom ash production has been increasing significantly in recent years. The ash is composed of irregular-shaped particles with a porous surface texture.

Wet bottom ash, often referred to as boiler slag, is generated from a slag-tap furnace. The word wet refers to the molten state of the ash which leaves the furnace as a liquid. The molten ash is then quenched in the water-filled hopper to form boiler slag. In this case, as much as 50 to 80% of the ash produced is collected as bottom ash. Wet bottom boiler slag is shiny black in color, angular, and smooth textured.

Power plant ash may be disposed of by two methods. Dry disposal implies transport and deposition of dry or moistened ash. This may involve temporary storage of ash in silos, subsequent hauling by trucks, and compacting at a landfill. Most power stations in urban area are handling their ash by the dry method, due to land limitation. An alternative method of disposal is to sluice ash from the plant to a pond or pit in which the ash settles. This is termed wet disposal, and is more commonly used, because of economy. Ash ponds also minimize dust problems and are simple to operate. In the case of wet disposal, the pipeline outlet may be gradually moved along the lagoon to ensure approximately uniform distribution of ash. Generally, crushing of bottom ash from the hopper is required for both dry and wet disposal methods to facilitate the transport.

EXPERIMENTAL PROGRAM

Selection of Ash Sources

The state of Indiana has more than 30 coal-fired power plants actively producing ash. However, it was impractical to sample bottom ash generated at every power station in the state. So the criteria for selection were established and these included: geographic distribution, type of coal, kilowatt capacity, ash disposal method, and coverage of all franchised power utilities in the state. Finally, 11 bottom ashes from 10 power stations were used in the study. A map showing the approximate location of each of the 11 selected sources is shown in Figure 1.

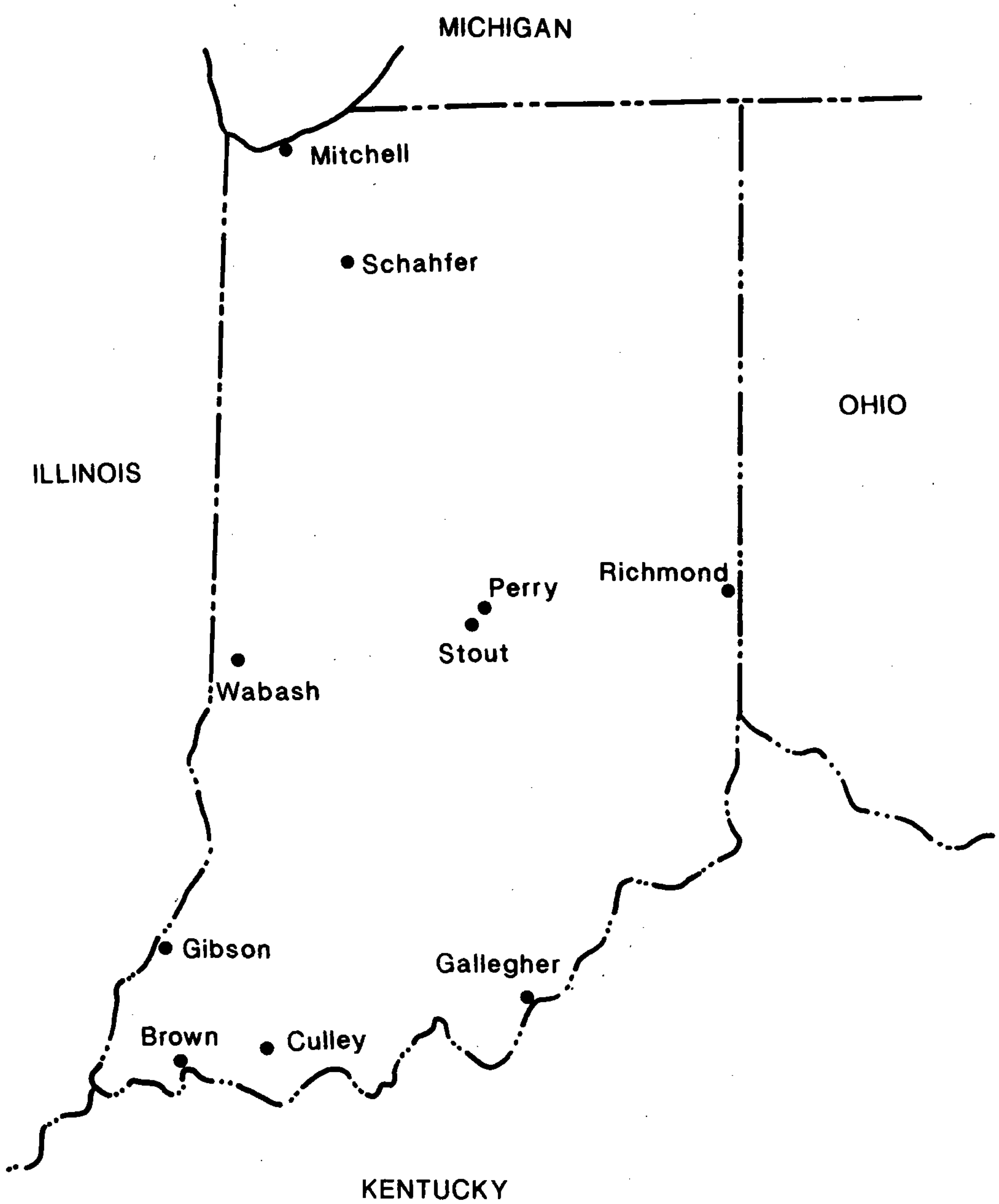

FIGURE 1. APPROXIMATE LOCATIONS OF BOTTOM ASH SOURCES IN INDIANA

In order to study the variability of ash properties, each source was sampled at least twice. All 11 ashes were subjected to a series of chemical and physical characterization tests, and then 3 ashes were chosen for detailed testing on the engineering properties and the potential environmental effects of the materials.

Chemical Analyses

The chemical compositions of Indiana bottom ashes are shown in Table 1. The principal constituents are silica (SiO_2), alumina (Al_2O_3), and iron oxide (Fe_2O_3). There are smaller quantities of calcium oxide (CaO), magnesium oxide (MgO), potassium oxide (K_2O), sodium oxide (Na_2O), and sulfur trioxide (SO_3), as well as minute traces of other elements. As can be seen from Table 1, the chemical composition of each bottom ash shows a reasonable degree of uniformity, except those ashes from Perry, Stout, and Richmond. These stations were burning different sources of coal just prior to the dates of sampling, and this is reflected by greater variations in the chemical composition of the bottom ash. The loss on ignition gives an approximate indication of the unburnt carbon content.

Gradation

Grain size analyses were performed, and the range of gradation for the 11 bottom ashes is shown in Figure 2. The range of gradation for fly ash from the same sources, which was determined in a previous study (Diamond, 1985), is also shown in Figure 2. The percentage of fines passing the No. 200 sieve for bottom ash ranges from 0 to 12%, and particles coarser than 1.5 in. are rarely found.

Among the 11 bottom ashes studied, 10 ashes are classified by the Unified Soil Classification system as sand. The other one is classified as gravel. Ten out of 11 samples have their coefficient of uniformity ranging from 7 to 33, while the most uniform ash has a uniformity coefficient of 3.7. By and large, bottom ash is a relatively well-graded, sand-sized material.

The gradation curves for bottom ashes sampled at different times provide an indication of the potential variability in the gradation. Figure 3 gives typical variations in the gradation of bottom ash. Generally, the ashes with uniform gradations tend to have less variation in the gradation.

TABLE 1. CHEMICAL COMPOSITION OF INDIANA BOTTOM ASHES

Ash source	date sampled	Percent by weight SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	K_2O	Na_2O	SO_3	loss on ignition
Schahfer unit 14	6-19-87	60.1	5.2	10.4	16.6	5.7	0.9	0.4	0.9	0.3
	5-12-88	53.4	6.0	13.5	18.5	5.7	1.2	0.3	1.0	0.1
Schahfer unit 17	6-19-87	58.1	15.2	12.7	7.0	0.8	1.9	0.3	2.2	0.1
	5-12-88	52.1	23.2	13.2	4.8	0.9	1.4	0.2	1.5	0.8
Gibson	5-18-87	58.7	14.6	14.1	3.1	0.8	2.0	0.4	1.3	0.4
	5-17-88	53.6	20.8	14.8	2.6	1.0	1.9	0.5	1.1	1.0
Gallegher	5-26-87	41.2	28.4	11.2	12.6	0.7	1.6	0.3	1.0	0.9
	5-14-88	49.3	24.2	16.4	3.9	0.9	1.7	0.2	2.6	1.4
Perry*	5-19-87	48.9	22.2	13.0	0.8	0.7	2.2	0.3	0.6	7.2
	7-19-88	52.5	6.0	24.3	0.9	0.1	2.3	0.4	0.6	6.2
Mitchell	6-19-87	58.8	6.8	7.8	7.9	2.2	1.4	0.1	3.3	8.1
	5-12-88	51.3	6.5	14.2	8.5	3.0	0.9	0.3	1.0	8.0
Wabash	6-23-87	55.7	21.5	14.3	1.7	0.7	1.9	0.3	0.8	0.2
	4-26-88	51.7	23.0	16.0	1.7	0.9	1.9	0.3	0.6	1.0
Richmond*	8-17-87	48.3	33.3	11.9	1.3	0.4	0.9	0.2	1.7	2.2
	5- 5-88	41.6	20.9	18.6	1.3	0.6	1.1	0.1	1.9	14.1
Stout*	5-27-87	24.2	42.0	6.9	2.2	0.4	0.6	0.2	0.8	18.4
	6-20-88	54.9	20.2	16.7	1.6	0.9	1.9	0.8	1.8	0.3
Culley	8-21-87	35.6	30.1	11.7	14.6	0.8	1.4	0.3	1.0	0.0
	5-14-88	32.0	31.1	11.8	13.9	1.1	0.5	0.2	0.9	0.3
Brown	8-21-87	48.1	27.6	13.4	3.1	0.8	2.1	0.3	1.7	1.9
	5-17-88	38.5	38.0	12.6	3.8	0.7	1.3	0.2	3.3	1.1

*** Plants were burning different sources of coal prior to the dates of sampling.**

Specific Gravity

The specific gravity of the 11 bottom ashes, as tabulated in Table 2, ranges from 1.94 to 3.46. This is a much wider range than for most soils (range from 2.5 to 2.8, Hough, 1969). The specific gravity of the ash is, to some degree, a function of the chemical composition. Obviously, high carbon content will result in a low specific gravity, whereas high iron content will produce high specific gravity.

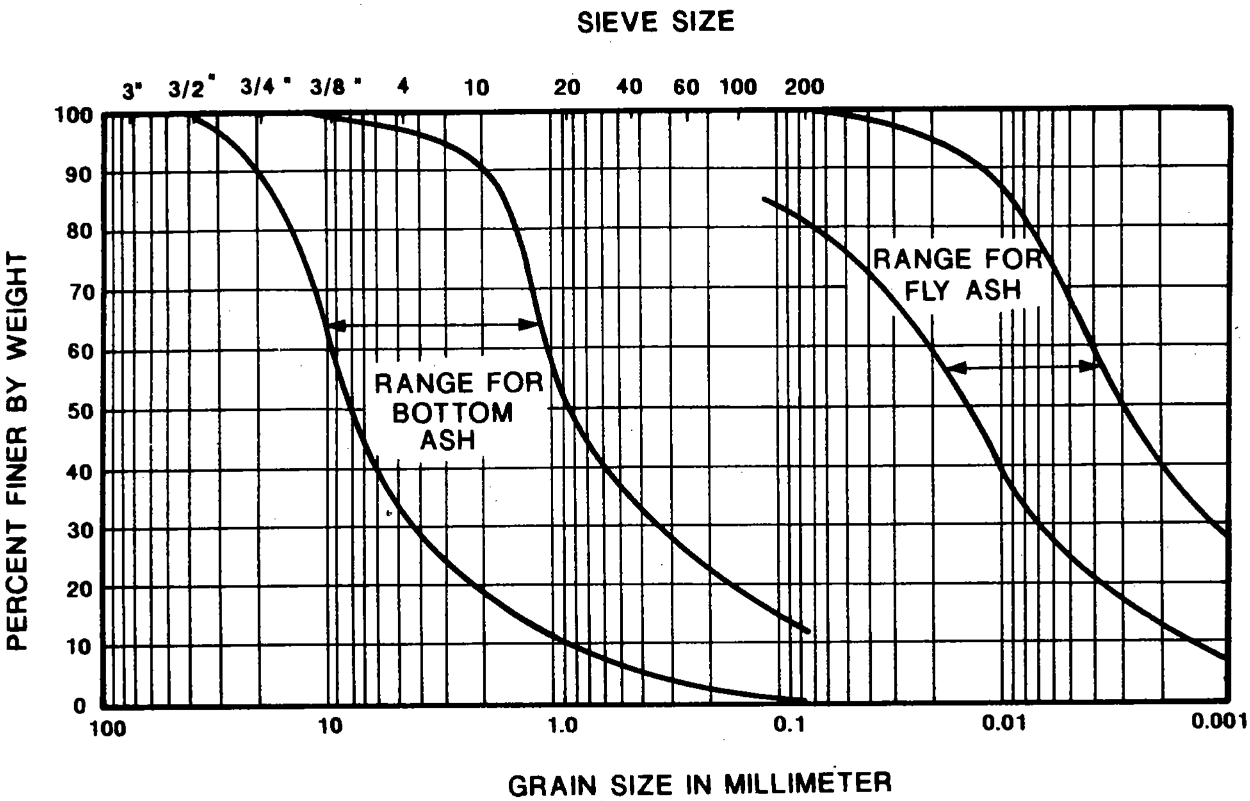

FIGURE 2. RANGES OF GRADATION FOR BOTTOM ASH AND FLY ASH

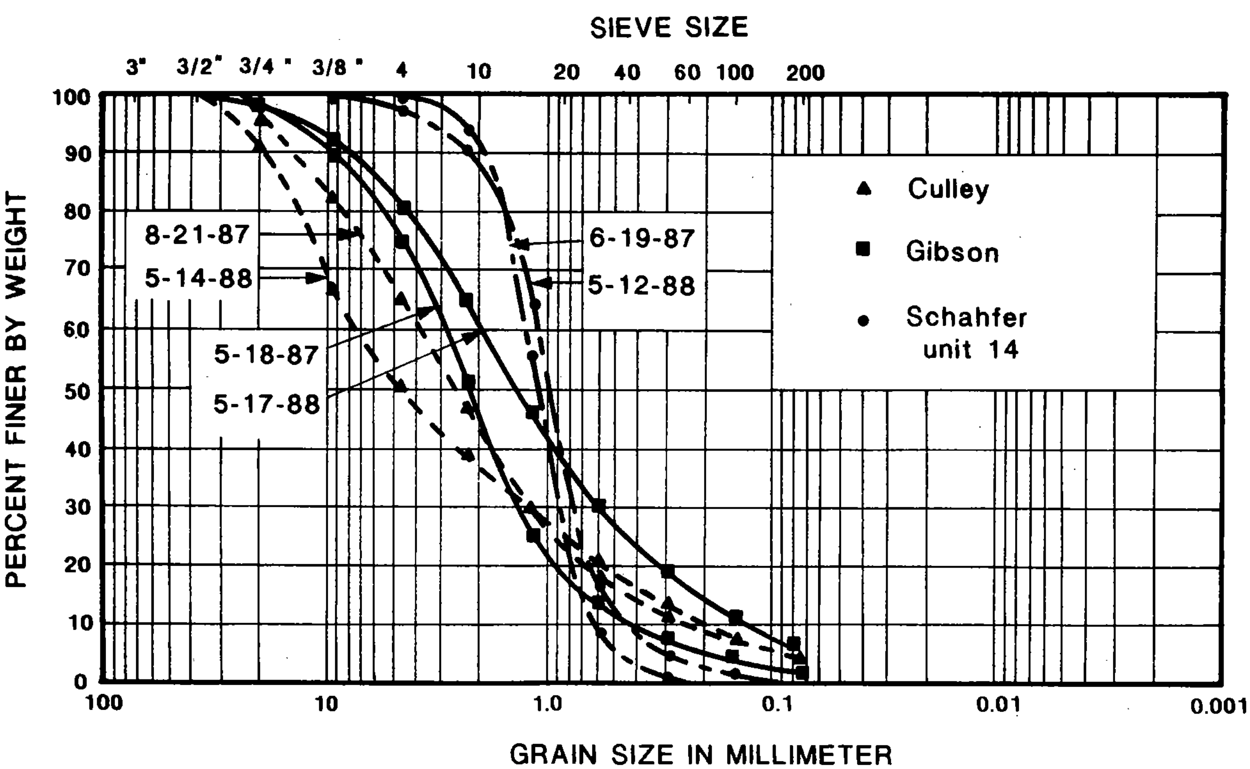

FIGURE 3. TYPICAL VARIATIONS IN GRADATION OF BOTTOM ASH

TABLE 2. SPECIFIC GRAVITY OF BOTTOM ASHES

Ash Source	boiler type	1st sample	2nd sample
Schahfer			
unit 14	wet bottom	2.82	2.81
unit 17	dry bottom	2.57	2.61
Gibson	wet bottom	2.67	2.56
Gallegher	dry bottom	3.08	2.64
Perry*	dry bottom	2.12	1.94
Mitchell	dry bottom	2.44	2.47
Wabash	dry bottom	2.56	2.48
Richmond*	dry bottom	2.90	2.40
Stout*	wet bottom	3.46	2.45
Culley	dry bottom	3.21	3.23
Brown	dry bottom	2.71	2.97

* Plant burned different sources of coal.

Soundness and Los Angeles Abrasion

The sulfate soundness (ASTM C88-83) and Los Angeles abrasion (ASTM C131-81) tests results are summarized in Table 3. The sodium sulfate soundness loss after 5 cycles of soaking and drying is the one reported. It is found that almost all bottom ashes, with the exception of Perry ash, would meet the soundness and abrasion requirements of the Indiana Department of Highways (1988) for "class C" subbase material.

TABLE 3. SOUNDNESS AND LOS ANGELES ABRASION OF BOTTOM ASHES

Ash source	Sulfate soundness	Los Angeles abrasion (%)
Schahfer		
unit 14	1.3	NA*
unit 17	2.8	37.8
Gibson	2.7	34.4
Gallegher	1.9	36.5
Perry	10.8	52.2
Mitchell	3.2	35.8
Wabash	3.0	41.4
Richmond	2.1	NA*
Stout	5.0	43.0
Culley	2.2	39.8
Brown	5.8	44.8
Indiana Specifications		
Class C subbase	16.0	45.0
Class A subbase	12.0	40.0

* NA = not applicable

Permeability

The coefficient of permeability of bottom ash was measured by falling head permeability tests. The permeameter had a 4-inch diameter and ash samples were about 8 in. high. Table 4 gives the results of permeability tests conducted on bottom ashes compacted to 95% of the Proctor maximum dry density before testing. It can be seen that the permeability of bottom ashes are comparable to those of soils with similar gradings.

TABLE 4. COEFFICIENTS OF PERMEABILITY OF BOTTOM ASHES

Materials	Particle size classification	Permeability (cm/sec)	Permeability classification[1]
Schahfer			
unit 14	uniform coarse sand	0.101	high
unit 17	well-graded sand with gravel	0.034	medium
Gibson	well-graded sand	0.005	medium
Uniform coarse sand[2]		0.4	high
Well-graded sand & gravel[2]		0.01	medium

.[1] Terzaghi and Peck, 1967
.[2] Hough, 1969

Compaction

Results of laboratory standard compaction tests for selected bottom ashes are plotted in Figure 4. The shape of the compaction curves is typical of that for cohesionless materials (Foster 1962). These curves are characterized by a fairly high density for the air-dried condition, low densities at low water contents, and high densities at the high water contents.

Because the compacted densities at the air-dried conditions are comparable to those at optimum moisture contents, it may be beneficial to compact ash air-dried. Thus, much effort and cost in the control of moisture content during compaction can be saved. However, some bottom ashes are reported to lose stability when they dry out (Moulton et al., 1973). To date, little data on the relative effects of gradation and water content on the compacted unit weight of bottom ash are available. Therefore, more research on the compaction characteristics of bottom ash, especially field compactions, is highly desired.

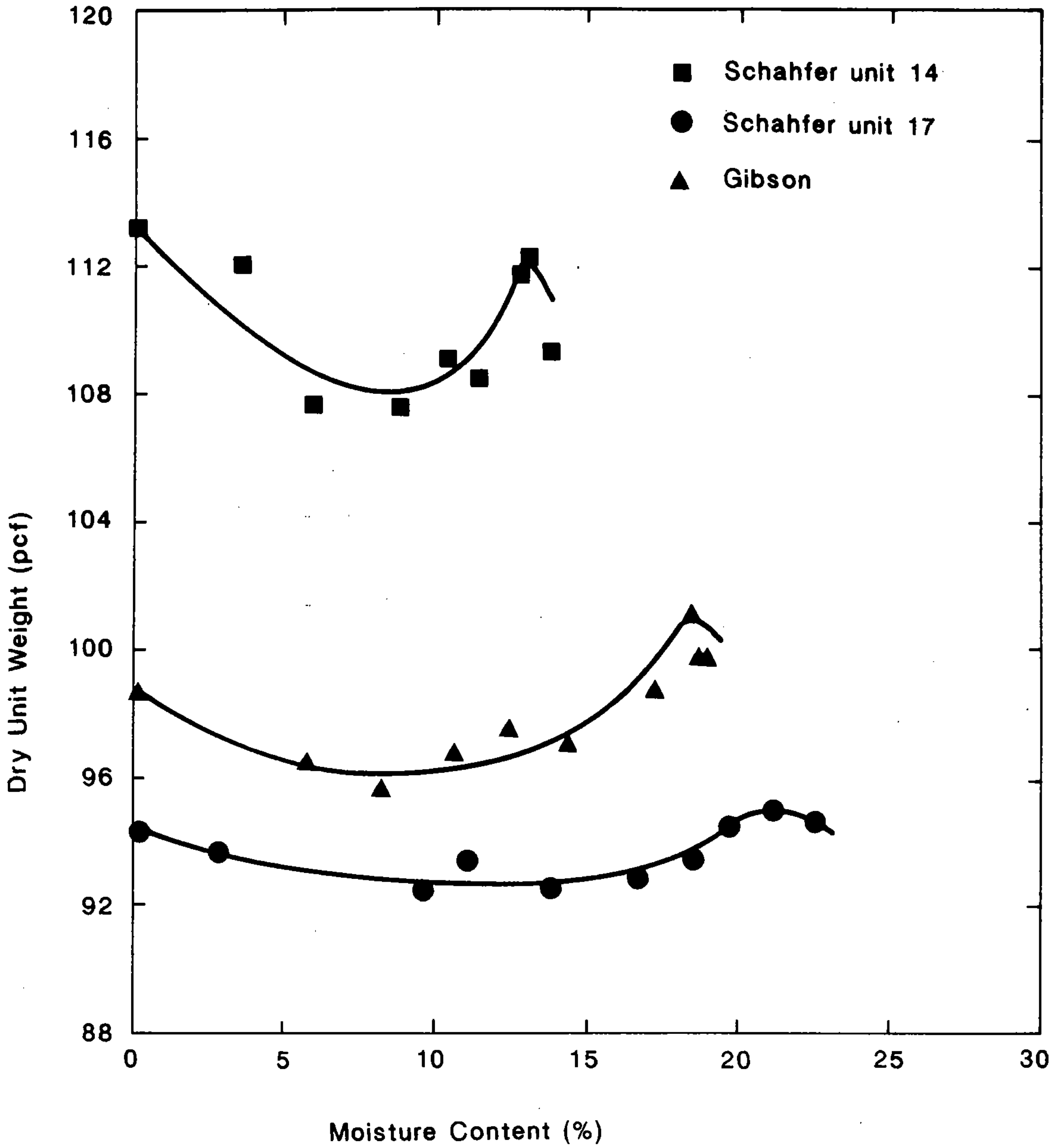

FIGURE 4. COMPACTON CURVES FOR BOTTOM ASH

Angle of Shearing Resistance

A number of direct shear tests were conducted on bottom ash in the laboratory. The direct shear box has a diameter of 2.5 in. and only materials finer than 3/8-in. were used. The tests were conducted at various relative densities and the normal stress varied from 5 psi to 34 psi.

The angles of friction obtained for bottom ash are plotted in Figure 5. Also shown in Figure 5 is the range of friction angle generally obtained for various sandy soils (Zeevaert, 1982). It is found that, at a given initial relative density, the friction angle of bottom ash is higher than that obtained for natural sandy soils. This can be attributed to the rough surface texture and angularity of the bottom ash particles, such that a higher degree of interlocking was developed in the shearing process.

One-Dimensional Compression

One-dimensional compression tests were performed using a consolidometer. Samples were loaded incrementally in a electronically controlled hydraulic loading system. The consolidation ring had a diameter of 4 in. and the bottom ash samples were 1.5 in. high. Initially, the samples were subjected to a seating pressure of 0.7 psi (AASHTO T216-81), and then soaked for at least 24 hours to measure swelling/collapse of the sample. Finally, the samples were maintained under maximum stress until no further deformation was observed. Only one ash experienced a small swelling of 0.07% and the swelling pressure was later found to be only 7 psi.

The stress-strain relationships for bottom ash obtained from one-dimensional tests are presented graphically in Figure 6. It is found that the compressibility of bottom ash is comparable to that of sand placed at same void ratio (Taylor, 1948).

If bottom ash is used as a fill material the compression of the bottom ash layer is usually estimated by elastic theory. Since bottom ash is a granular material, consolidation and secondary settlements are not significant (Holtz and Kovacs, 1981). When vertical loads of infinite length are applied to the bottom ash layer, the compression behavior becomes one-dimensional and the parameter used in estimating settlement is the secant constrained modulus (Das et al., 1983).

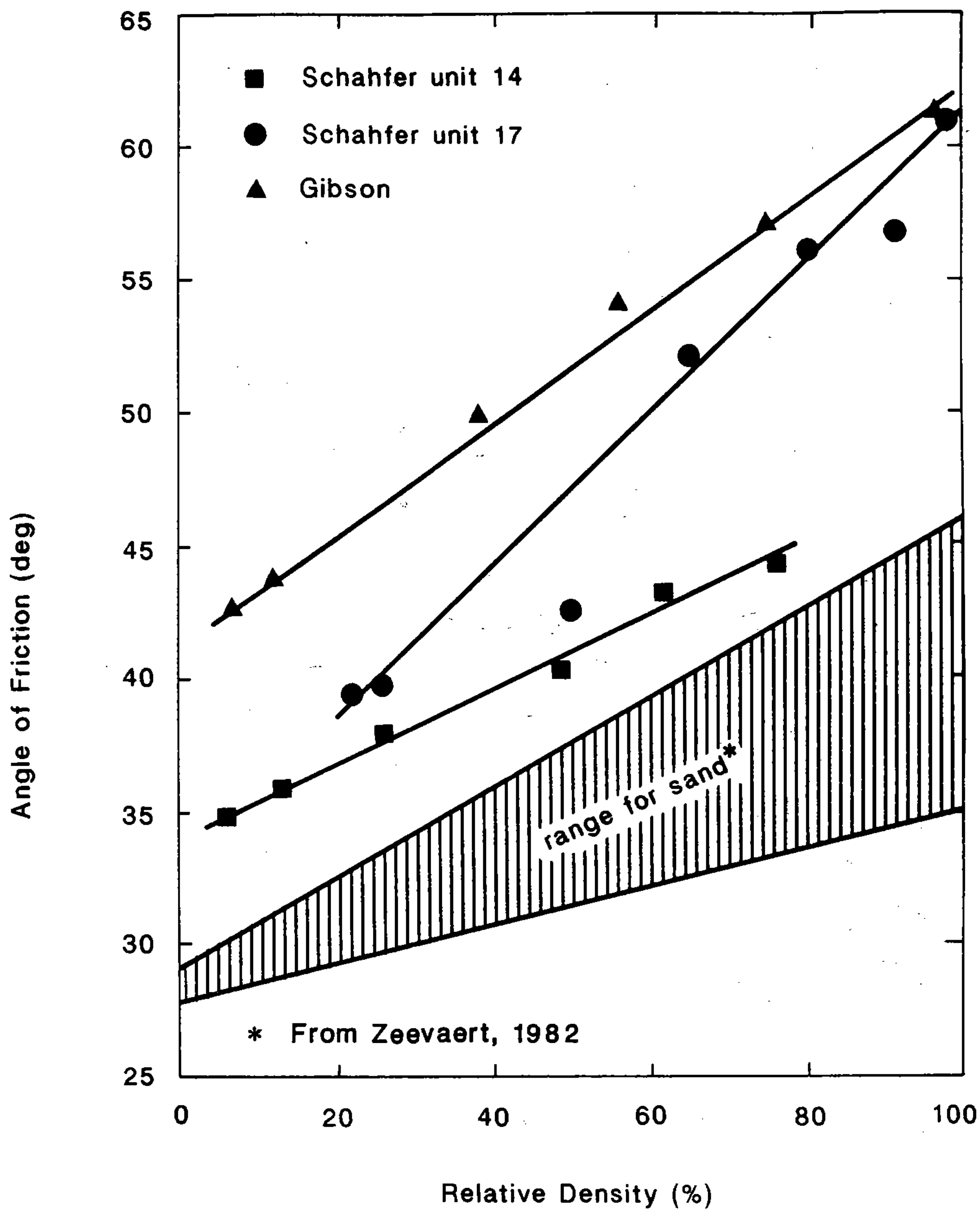

FIGURE 5. ANGLE OF FRICTION OF BOTTOM ASH

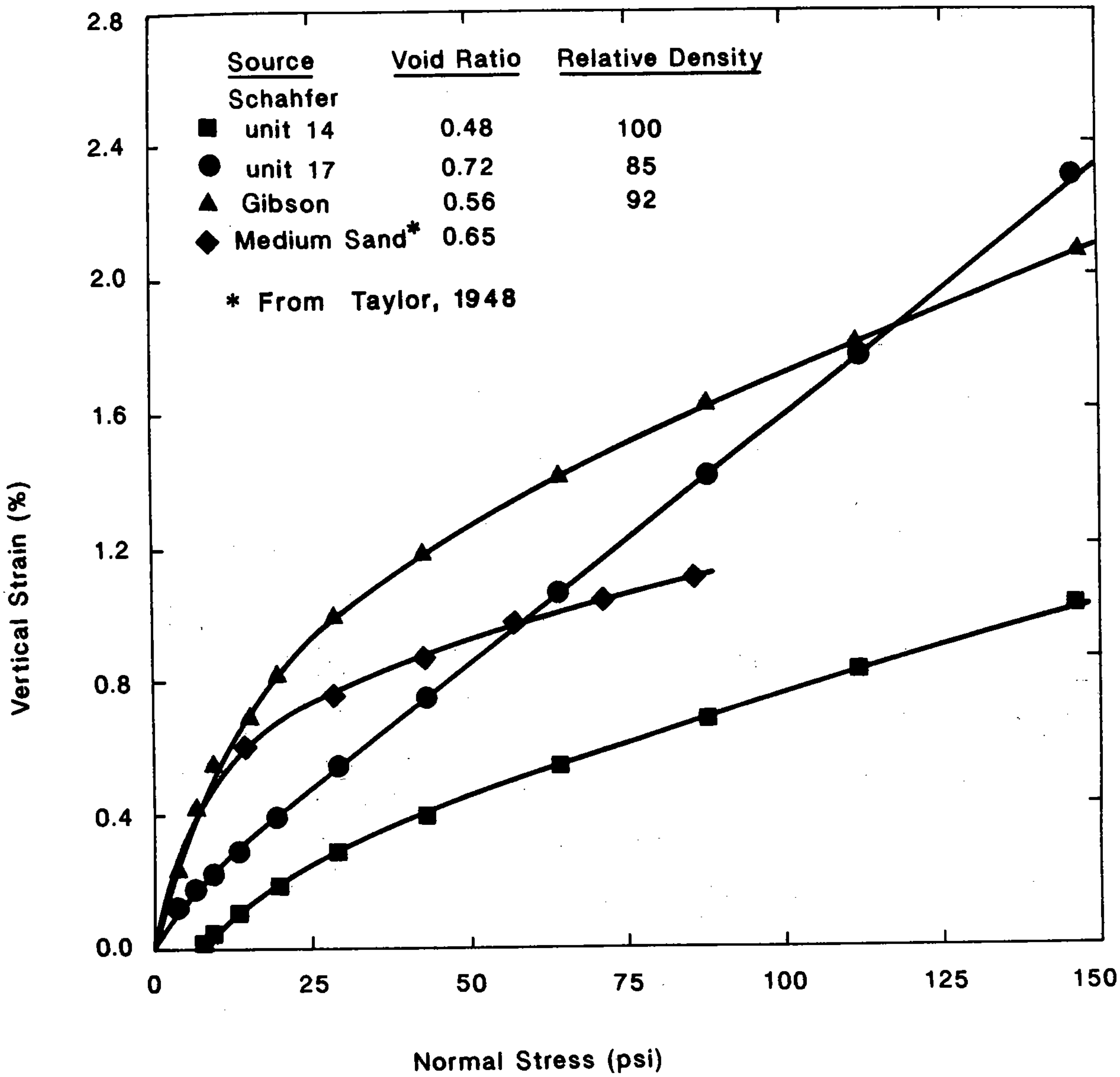

FIGURE 6. VERTICAL STRAIN VS. NORMAL STRESS

The secant constrained modulus is defined as

$$D = \frac{\Delta\sigma_v}{\Delta\epsilon_v} = \frac{\sigma_{v2} - \sigma_{v1}}{\epsilon_{v2} - \epsilon_{v1}}$$

where D = secant constrained modulus,

ϵ_{v1} = vertical strain at a stress level of σ_{v1}, and

ϵ_{v2} = vertical strain at a stress level of σ_{v2}.

Figure 7 shows the secant constrained modulus calculated from zero stress to various stress levels. In order to relate the secant constrained modulus of bottom ash to more familiar soil materials, a comparison is made between the D values for bottom ash and those for well-graded sand (Lambe and Whitman, 1969), and the results are shown in Table 5. It is found that the moduli for one ash (Schahfer unit 14) are comparable to those of the well-graded sand. The values for the other two ashes are somewhat lower than those for sand, especially at the high stress level. The crushing of the angular particles at high stress may play an important part in the phenomenon.

TABLE 5. SECANT CONSTRAINED MODULUS OF BOTTOM ASHES

Materials	Relative density	Modulus (psi x 10^3) $\sigma_{v1} = 9$ psi, $\sigma_{v2} = 15$ psi	Modulus (psi x 10^3) $\sigma_{v1} = 29$ psi, $\sigma_{v2} = 74$ psi
Schahfer			
unit 14	100	7.2	14.3
unit 17	92	5.4	7.2
Gibson	85	3.3	8.9
Well-graded sand[a]	0	2.0	3.7
(0.02mm<D^b<1mm)	100	7.5	17.6

.[a] Lambe and Whitman, 1969

.[b] D = particle size

Environmental Effects

The environmental concerns regarding the use of ash center around possible leaching of heavy metals and soluble salts from ash-constructed embankments. Leachate containing heavy metals and salts may enter the ground water system and contaminate present or future drinking water sources. Small amounts of heavy metals released to the environment may constitute a hazard both to environment and health. The high content of salts may adversely affect the

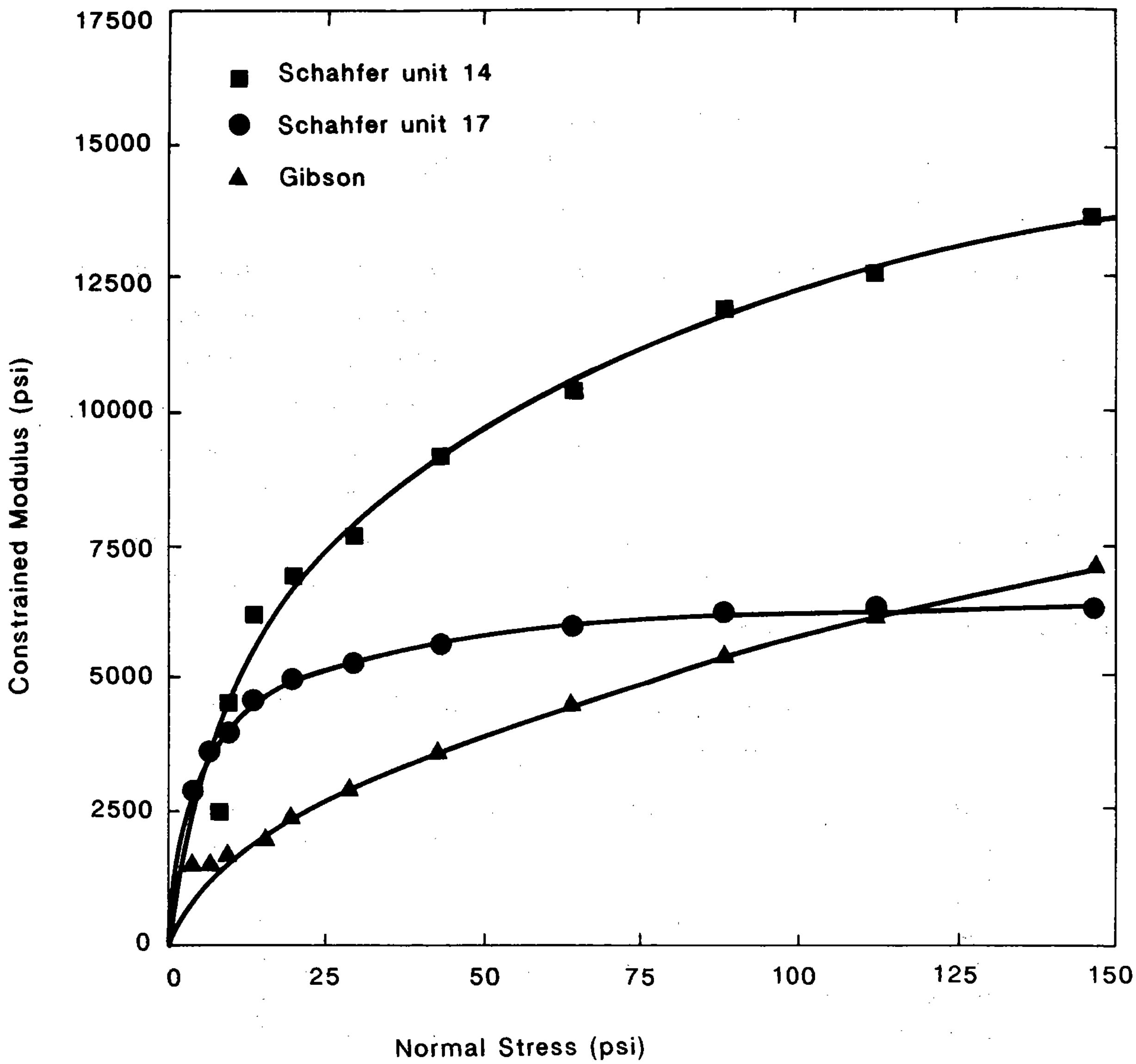

FIGURE 7. CONSTRAINED MODULUS OF BOTTOM ASHES

quality of ground water, although it does not constitute any danger to human health. These salts are principally calcium and sulfate, but also sodium, potassium, magnesium and chloride.

The chemical composition of ash is important to the leaching processes, but it forms an insufficient basis for an estimate of the leachate composition (contaminants and concentrations). Consequently, the environmental effects resulting from ash fills must be evaluated based on direct analyses of the leachate properties.

The Environmental Protection Agency (EPA) designed an Extraction Procedure (EP) toxicity test to simulate the leaching a solid waste will undergo in a sanitary landfill (EPA, 1985). In this test a representative sample of a solid waste is extracted with deionized water maintained at a pH of 5 using acetic acid. Table 6 summarizes the results from the analysis of bottom ash leachate generated by the EP toxicity test. The concentrations for bottom ash extract are far below the maximum concentrations specified by the EPA for characterizing hazardous solid wastes. Therefore, bottom ashes are characterized by the EP toxicity test as nonhazardous.

TABLE 6. RESULTS OF EP TOXICITY TESTS

Contaminant	Concentrations (mg/L)				EPA Maximum Allowable
	Schahfer unit 17	Gibson	Schahfer unit 14	Perry	
Mercury	0.0002	0.0001	ND*	0.0002	0.2
Silver	0.001	ND*	ND*	ND*	5.0
Cadmium	0.0008	0.025	0.0007	0.0004	1.0
Chromium	0.0009	0.0005	0.0012	0.0009	5.0
Arsenic	0.020	0.010	0.005	0.008	5.0
Selenium	0.005	0.005	0.003	0.004	1.0
Barium	0.098	0.103	0.136	0.108	100.0
Lead	0.007	0.002	ND*	0.005	5.0

* ND = non-detectable

The salt content of bottom ash leachate was tested by the leaching method test specified in the Indiana Administrative Code 329 IAC 2-9-3 (Indiana Register, 1989). The Indiana leaching method test is conducted as specified for the EP toxicity test, except with no addition of acetic acid. Table 7 summarizes the test results and the maximum concentrations specified for restricted waste site type IV, which is the most restrictive type of waste facility in the code. Again, the salt concentrations of the bottom ash extracts meet all the requirements.

TABLE 7. RESULTS OF INDIANA LEACHING METHOD TEST

Contaminant	Concentrations (mg/L)				Indiana Maximum Allowable
	Schahfer unit 17	Gibson	Schahfer unit 14	Perry	
Barium	0.098	0.103	0.136	0.108	1
Chlorides	<1	<1	<1	1	250
Sodium	0.8	1.0	<0.5	1.5	250
Sulfate	31	55	19	26	250
Total Dissolved Solids	90	140	10	145	500
Calcium	19	24	2	30	-
Magnesium	0.7	2.0	0.2	0.1	-
Potasium	1.0	0.7	0.1	2.0	-

Due to the nature of the transport system, it was not possible to sample bottom ashes that had not been exposed to some degree of leaching, if the ash was wet-disposed. In this study, only Perry ash could be sampled directly from the hopper. No significant difference in the concentrations of leachate was observed between Perry and other ashes.

SUMMARY AND CONCLUSIONS

Laboratory studies on the physical and chemical properties of bottom ash, and potential environmental effects resulting from the use of bottom ash have been presented. Based on the laboratory studies, it can be concluded that the properties of bottom ash compare favorably with those of traditional natural granular soils, and that the material can be successfully utilized in a variety of highway applications, e.g., highway subgrades, subbases, and embankments. Before this can be achieved, trial uses in the field and modification of existing material specifications may be required.

Based on the results from leaching tests performed in the laboratory, it is believed that the bottom ashes are nonhazardous and that their effects on the quality of ground water are minimal.

It is obvious that utilization of such an extensively produced byproduct of the power industry as a highway construction material could become more desirable in the future. If the conclusions from the present work hold true for other power plants, the prospects for bottom ash use in highway construction are very encouraging.

REFERENCES

1. AASHTO T216-81, "One-Dimensional Consolidation Properties of Soils," Designation T216-81, Standard Method of Test, American Association of State Highway and Transportation Officials, 1986.

2. ASTM C88-83, "Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate," Designation C88-83, 1985 Annual Book of Standards, American Society for Testing and Materials.

3. ASTM C131-81, "Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine," Designation C131-81, 1985 Annual Book of Standards, American Society for Testing and Materials.

4. Das, B. M., Tarquin, A. J., and Jones, A. D., 1983, "Geotechnical Properties of a Copper Slag," Transportation Research Record 941, pp. 1-4.

5. Diamond, S., 1985, Selection and Use of Fly Ash for Highway Concrete, Report JHRP-85-8, Purdue University, West Lafayette, Indiana.

6. EPA, 1985, Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, SW-846, 2nd ed., Environmental Protection Agency.

7. Foster, C. R., 1962, "Field Problems: Compaction," Foundation Engineering, G. A. Leonards (ed.), McGraw-Hill, New York, p. 1010.

8. Holtz, R. D. and Kovacs, W. D., 1981, An Introduction to Geotechnical Engineering, Prentice-Hall, Inc., New Jersey, p. 286.

9. Hough, B. K., 1969, Basic Soils Engineering, 2nd ed., The Ronald Press Company, New York, p. 30 and p. 76.

10. Indiana Department of Highways, 1988, Standard Specifications, State of Indiana.

11. Indiana Register, 1989, Volume 12, Number 5, February 1.

12. Lambe, T. W. and Whitman, R. V., 1969, Soil Mechanics, John Wiley and Sons, Inc., New York, p. 155.

13. Moulton, L. K., Seals, R. K., and Anderson, D. A., 1973, "Utilization of Ash from Coal-Burning Power Plants in Highway Construction," Highway Research Record 430, pp. 26-39.

14. Taylor, D. W., 1948, Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, p. 216.

15. Terzaghi, K. and R. B. Peck, 1967, Soil Mechanics in Engineering Practice, 2nd ed., John Wiley & Sons, Inc., New York, p. 381.

16. Zeevaert, L., 1982, Foundation Engineering for Difficult Subsoil Conditions, 2nd ed., Van Nostrand Reinhold Company Inc., New York, p. 140.

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GEOGRID REINFORCEMENT FOR COCHRANE BRIDGE EMBANKMENT

ABSTRACT

Construction of a 6.62 m (21.7 ft) high bridge approach embankment over weak marsh deposits in Mobile, Alabama was accomplished through the use of geogrid reinforcing and wick drains, resulting in an estimated savings of $600,000. Lightweight polypropylene geogrids and a nonwoven polyester geotextile were used in tandem to permit the construction of a 0.61 m (2 ft) sand blanket across the marsh. The sand blanket served as a working platform for the installation of wick drains which were used to reduce the time required for settlement of the embankment to less than two months. High density polyethylene geogrids were employed to reinforce the embankment so that full embankment height could be constructed without incurring the time delays of stage construction. The advantages of geogrid reinforcement and the construction procedure are discussed.

INTRODUCTION

The first bridge encountered on the recently completed Tennessee-Tombigbee Waterway is U.S. Alternate 90 which crosses the waterway on the north side of Mobile, Alabama from the mainland to Blakely Island where the highway runs due south along the edge of the Mobile River until it reaches an interchange with Interstate 10 immediately east of the twin tunnels under the river. This route is important in that hazardous or explosive materials are not allowed to be carried through tunnels and this route is the only east-west access to or from Mobile for these truck cargos. The bridge crossing the waterway is known as the Cochrane Bridge and the original structure was a lift span to allow clearance of river traffic under the bridge. In the early 1900's, travel from the eastern shore area of Mobile Bay to the city was by ferry which ran from Fairhope, Alabama to the Mobile city docks. The Cochrane Bridge Company was formed in the mid 1920's and was responsible for the design and construction of the bridge and a causeway which crossed the upper reaches of Mobile Bay to the eastern shore at Spanish Fort, Alabama. This private toll facility operated until the early 1940's when it was acquired by the Alabama Highway Department.

Due to the deterioration of the old structure and to the vertical clearance requirements for the Tennessee-Tombigbee Waterway, it was determined that the Cochrane Bridge would be demolished and replaced by a new bridge having a vertical clearance of 42.68 m (140 ft) and a main span length of 237.80 m (780 ft). In order to avoid the sharp curve on the east approach of the old bridge, the new facility was extended to a length of 2222.86 m (7291 ft) with a curving alignment that swung across a dredge disposal area and marshlands on Blakely Island. This paper presents the design and construction of the south bridge approach embankment.

SITE CONDITIONS

Due to the extremely soft conditions in the marsh, borings were taken along the shoulder of existing U.S. 90. The surface of the marsh lies at approximately elevation +0.61 m (+2 ft). The borings revealed that the bottom of the existing embankment ranged from elevation -0.91 m (-3 ft) to -1.52 m (-5 ft). Beneath the embankment prism, a stratum of silty to sandy clay ran throughout the site. Grain size analyses showed that the sand fraction for this stratum held constant at approximately 33% with the silt and clay fractions varying considerably throughout the deposit. Accordingly, moisture contents varied from 45% to 105%. The thickness of this stratum ranged from 2.74 m (9 ft) to 5.18 m (17 ft) and liquid limits varied from a low of 32 to a high of 55 with P.I.'s ranging from 9 to 30. The results of unconfined compression tests showed shear strengths ranging from a low of 8.14 kPa (170 psf) to a high of 11.01 kPa (230 psf) near the bottom of the stratum. The standard penetration test results throughout this stratum were basically uniform and showed N-valves of 2 blows per foot.

The second stratum ranged in thickness from 3.05 m (10 ft) to 6.71 m (22 ft) and consisted of silty sands to clayey sands. Standard penetration tests in this material showed N-valves ranging from 2 to 6 with moisture contents ranging from 23 to 33%. The major fraction of this material was predominately sand and the majority of the Atterbreg Limits showed the material to be nonplastic.

Beneath this layer, a stratum of medium dense to dense sand was found down to elevation -18.3 m (-60 ft). Standard peneration test results showed blow counts ranging from 14 up to 50 blows per foot.

Undisturbed samples taken from the first stratum of silty clay revealed a large variance in the material. Void ratios generally fell bwtween 1.0 and 2.0, however, some samples revealed void ratios as high as 5.0 with a significant organic content. The dry unit weight for the material ranged form a low of 3.14 kN/m^3 (20 pcf) near the surface to 8.64 kN/m^3 (55 pcf) near the bottom of the stratum. The average coefficient of consolidation was 41.5 m^2/s (0.386 ft2/day).

Since the borings were taken along the shoulder of the existing embankment, the results are indicative of a preconsolidated condition of the marsh deposits due to the imposed stresses of the existing highway. The surface of the marsh outside of the existing embankment was extremely soft and virtually impossible to traverse by foot. Based on the data obtained the shear strength of the virgin marsh was estimated to vary linearly from 0 at the surface to 10.39 kPa (217 psf) at the bottom of the stratum.

EMBANKMENT DESIGN

The construction schedule for the project did not allow sufficient time for stage construction of the bridge approach embankment. Cost estimates of several schemes showed that the use of wick drains coupled with geogrid reinforcement provided the most economical approach.

Based on the tight construction schedule, only two months could be used to achieve 100% consolidation under the embankment. Analyses were conducted to determine the optimum spacing of wick drains which would achieve an estimated settlement of 1.7 m (5.6 ft) beneath the maximum fill height of 6.62 m (21.7 ft).

It was determined that an embankment construction rate of 0.30 m/day (1.0 ft/day) coupled with a 1.22 m (4.0 ft) triangular spacing for the wick drains would produce a construction and surcharging period which fit the overall schedule for the project (Fig. 1 and 2). Using this criteria, it was then possible to estimate the residual piezometric pressures which would exist within the marsh deposits at any time during the construction of the embankment (Figure 3). These estimates of the piezometric regime during construction enabled the shear strength of the underlying soft deposits to be calculated at various times during construction and stability analyses were conducted to ascertain the amount of horizontal reinforcing required to achieve a minimum factor of safety of 1.20. Using this approach, it was determined that six layers of Tensar SR2 geogrid would be required to achieve this minimum factor of safety at the bridge abutment. A profile of the embankment and the layout of the SR2 geogrid is presented in Figure 4.

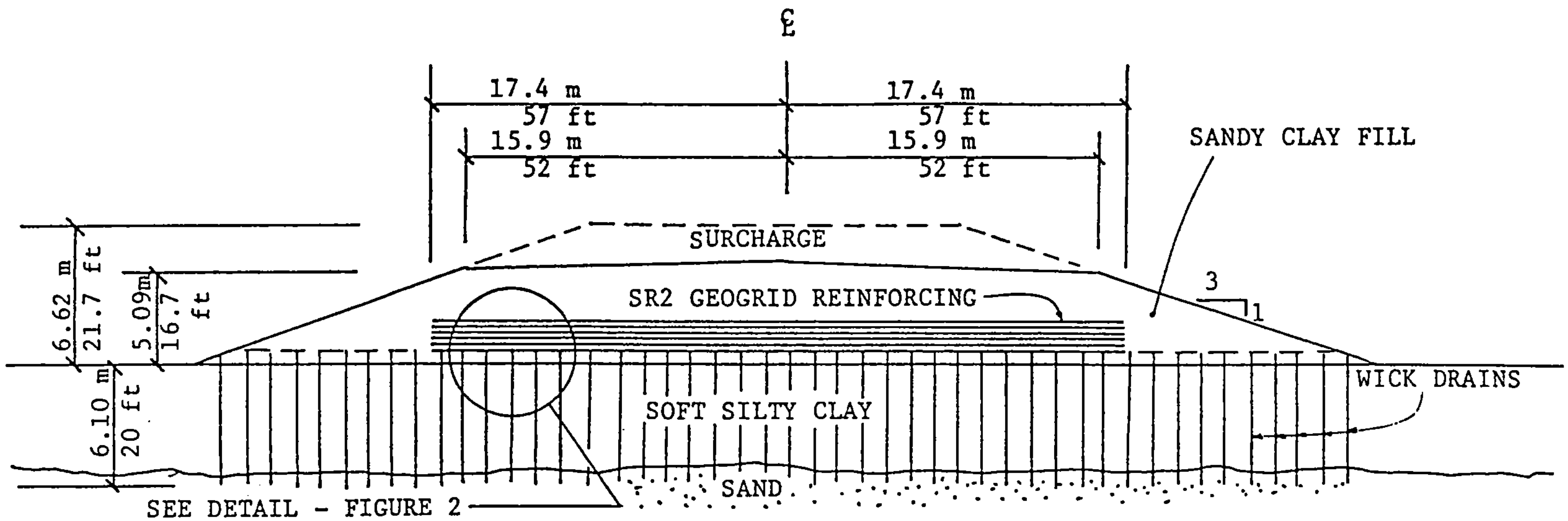

FIGURE 1: TYPICAL SECTION OF REINFORCED EMBANKMENT

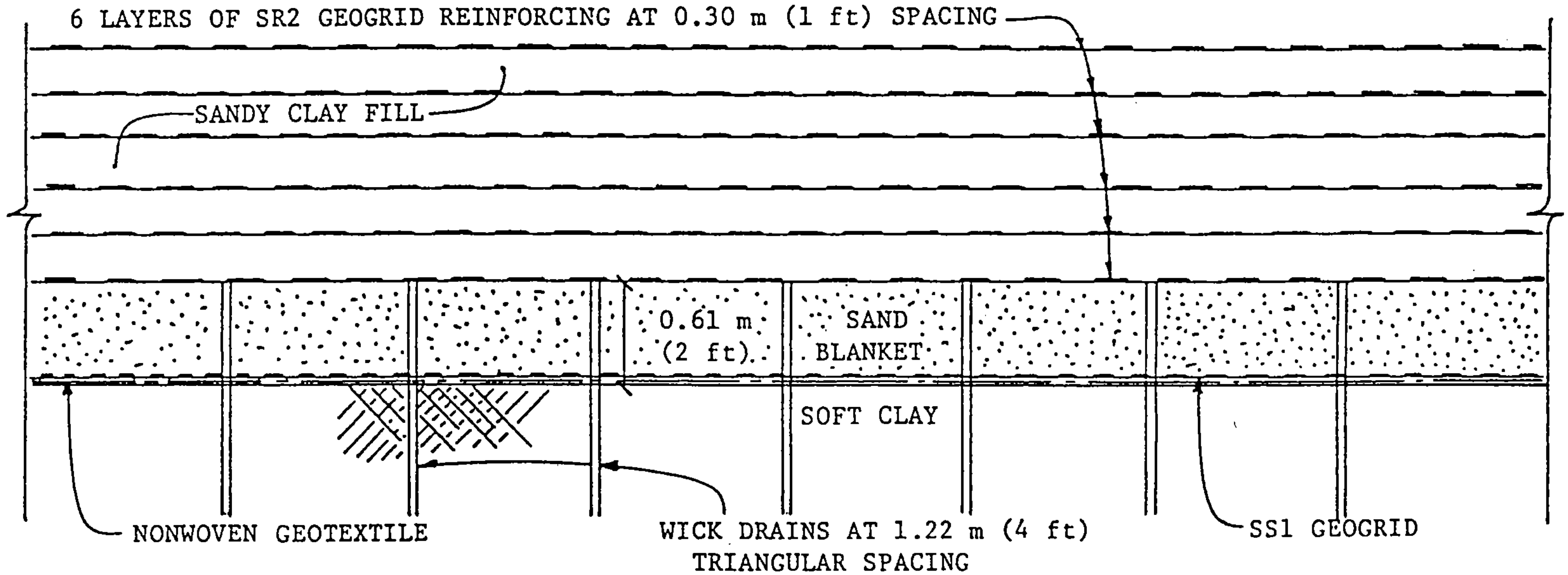

FIGURE 2: DETAIL OF EMBANKMENT REINFORCING AND WICK DRAINS

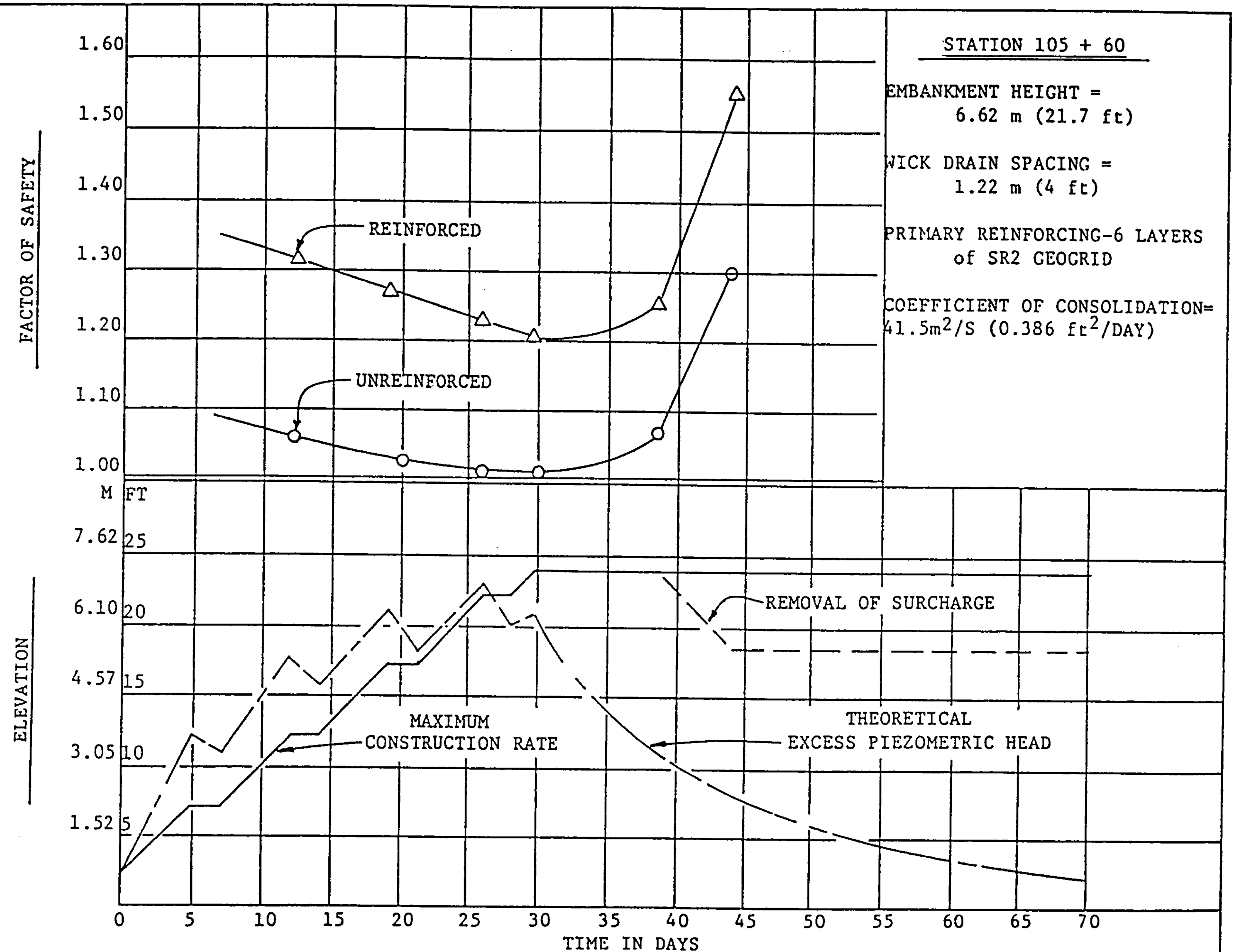

FIGURE 3: MAXIMUM CONSTRUCTION RATE, PIEZOMETRIC HEAD AND FACTORS OF SAFETY VS. TIME

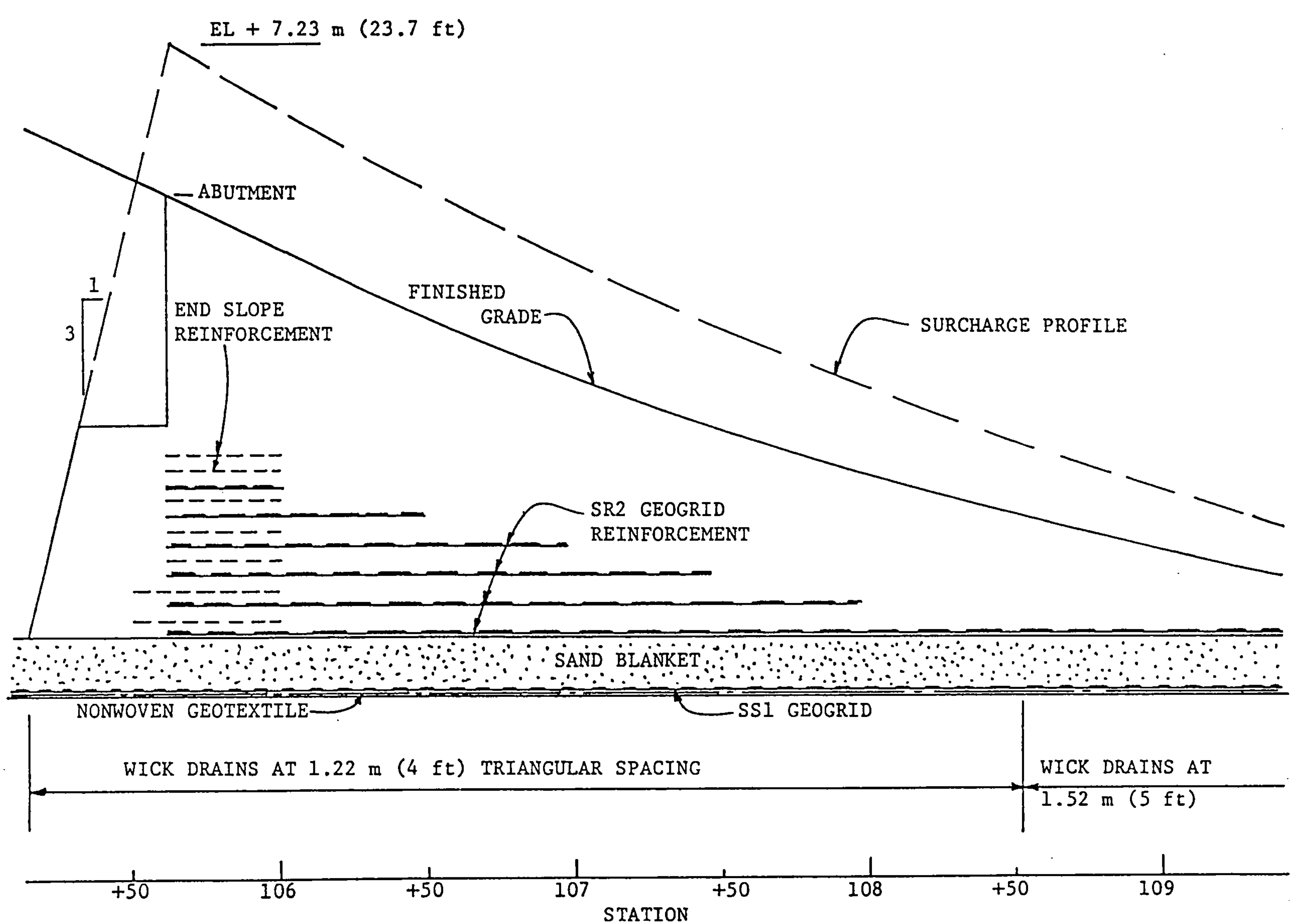

FIGURE 4: PROFILE OF REINFORCED BRIDGE APPROACH EMBANKMENT

Further analyses also showed that the wick drain spacing could be spread to 1.52 m (5 ft) on center for embankment-surcharge heights up to 4.57 m (15 ft) and still achieve the desired degree of consolidation within the specified surcharging period. This was made possible due to the fact that this portion of the embankment would reach final surcharge height while the filling process was continuing on the higher portions of the embankment near the bridge abutment. Therefore, a longer period of consolidation time was available for this portion of the embankment thus allowing the increase in wick drain spacing.

REINFORCEMENT GEOMETRY

Normally, the reinforcement for an embankment over a soft foundation is placed at the bottom of the embankment directly on the soft soil. This procedure was not used for three primary reasons: (1) Installation of wick drains through the reinforcement would significantly reduce the tensile capacity of the reinforcement (2) expensive, high strength reinforcement would be placed in areas where it was not needed and (3) placement of reinforcement on soft soil significantly reduces the anchoring effectiveness of the reinforcement.

Based on these assessments, it was obvious that two reinforcing requirements were necessary for project construction. The first was reinforcement for the sand blanket which would also serve as a working platform for wick drain installation. This reinforcement was provided by the SS1 geogrid. The interlocking capacity of this grid coupled with it's flexural rigidity, which produces a "snowshoe" effect, would create a stiff working platform with only a 0.61 m (2.0 ft) thick layer of sand. This reinforced working platform would provide excellent support for the equipment used in the installation of the wick drains. A lightweight nonwoven geotextile was used under the SS1 geogrid to prevent any possible contamination of the sand blanket which would impede the flow of water from the wick drains (Figure 2).

The use of the reinforced working platform allowed the primary SR2 geogrid to be placed in the embankment where the reinforcing effects were fully optimize. Since the SR2 geogrid was placed in select fill, full anchorage of the grid beyond potential failure planes could be achieved with a minimum embedment length. Through stability analyses it was determined that primary reinforcing was required for only 56% of the base width of the embankment (Figure 1).

Since the bridge approach was set on a grade of 4.7%, further economy could be achieved through the reduction in reinforcement as the embankment height dropped. The final layout of primary reinforcing in the embankment profile is presented in figure 4.

EMBANKMENT CONSTRUCTION

Construction of the project began in February, 1986. The first step of construction on the bridge approach embankment was the installation of the sand blanket on the marsh surface on both the east and west sides of the existing facility. This was accomplished through the use of the lightweight nonwoven geotextile and the Tensar SS1 geogrid for reinforcement. The placement of the sand blanket was achieved with virtually no mudwaving or shear distortions occurring either in front of or at the side of the filling operation. Wick drains were then installed using a hydraulic activated mandrel to a depth of 6.10 m (20 ft). The wick drain equipment was mounted on a Cat 235 backhoe with the wick drain material being supplied to the installation equipment by truck. No mobilization difficulties were encountered during the wick drain installation. The subcontractor achieved a production rate of over 3048.7 m (10,000 ft) of drains installed per eight hour shift. The subcontractor chose Amerdrain for installaton on this project.

While the wick drain installation was proceeding, the contractor began the removal of the pavement and base of the old U.S. 90 facility. This area was undercut 0.61 m (2 ft) and the sand blanket was continued across the old embankment prism. When this work was completed, the wick drain installation began in this area. The wick drain subcontractor immediately ran into penetration difficulties near the surface. It was discovered that two other old layers of asphalt existed below sea level. These old pavements had not been encountered in the original soil borings since they had been taken on the shoulders of the existing four lane facility. It became apparent that the ancient pavements were initially constructed for a two lane roadway which subsequently settled, thus necessitating reconstruction over the marsh area in order that the surface of the roadway be maintained at an elevation above storm flood level.

It was determined that the most feasible approach to overcoming this obstacle to drain installation would be predrilling through the old pavement followed by installation of the drains. Approximately 1865 m (6,118 ft) of predrilled holes were required to penetrate the ancient pavement systems. The installation of 90,180 m (295,792 lf) of wick drain for the mainline facility was completed in mid October, 1986.

After the installation of the wick drains, the first layer of primary reinforcing was placed directly on top of the sand blanket from station 105+60 to station 110+00, (Figures 1 and 4). The truck hauled embankment material was then spread and compacted until the next prescribed elevation for reinforcing was reached. Each layer of geogrid was precut to the design length and field laydown was accomplished very quickly. The contractor reported that no difficulties were encountered with either the laydown operation or the spreading and compaction operation.

The fill reached final surcharge elevation on November 19, 1986. Four clusters of piezometers had been installed with each cluster having a piezometer located at elevations-1.52 m (-5 ft), -3.05 m (-10 ft) and -4.57 m (-15 ft). The rise in pore pressure closely tracked the prediction (Figure 3) in the upper and center piezometers. Of these eight pieozometers, seven gave residual heads at or below the predicted level, while one showed a head of 0.76 m (2.5 ft) above the predicted level. All of the piezometers located at elevation -4.57 m (-15 ft) read pore pressures consistent with the original ground water table at elevation +0.61 m (+2 ft), thus indicating that they were placed in the sand stratum which underlies the compressible soils.

Both the piezometers and the settlement plate data showed that the 100% consolidation mark was achieved within the specified surcharge time frame. The surcharge was then removed and construction on the balance of the project is still underway. Total settlements in the range of 1.8 m (5.9 ft) occurred in the areas of maximum embankment height.

There was no evidence of embankment cracking, lateral spreading or shear displacements in the marsh either during or after embankment construction.

CONCLUSION

The success of this project demonstrates the economic advantages that can be achieved through multilayer reinforcing provided by geogrids. For a given cross section, the primary reinforcing is placed only where needed and the required anchoring length is optimize by placement of the grid in select material. Varying reinforcement requirements can be easily achieved through the use of multilayer reinforcing. In addition, the integrity of the reinforcement is not threatened by the installation of wick drains.

Properly designed, multilayer geogrid reinforcing systems will offer many technical and economical advantages over single layer high strength reinforcement.

Update on STABL...PCSTABL5M

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ABSTRACT

The widespread use of the STABL program since its introduction in 1975 has produced direct feedback from the users suggesting further revisions and improvements. This updating policy has created, through the years, a program which is progressively more versatile, practical and reliable. The program's latest version, (PCSTABL5M) incorporates options that improve the reliability and consistency of the analysis. It is the purpose of this paper to explain in detail these modifications and their effects on the resulting factors of safety.

INTRODUCTION

STABL is a computer program for the general analysis of slope stability by a two dimensional (2-D) limiting equilibrium method. The program uses the method of slices to analyze the slope and calculates the factor of safety (FOS) according to the simplified Janbu , simplified Bishop , or Spencer methods, for any kinematically acceptable sliding surface. A unique feature of the program allows random surfaces to be generated, allowing the user to determine the critical, minimum FOS more easily.

Since STABL was developed by Ronald A. Siegel in 1975 it has undergone numerous modifications and revisions. The most important of these modifications were the introduction of the tieback load analysis (Carpenter; 1984), and the Spencer method of slices (Carpenter;1985). The latest version of STABL is called PCSTABL5M, and it is written for the Microsoft Fortran compiler package. The present paper explains the modifications in PCSTABL5M, with accompanying discussion of background information.

Four modifications and improvements are described. The first modification enables the user to use Janbu's empirical coefficient for a more rigorous generalized method of slope analysis. The second improvement is an increase in the options available for pore pressure determination. By this modification three different methods are available to the user for pore pressure determination. The third improvement concerns the occasional malfunction in the routine SURBIS, which evaluates the FOS for a defined sliding surface. The fourth improvement allows the data on each slice of the most critical failure surface to be printed out.

JANBU'S CORRECTION COEFFICIENT

PCSTABL5 uses Janbu's simplified analysis for generalized slide surfaces, which neglects the interslice forces (Siegel, 1975a; Siegel 1975b; Boutrup, 1977). This simplified solution can be improved by introducing a correction coefficient fo, to take account of the interslice shear forces. The factor fo depends on the strength intercept and angle of shearing resistance; and on the geometry of the slide surface defined by the ratio d/L, where d is the largest slice depth, and L is the sloped distance between the ends of the slide surface (Figure 1). The former versions of STABL did not incorporate this correction factor, which may lead to conservativism of as much as 13% for deep surfaces in cohesive soils. In STABL5M this correction factor is automatically calculated, based on a set of three equations, which represent the curves appearing in Figure 2, (Janbu;1954).

(a) for $\phi'=0$

$$fo=1.0+0.69[(d/L)-1.4(d/L)^2]$$

(b) for $\phi'>0$ and $c'>0$

$$fo=1.0+0.5[(d/L)-1.4(d/L)^2]$$

(c) for $c'=0$

$$fo=1.0+0.31[(d/L)-1.4(d/L)^2]$$

The last two equations were taken from Hoek & Bray; 1981, while the first one is a logical inference from the other two.

PCSTABL5M allows the user to apply the correction factor, or not, and also to decide which case of the three above is appropriate. For homogeneous slopes, it is simple to choose one of the three cases, but for layered slopes, the selection can sometimes be difficult. For such cases, judgment should be used, or a more general slices method, e.g., Spencer should be selected.

It is very important to realize that the use of a correction factor does not elevate simplified methods to the category of general methods. The ignoring of interslice forces in Janbu's Simplified Method can produce non representative factors of safety, when large and non uniformly distributed pore pressures are present in the slope. The use of the correction factor will NOT remove this difficulty. Thus it must be kept in mind that although Janbu's Simplified Method is a flexible, rapid and widespread method of analysis, it is an approximate method, and can not substitute for more accurate methods in the presence of complex and/or critical conditions.

PORE PRESSURES

PCSTABL5M, as its predecessors, allows the user to define pore pressure conditions by using phreatic lines. Excess pore water pressure due to shear is taken in account by the single parameter (r_u) which is related to the overburden pressure. However, a phreatic line can not accurately describe the pore pressures in the soil beneath it, without the appropriate estimate of the associated equipotentials. Different failure surfaces, have different piezometric lines for a single phreatic line. Since an accurate estimate of such equipotentials for general situations can only be performed by finite differences or similar methods, the former versions of STABL (STABL2 to STABL4) used a conservative approach of calculating the steady state pore pressure at the bases of slices, based on hydrostatic pressure. That means that the vertical distance from the base of the slice to the phreatic line immediately above was taken as the water head (Figure 3). However this procedure was thought to be too conservative, overestimating pore pressures.

An approximate method has been proposed (STABL5) where the equipotential passing through the base of the slice would be approximated by a straight line normal to the tangent to the phreatic line at the top of the slice (Figure 3). This method, called the perpendicular method, is considered less conservative than the piezometric method, and in some cases is unconservative.

In PCSTABL5M another method was introduced, which was simply the average of the two methods which was supposed to reduce the excessive conservatism of the piezometric method, while preventing the nonconservative estimates of the pore pressures resulting from the perpendicular approach (Figure 3).

However, it was felt that none of these methods was able to approximate all the flow net conditions encountered in practice. In fact, there was no evidence for which method to be used in different cases. To clarify this situation a parametric study was carried out using four separate STABL programs , the only difference among them being the pore pressure determination (Achilleos; 1989). The four programs were named F1, F2, F3, and F4 corresponding to the piezometric, the perpendicular, the PCSTABL5M, and the true method respectively. A common critical failure surface was used for all the programs.The study was carried out for six different cases. The cases were selected in such a way so as to represented situations encountered in practice.Three different types of soils, a purely cohesive soil (c'=300 psf, ϕ'=0), a purely cohesionless soil (c'=0,ϕ'=30 deg), and a soil having c'=150 psf and ϕ'=20 deg, were used for both Janbu and Bishop methods of slices.

The results obtained showed that for rotational failure surfaces the difference between the factors of safety calculated with the different methods was small. The piezometric method was found to be the most conservative, with a conservatism ranging from 1.5-5.0%, while the perpendicular method was found to be slightly unconservative, with an unconservatism of 0.3-2.1%. The PCSTABL5M produced values in between. However, for translational slides, the results were different. In such a case the piezometric and PCSTABL5M methods can be shown to be conservative. The results obtained showed a conservatism up to 27.0% and 15.0% for the piezometric and the PCSTABL5M approaches respectively. On the contrary, the perpendicular method attained unconservative values up to 2.5%. The study did not show any significance difference in the results obtained by the Janbu or the Bishop methods. However the results were quite different for different soils. Granular soils in general showed much more sensitivity to changes in pore pressure than cohesive soils. For example, for one case involving a translational slide the percent conservatism of the piezometric method for cohesionless soils increased to 27.0%, while that of cohesive soils increased only 2.3%.

Since no single method can model all flow net conditions it was thought that the best solution to the pore pressure problem is to install all three available approximate methods in STABL, and allow the user to choose which method to use. It is expected that such a modification will give a reasonable number of options for users who are involved in a wide variety of different pore pressure situations.

MODIFICATION OF THE ROUTINE SURBIS

The routine SURBIS evaluates the factor of safety of user-defined surfaces using the simplified Bishop method. This method was developed to be used with circular surfaces, and the program thus assumes that the points used to define the surface follow a circular pattern. In the former versions of STABL, the three first points of the defined surface were used to calculate the center about which the moment equilibrium should be performed (as well as the circle radius). This procedure was found to be the source of inaccuracies for the following reasons:
a) The first three points are usually very close together and represent a very small portion of the surface. Consequently scale effects would increase the influence of any minor misalignment on the calculation of the center.
b) This fitting method is very sensitive to slight variations of accuracy in the coordinates of the points, including rounding of decimals.

The new PCSTABL5M use the first, the last and the middle points to calculate the center of rotation. This modification virtually eliminates the former problems. Besides, it better represents the way data are entered for back analysis of real situations, where the initial and final points of the failure surface are known with some certainty, as well as some data defining the middle section of the slide.

SLICE DATA

In PCSTABL5M data on each slice of the critical failure surface are printed out along with the coordinates of this surface and the factor of safety. Data on each slice include thickness, weight, water force on top of the slice, water force on the bottom of the slice, normal tieback force, tangential tieback force, boundary load, horizontal earthquake force and vertical earthquake force. These forces correspond to Figure 4. All improvements were made in the subroutine RANDOM.

CONCLUSIONS

This paper presented the latest improvements of the slope stability program STABL. The new PCSTABL5M now incorporates Janbu's correction factor and has the ability to calculate pore pressure using three different methods. A correction in the subroutine SURBIS, and an ability to display slice data for the most critical surface further enhance the capabilities of STABL.

REFERENCES

1. Achilleos E., (1989), Pore Pressure Determination in Slopes, Internal Report No 147, Ground Engineering, School of Civil Engineering, Purdue University, West Lafayette,Indiana, 37 pp.

2. Boutrup E., (1977), Computerized Slope Stability Analysis for Indiana Highways, Joint Highway Research Project No. 77-25 and 77-26 (2 volumes), School of Civil Engineering, Purdue University, West Lafayette, Indiana, 512 pp.

3. Carpenter J., (1984), Slope Stability Analysis With STABL4, Joint Highway Research Project No. 84-19, School of Civil Engineering, Purdue University, West Lafayette, Indiana, 109 pp.

4. Carpenter J., (1985), STABL5... The Spencer Method of Slices: Final Report, Joint Highway Research Project No. 85-17, School of Civil Engineering, Purdue University, West Lafayette, Indiana, 26 pp.

5. Hoek, E. and Bray, J.W. (1981), Rock Slope Engineering, Institution of Mining and Metallurgy, London, 358 pages.

6. Janbu, N. (1954), Stability Analysis of Slopes with Dimensionless Parameters, Harvard University Soil Mechanics Series No. 46.

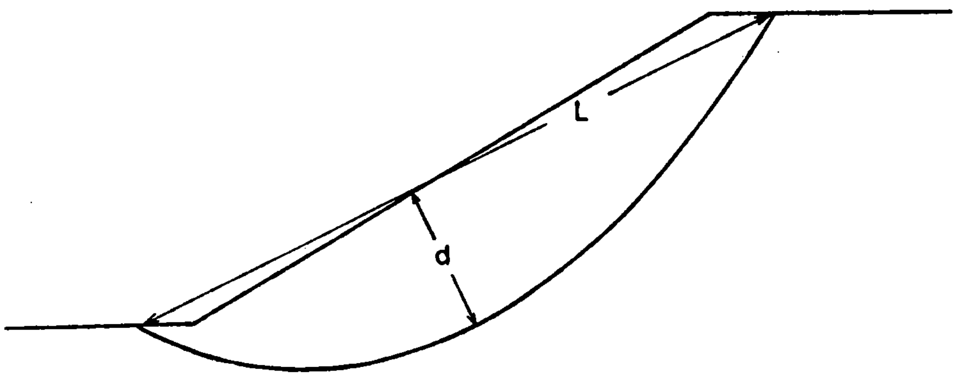

Figure 1. Definition of d & L.

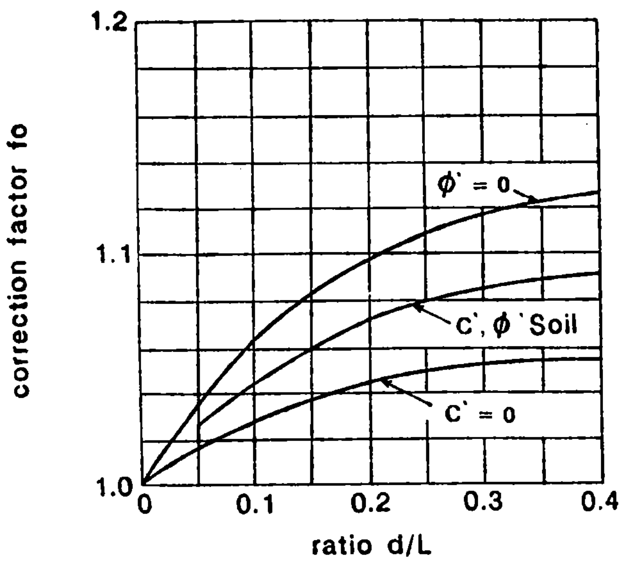

Figure 2. Janbu's Correction Factor Chart.

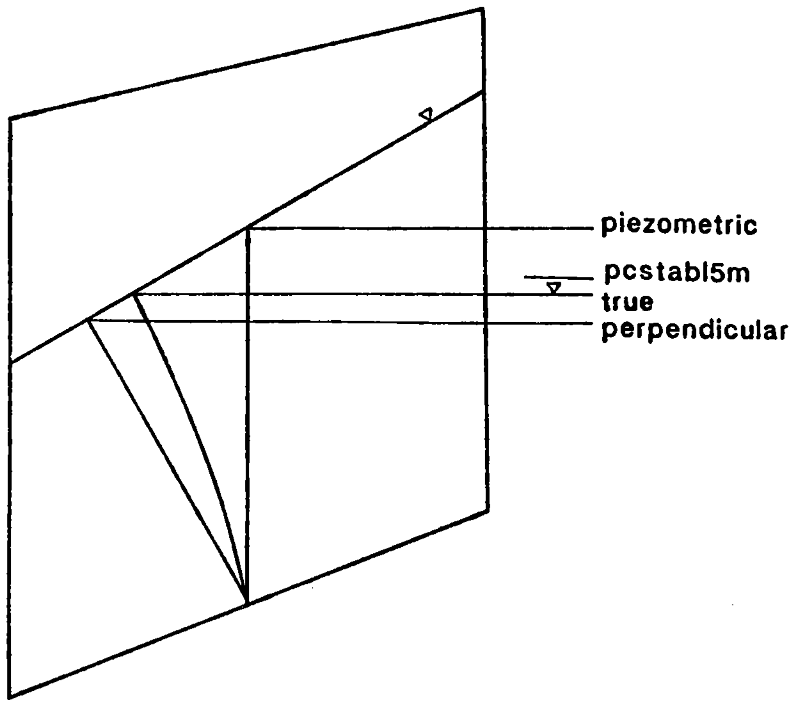

Figure 3. Methods of Pore Pressure Determination.

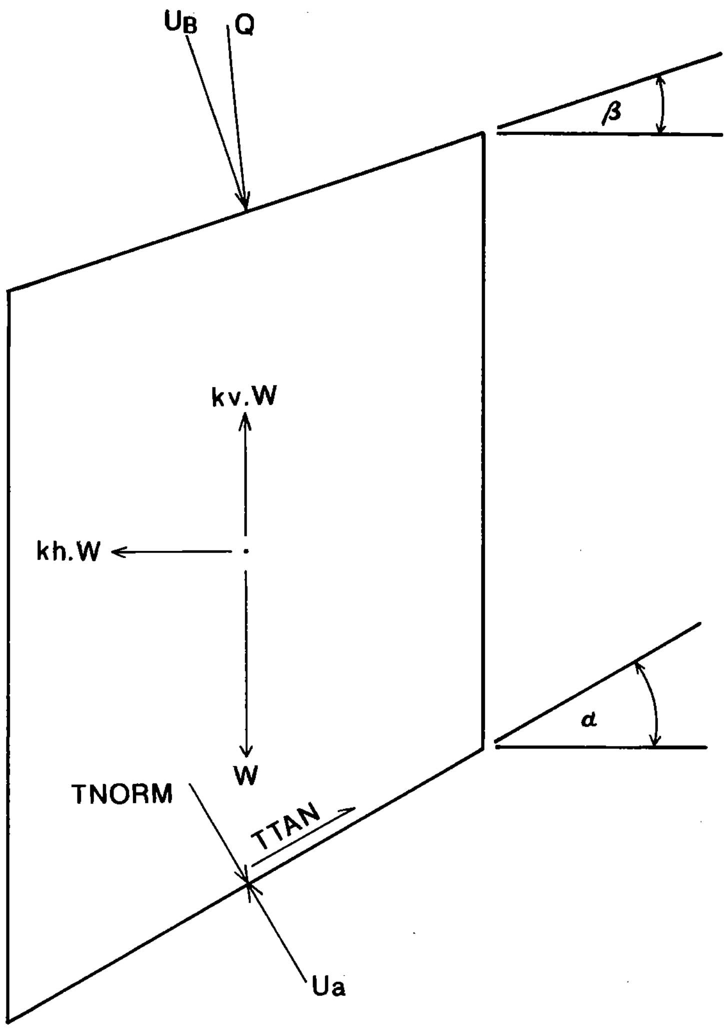

Figure 4. Slice Data.

ENGINEERING GEOLOGIC METHODS USED TO PLAN BUILDING DEMOLITION AT AN ABANDONED COAL MINE IN SOUTHWESTERN INDIANA

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ABSTRACT

Demolition of buildings and industrial facilities is an activity which is gaining in importance for three reasons: 1) rapidly advancing technology and changing economic factors are making many facilities in the old industrialized nations obsolete, 2) high land values require recycling some sites for modernized or alternative commercial uses, and 3) public awareness of environmental factors requires recycling other sites to clean up pollution and restore natural conditions. We feel that planning demolition work is important in order to reduce its direct cost, time for completion, and occupational or environmental hazard.

This paper suggests a procedure for planning demolition work which is based on an extension of standard engineering geologic methods of site investigation, including: 1) literature search and personal interviews, 2) aerial photograph interpretation, 3) field measurements on the surface and in test pits, and 4) office calculations and recommendations. This procedure is demonstrated by a recent case history, where we used it to plan the demolition of buildings at an abandoned coal mining and preparation facility in southwestern Indiana.

ACKNOWLEDGEMENTS

The financial support for this research was provided by the U.S. Department of the Interior, Office of Surface Mining, Abandoned Mine Lands Program through the State of Indiana, Department of Natural Resources. Mr. Steve Herbert at the Reclamation Division in Jasonville provided special large scale aerial photographic coverage and a topographic map of the site. Dr. Denver Harper at the Geological Survey Division in Bloomington provided information on coal geology and the history of coal mining in southwestern Indiana. The work described in this paper is part of a comprehensive research and development program being carried out at the Friar Tuck site by personnel of the Reclamation Division, Geological Survey Division, Indiana University and Purdue University.

INTRODUCTION

This paper outlines a procedure for planning building demolition which is similar to the standard procedure used for planning new construction projects. The problem can be defined in three parts: 1) description of the present land condition, 2) understanding the desired land use, and 3) developing performance criteria. The solution is accomplished in four steps: 1) literature search and personal interviews, 2) aerial photograph interpretation, 3) field measurements, and 4) office calculations and recommendations.

The main difference between demolition or reconstruction projects and new construction projects is that the engineering geologist spends most of his effort on man-made features instead of on natural features. This change in focus is not difficult for a geologist with a reasonably good engineering background or some practical experience in building construction. Furthermore, we feel that an engineering geologist, with typical skills of careful observation and description in the field, is better suited for this task than an architect or structural engineer, whose primary skills are conceptualization and design in the office.

During the past 50 years our profession has made significant progress in convincing cost-conscious owners that engineering geologic investigations can reduce the losses from failures or poor performance of new construction projects. Perhaps everyone has seen the cartoon showing an Italian dressed in 16th century garments and pointing to a drawing of a beautiful cylindrical shaped tower. The drawing is titled "Tower of Pizza" and the cartoon caption reads "We can save 500 lire by omitting the engineering geologic investigation". Obviously, history has proved this to be a poor cost-cutting decision because the tower leans noticeably, and it has required expensive foundation repair work.

At the present time structural demolition work is seldom taken more seriously than the removal of weeds and brush on a project site. We feel that this old attitude is not compatible with new changes in our culture. After a century of unparalleled industrial growth combined with technological obsolescence, the United States now has a large inventory of useless buildings and industrial structures. In some cases, demolition is necessary to clear a valuable location for reconstruction of new facilities serving the same land use, or for a new land use, such as a highway. In other cases, demolition is necessary to abate a direct hazard to the public or an environmental hazard. Although demolition for reconstruction has been common for centuries, demolition to abate hazards is mostly a new phenomena. Since the cost of this increasing amount of demolition work is skyrocketing along with other building trade work, we feel that it deserves more careful planning than has been done in the past. Planning is expected to reduce demolition cost the same way it reduces new construction costs. If the contractor knows exactly what is required of him, he can bid intelligently instead of increasing his bid to allow for the worst case scenario. Also, once work is in progress there is less chance of surprises which can generate expensive delays and change orders.

The demolition of a coal mining and preparation facility, which we discuss in this paper, is an example of the increasing amount of recent demolition work which is solely done to abate hazards. The facility is part of a two square mile site located on the boundary between Sullivan and Greene Counties in a rural area of southwestern Indiana near the town of Dugger. Two coal seams originally cropped out along the valley walls of a northward flowing tributary of Mud Creek. The upper seam of coal (number VII) and part of the lower seam (number VI) were extracted by strip mining. This surface mine, called the "Friar Tuck Mine", had a coal preparation plant located on the west side of the tributary valley. Some of the lower coal seam (number VI) and a third seam (number V) below it were extracted by the room and pillar method. This underground mine, called the "New Hope Mine", had two entrance slopes and a coal preparation plant located on the east side of the tributary valley. A coal powered electric generating station, called the "Antioch Power Plant," was built north of these two coal preparation plants on the west side of the tributary valley near Mud Creek.

PRESENT LAND CONDITIONS

The New Hope Plant, located on the east side of the Mud Creek tributary, operated from 1933 to 1947, according to mine maps on file at the Indiana Bureau of Mines. Aerial photographs indicate that the tipple and associated support buildings were "demolished" in 1953; and that the area was not substantially disturbed during a reclamation effort in 1971. Therefore structural remains visible in the field today have been mostly undisturbed for about 36 years, which has allowed enough time for fairly large trees to grow up within them.

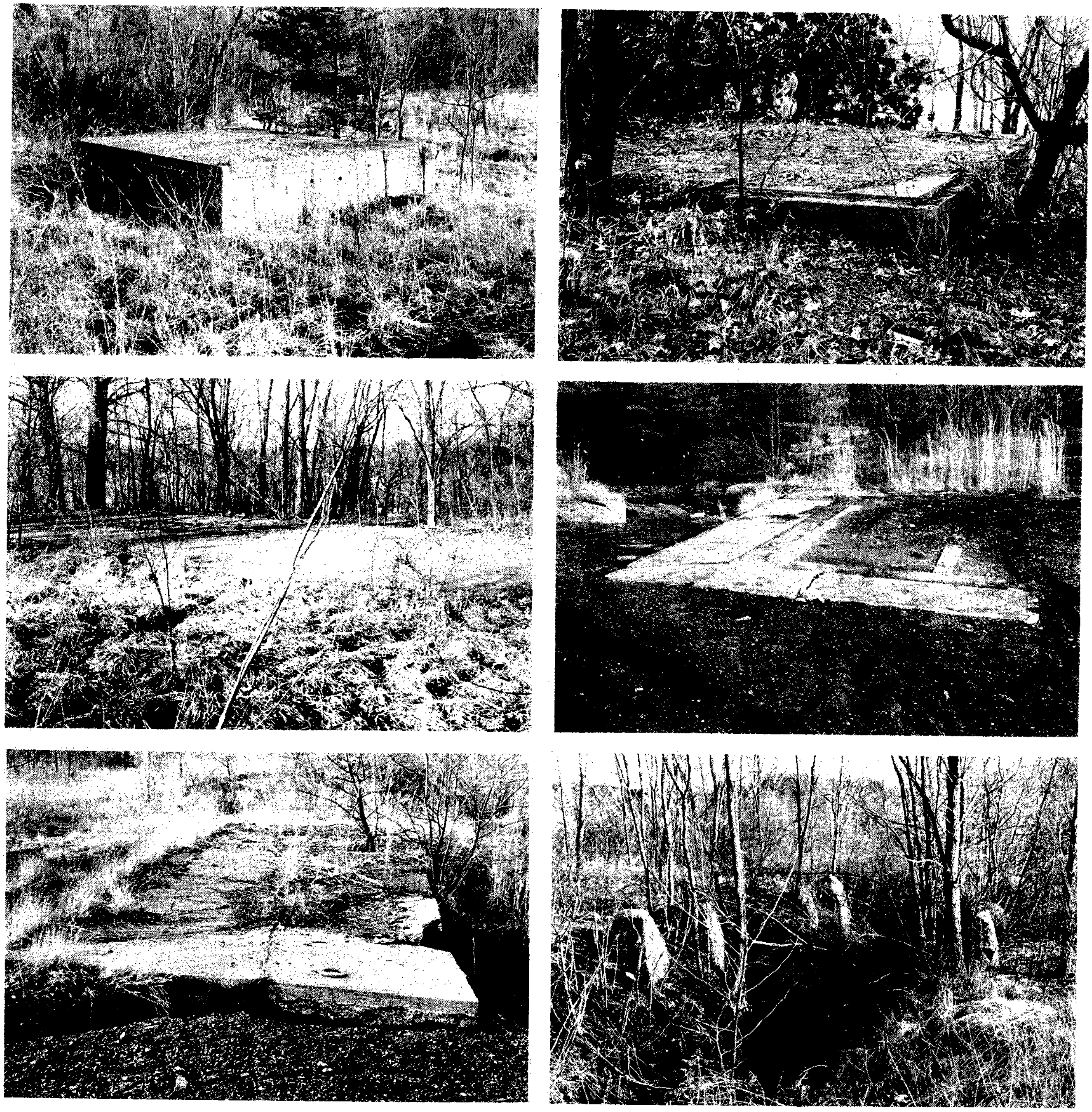
In 1953, the concept of demolition work did not include a thorough cleanup of the area. The first concern was salvaging equipment, scrap metal, and building materials. This was usually followed by burning or pushing down the remaining wood framing. Therefore, there are now several types of structural remains at the New Hope Plant, including: 1) concrete foundation walls and portions of brick building walls which project up to 12 feet above the existing ground surface, 2) concrete and brick rubble piles, 3) concrete retaining walls which create up to 4 feet high changes in ground elevation, 4) two sloping entries into the underground mine system, which have been mostly covered with earth and rubble, except for a 10 feet deep crevice up to 1 foot wide, 5) a concrete lined pit, which holds water up to 3 feet deep, and 6) concrete floor slabs, concrete pipes, and wood railroad ties, which rest on the existing grade or are buried below it (see figure 1).

The Friar Tuck Plant, located on the west side of the Mud Creek tributary, probably operated from 1929-1965, according to the recollection of old miners. Aerial photographs indicate that buildings were both added and deleted from this plant during its period of operation, and that most of them were demolished in 1967. The procedure for demolition work apparently had not changed much in the 14 years since the New Hope Plant was demolished. Again the main concern was salvaging equipment, scrap metal, and building materials. At the present time structural remains include: 1) concrete slabs resting on foundation walls up to 4 feet above surrounding grade, 2) concrete slabs resting on the existing grade, 3) concrete slabs undermined by gully erosion, creating vertical drops of up to 4 feet below surrounding grade, 4) isolated concrete piers projecting up to 2 feet above surrounding grade, and 5) vitrified clay pipes and wood railroad ties which rest on the existing grade or are buried below it (see figure 2).

The few remaining buildings at the Friar Tuck Plant were demolished during a reclamation effort in 1971; and some concrete slabs and foundations were buried at that time during massive earth work intended to cover acid producing coal preparation waste with neutral spoil material. Therefore, some structural remains are not now visible at the ground surface, but they could be exposed as soil erosion continues to remove the cover material.

The Antioch Power Plant, which is located north of the Friar Tuck Plant, operated from 1936 to 1953, according to both aerial photographs and the recollection of old miners. The garages in the power plant area were demolished in 1967, and the crane was probably dismantled for scrap iron at that time. However, the power plant area was otherwise left undisturbed, except for an "accidental" explosion one evening in 1971 that felled the smoke stack. A large part of the smoke stack rubble was covered with about 1 foot of cinder fill during the reclamation work which was then in progress.

The power plant was recently plundered for brick and scrap iron in 1987-88, and the coal silo was also felled in 1988. This recent work was more like malicious mischief than demolition, because it left the power plant area in a more dangerous condition than any other part of the coal mining and preparation facility. The structural remains there at the present time include: 1) the power plant foundation wall which projects 6 feet above and 7 feet below surrounding grade, creating a 13 foot vertical drop with dangling pieces of concrete floor slab, 2) an 8 feet deep underground room with a 2 feet square open access hole concealed by vegetation on the ground

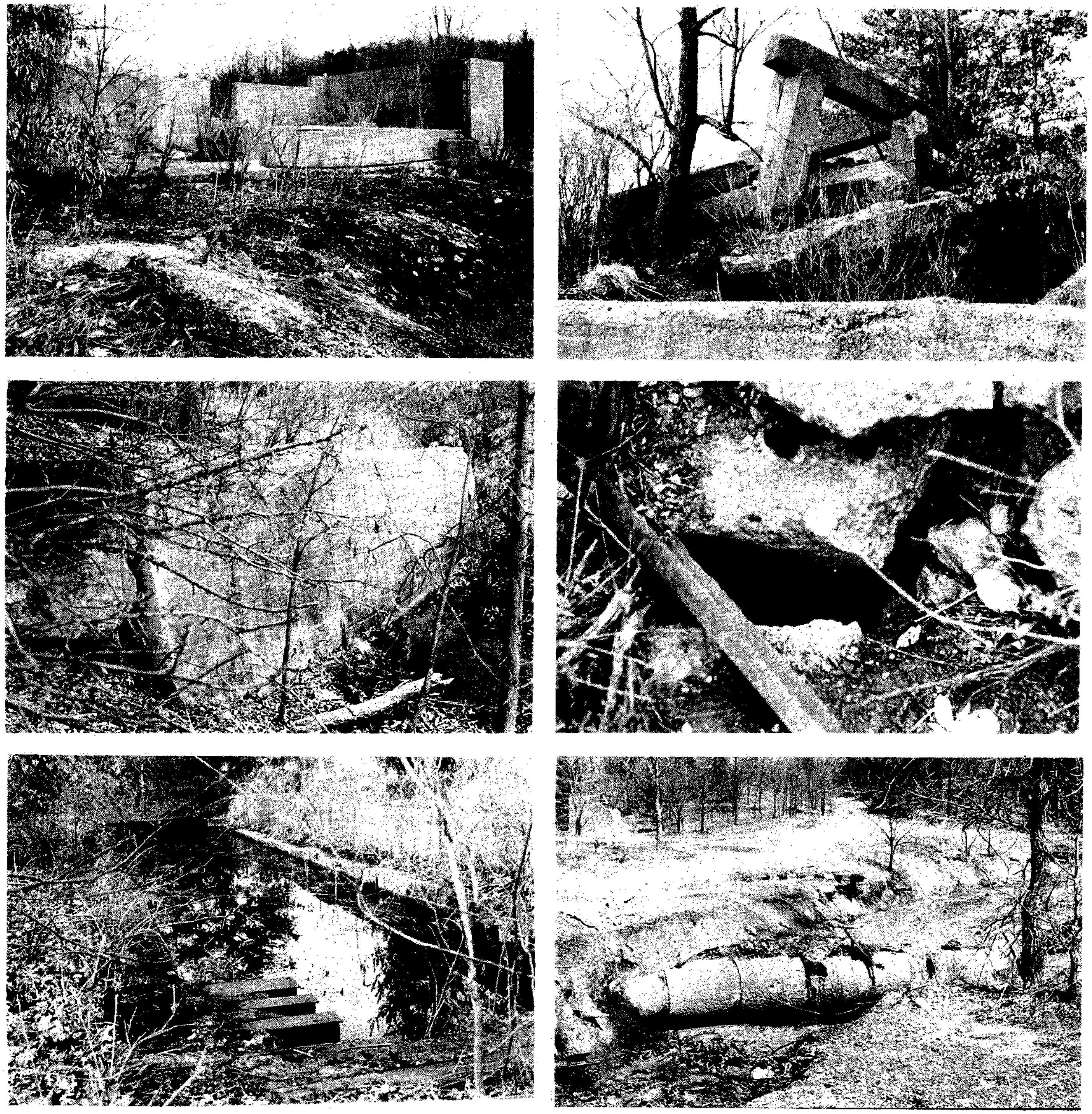


PRESENT CONDITIONS AT FRIAR TUCK PLANT

- building foundation walls
- concrete floor slab on grade
- slab undermined by erosion

- building foundation walls
- concrete floor slabs on grade
- isolated concrete piers

Figure 1



PRESENT CONDITIONS AT NEW HOPE PLANT

- building foundation walls
- pile of concrete rubble
- earth retaining wall
- crevice at covered mine entrance
- water filled pit
- buried concrete pipe and RR ties

Figure 2

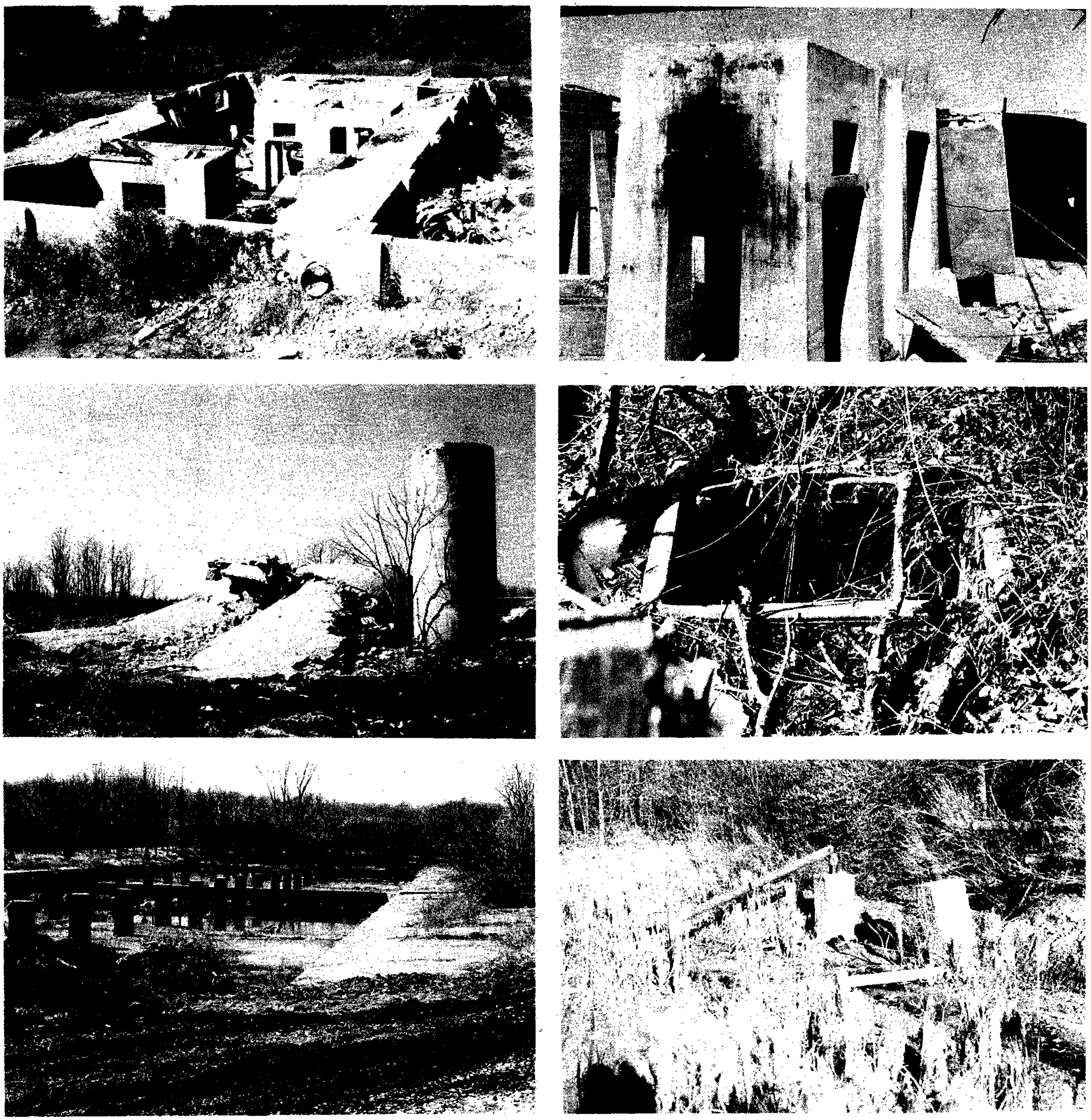
surface, 3) large fragments of the collapsed concrete smoke stack and coal silo arching up to 10 feet above the ground surface, 4) large tangles and separate protruding pieces of concrete reinforcing steel hidden by vegetation, 5) several piles of brick masonry and sheet metal rubble up to 15 feet high, 6) a concrete crane pedestal 22 feet high with an access ladder to the top, 7) a concrete lined cooling pond the size of a football field with 3 feet deep sidewalls, except for 10 feet sidewalls in the west pump pit, 8) a garage floor slab resting on a foundation wall up to 4 feet high, and 9) steel pipes and concrete footings from 20 feet high flume supports scattered in a wetland area (see figure 3).

DESIRED LAND USE

Traditionally, land use decisions have been the sole perogative of the land owner. In the past a design team could quickly proceed to develop a logical, cost effective plan to satisfy the owner's needs for using his property. However, since the 1970's environmental legislation has changed the decision making process. Now small groups of citizens are able to influence how the land will be used and they are becoming better organized to oppose land uses that they feel are not in their best interest. Considering this uncertainty about the final land use during the initial phases of investigation, it is usually prudent to collect field data in sufficient detail so that the needs of two or more land uses can be satisfied.

Although the two-square mile Friar Tuck Site is still owned by three mining companies, it is being reclaimed under the federal Abandoned Mine Lands Program. A wildlife preserve would require only minimal effort to enhance the natural rehabilitation which is already well underway. During the past 25-60 years since mining, forests have covered spoil ridges and earth work structures, such as dams and dikes; and grasses have become established on some former slurry ponds (see figure 4). The only unanswered question about a wildlife land use is what degree of thoroughness in structural demolition work at the coal mining and preparation facility will satisfy the concerns of the owners, government personnel, and citizens groups.

Some of the local citizens would prefer that the Friar Tuck Site retain its unique features which are ideal for off-road vehicle recreational activity: 1) large acreage, 2) barren ground, and 3) long steep slopes. They have enjoyed almost unobstructed use of the site for the past 25 years (see figure 5). However, this land use is not compatible with that of a wildlife preserve, because is tends to perpetuate gully erosion and stream sedimentation. Therefore, off-road vehicles will probably be prohibited here in the future. In personal interviews many of these local citizens



PRESENT CONDITIONS AT ANTIOCH POWER PLANT

- power plant basement
- dangling concrete slabs in basement
- crane pedestal and collapsed coal silo
- access hole to underground room
- water cooling pond
- support pedestals for coal slurry flume

Figure 3

WILDLIFE LAND USE AT FRIAR TUCK MINE

- natural channel of Mud Creek through unmined land
- water supply reservoir
- deciduous forest on spoil ridge
- slurry pond dike and man-made water channel
- wetland vegetation on slurry pond
- coniferous forest on spoil ridge

Figure 4

OFF ROAD VEHICLE LAND USE AT FRIAR TUCK MINE

- truck fording Mud Creek
- truck wheel ruts in revegetated area
- bike on edge of slurry deposit
- bike wheel ruts on edge of slurry deposit
- bikes disturbing revegetated area
- bike wheel ruts in drainage channel

Figure 5

have stated that they intend to continue using the site for off-road vehicles, even if it means ignoring "no trespassing" signs or vandalizing fences.

DEVELOPING DEMOLITION CRITERIA

The development of demolition criteria naturally follows the decision on what will be the desired land use. If the area is to be strip mined for a deeper seam of coal, the present degree of structure demolition would be "good enough"; but if the area is to be developed as a housing subdivision, any remaining concrete above or below grade could interfere with utility trenches, foundation excavations, or water well drilling. Therefore, reasonable demolition criteria could range anywhere from demolition of building superstructures, as has already been done at this coal mining and preparation facility, to removal of all concrete or other man-made materials from the site. Obviously, the decision on what demolition criteria to follow must be made before a contractor can make a meaningful bid on the work; but it is also helpful to know this before the site investigation is performed.

It is difficult to find guidelines for developing demolition criteria in the literature. The "Uniform Code for the Abatement of Dangerous Buildings" by the International Conference of Building Officials, which is adopted by a majority of cities in the United States, gives 18 criteria to help define what constitutes a dangerous building, but all these criteria deal with the superstructure and not the foundation. The "Abandoned Mine Lands Reclamation Control Technology Handbook" by the Office of Surface Mining gives a detailed review of factors related to demolition of the building superstructure, but it also neglects to mention the foundation and other buried structures.

The only book on this subject that we found, "A complete Guide to Demolition", implies that it is typical British practice on reconstruction projects to remove all shallow building foundations, and to backfill basements if they are not within the area of new construction activity. In the case of basements, the British also require that wood debris be removed and that the floor slab be broken up prior to backfilling with select debris, such as concrete and brick rubble. The "Standard Specifications" of the Indiana Department of Highways require removal of building foundations to a depth of 1 foot below the original ground level, and that basements be cleaned of all debris and backfilled with compacted borrow material. These specifications also require removal of concrete slabs on grade along with their base material, and removal of underground items, such as pipes, which cannot be backfilled satisfactorily. The "Final Rules Concerning the Regulation of Water Well Drilling" of the Indiana Department of Natural

Resources require that abandoned wells be plugged with clay or cement slurry from the bottom to within 2 feet of the ground surface, and that the well casing be severed at least 2 feet below the ground surface.

In the absence of definitive demolition criteria in the literature, we have developed our own criteria based on past experience at the site, the increasing concern for environmental issues in our society, and estimated cost benefit ratios. The site has been abandoned for almost 25 years, and during this time it has been heavily used as both a wildlife preserve and off-road vehicle recreation area. After interviewing several local users of the site, we heard of no hunters who were injured by falling into a pit or being stabbed by a protruding piece of concrete reinforcing steel. This statistic follows the common sense notion that hunters are physically fit and accustomed to watching where they step. Similarly, we heard of no off-road vehicle operators who were injured by the assortment of structural remains now present at the site, although we did learn about a few moderately serious injuries related to vehicle accidents which occurred while jumping ravines or climbing steep gob piles. Therefore, it is difficult to argue why additional demolition work is needed at the site to protect citizens in either a wildlife or off-road vehicle land use situation.

However, the reality of citizens groups and lawyers cannot be ignored, and their impact on defining what constitutes an eyesore or hazard in southwestern Indiana is expected to be more in the future than it has been in the past. From an aesthetic viewpoint, structural remains projecting above the ground surface do not give the impression that the area has been returned to nature. From a legal viewpoint, the site should be safe for the most awkward individual, as well as for the hunter or biker. If a loss or injury should be related to a structural remain, it is likely that the cost of defending the lawsuit could exceed the cost of cleaning up all the structural remains at the site.

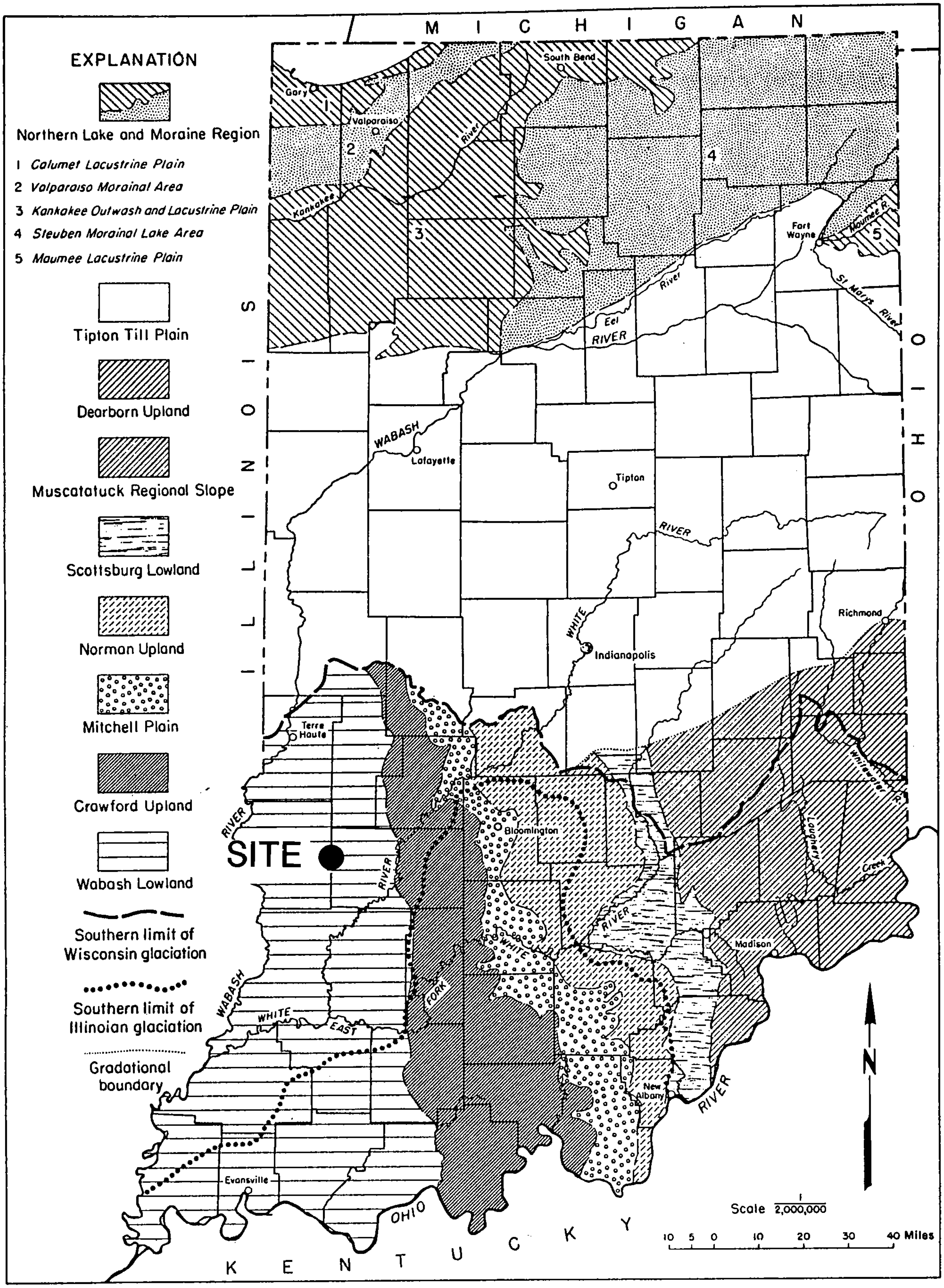
Also, reclamation work already planned (re-vegetation) requires discing or plowing to a depth of 9-12 inches. Although structural remains now buried a foot or less below the surface could damage agricultural equipment, in the long term both surface soil erosion and cycles of frost heave could cause even more deeply buried structural remains to become an equipment hazard. Since the average depth of frost penetration at the site is 2 feet deep, the frost line makes a better lower limit for removal of buried structural remains. Because most light duty building foundations extend to this depth, it would also be impractical to specify a 1 foot deep lower limit, which theoretically requires splitting the foundation along its entire perimeter at 1 foot below grade.

LITERATURE SEARCH AND PERSONAL INTERVIEWS

Every investigation for a project involving structural foundations or earthwork should begin with a review of published geologic maps, because the generalized information shown on maps provides a framework for intrepeting and organizing new information observed at the site. The Geologic Map of Indiana shows that the site is underlain by shale and sandstone deposits of Pennsylvanian age. Seven major coal seams have also been identified in these deposits, which have a northward strike and a gentle westward dip. During the Pleistocene Epoch the bedrock surface was covered with 10-30 feet of Illinoian glacial till, and a few feet of Wisconsinan loess. Soil profiles developed in these Pleistocene deposits at the site are mapped as Ava and Hickory Series. The Ava soils are located on gentle upland ridges and typically consist of 40 inches of silt loam (loess) over loam (till) with a fragipan horizon at 29-53 inch depth. The Hickory soils are located in adjacent upland draws and typically have less loess and do not have a fragipan horizon. The Physiographic Map of Indiana shows that the site is located in the Wabash Lowland (see figure 6). About 80% of this area of broad valley flats and low rolling hills is suited to general agriculture, and the remainder consists of steep slopes.

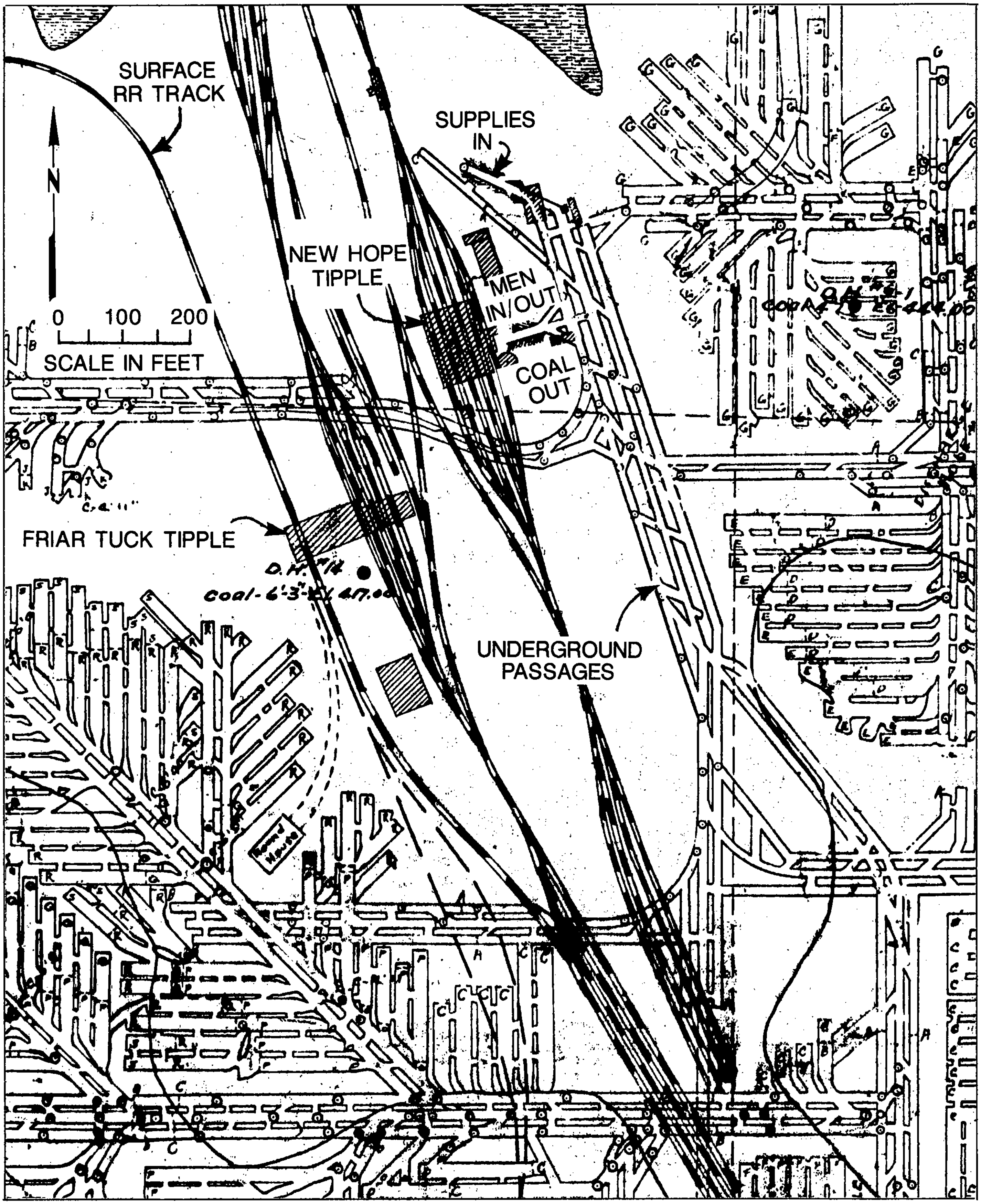
In addition to published maps prepared by state and federal agencies, unpublished maps prepared by private companies or their consultants are commonly available. These unpublished maps, which usually cover small areas in great detail, reach the public domain when they are submitted to meet the requirements of some government agency. For example, soil and geologic reports for thousands of new construction projects can be reviewed at the building inspection department in major cities and urban counties. In our case, the site is located in a rural area where this type of report is not available. However, we were able to locate an original mine map at the Indiana Bureau of Mines. Preparation of this underground mine map was required by state law for the purpose of rescuing men in the event of a mine disaster. Therefore, it shows the location of all underground passages on a large scale (1 inch = 200 feet). The map also shows all mine entrances and air vents, as well as nearby surface facilities. This information is very useful to demolition work because it allows the location and proper sealing of old mine entrances which are often difficult to find in the field. A portion of the map showing the main level (number V) of the New Hope Mine is reproduced in figure 7.

As we mentioned in the introduction, some background in engineering or building construction is necessary for the engineering geologist who is involved with planning structural demolition work. Even then this background may be limited to a certain type of structure, such as residential buildings; or it may be limited to a certain era of construction, such as modern practice. Therefore, it is desirable to extend the literature search to include a review of



PHYSIOGRAPHIC MAP OF INDIANA

Figure 6



NEW HOPE MINE MAP (NO. V SEAM)

information on the type and age of facility being demolished. In our case, we reviewed references on coal mining, coal preparation, and coal powered electric generating stations. If possible, we selected books published in the 1930's, which is the period when most of the facility was developed (see the reference list at the end of this paper). We reproduced some of the figures from these old books to show the kind of information available (see figures 8-12).

Although a literature search may be very productive, it is likely to leave some large gaps in your understanding of a particular site. At this time it is advisable to make personal interviews with any management or labor employees who worked at the facility being demolished. Armed with maps, photographs, and a general knowledge of equipment and terminology obtained in the literature seach, it is possible to fill in many details of events that occurred up to 60-70 years ago. For example, we learned that the southern gob pile (coarse shale, sandstone and pyrite refuse derived from coal preparation) of the Friar Tuck Mine apparently contains 30 barrels of unused motor oil; and that a miner was injured by a methane gas explosion, which occured while cutting a water well casing at the ground surface (the well water source was in old underground mine workings).

Finally, if you are lucky, some of the people interviewed will have historic photographs of the facility during the period of its construction or operation. These historic photographs aid in the interpretation of small scale aerial photographs, and give additional detailed information on the site. We have reproduced two of the several historic photographs which we were able to copy. Figure 13 shows the heart of the coal mining and preparation facility. Notice the old car and gasoline pump at the center of the photograph. This photograph alerted us to the possible presence of an underground gasoline storage tank, in addition to the details of building construction which we were looking for. Figure 14 shows the electric generating station with its coal handling equipment and cooling pond in the foreground. The steel framework at the left edge of the photograph is a bank of electrical transformers. This photograph indicated the possibility of PCB contamination in the soil below the transformers.

AERIAL PHOTOGRAPHS

Vertical aerial photographs, which provide an almost uniform scale across the image area, are the best single source of both physical and historical information at a site. Building size can be roughly determined by measuring its area with a simple ruler and its height with a parallax bar. In cases where the superstructure has been demolished, this will usually be the only estimate of building size available. When a site has been abandoned for several years and some

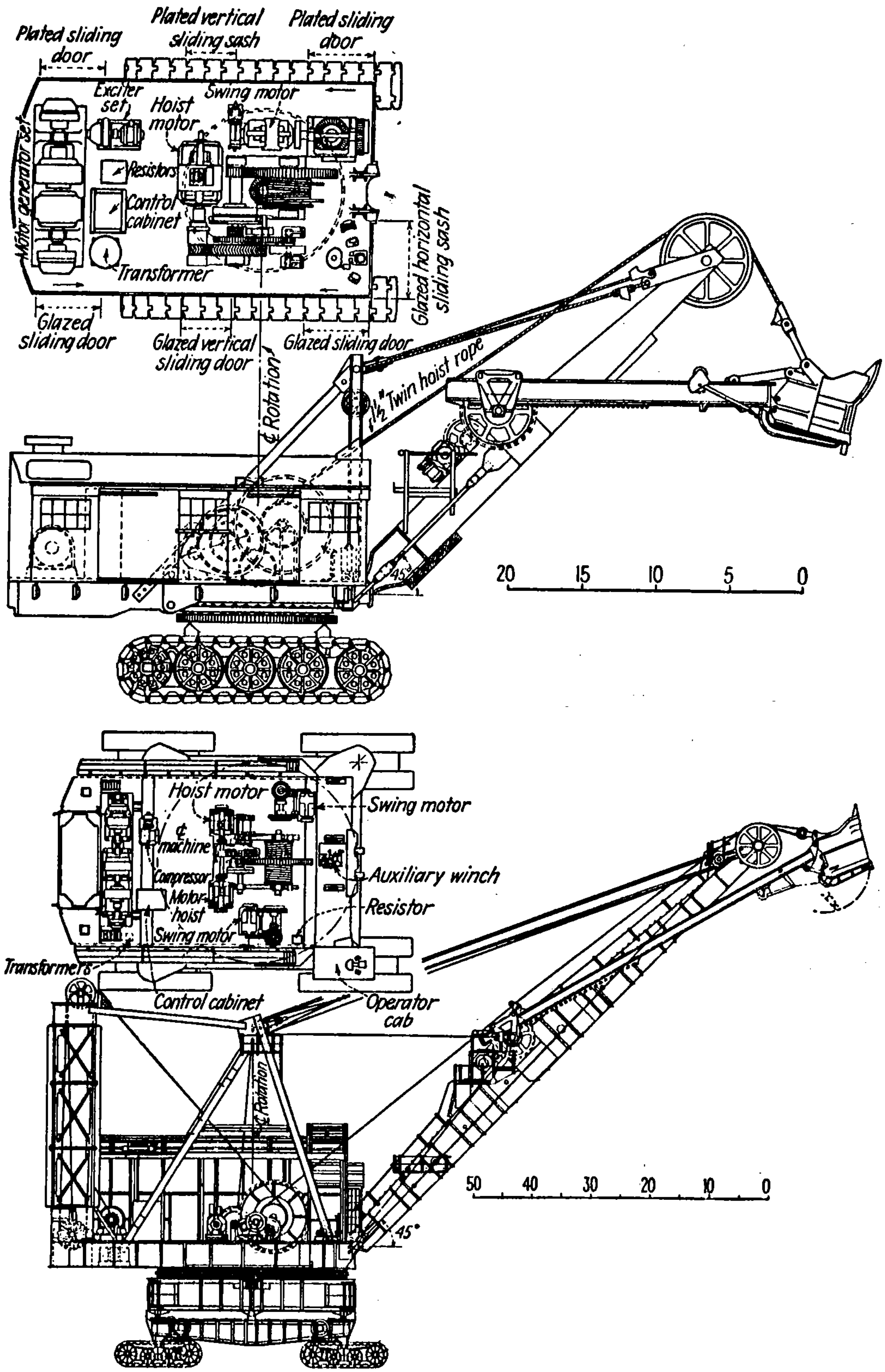


Fig. 189.—Upper figure is a 5-yd. electric shovel; lower is a stripper or long-boom shovel. (*Bucyrus-Erie Co.*)

TYPICAL SURFACE MINING EQUIPMENT IN 1930's (Similar to that originally used at Friar Tuck Mine)

Figure 8

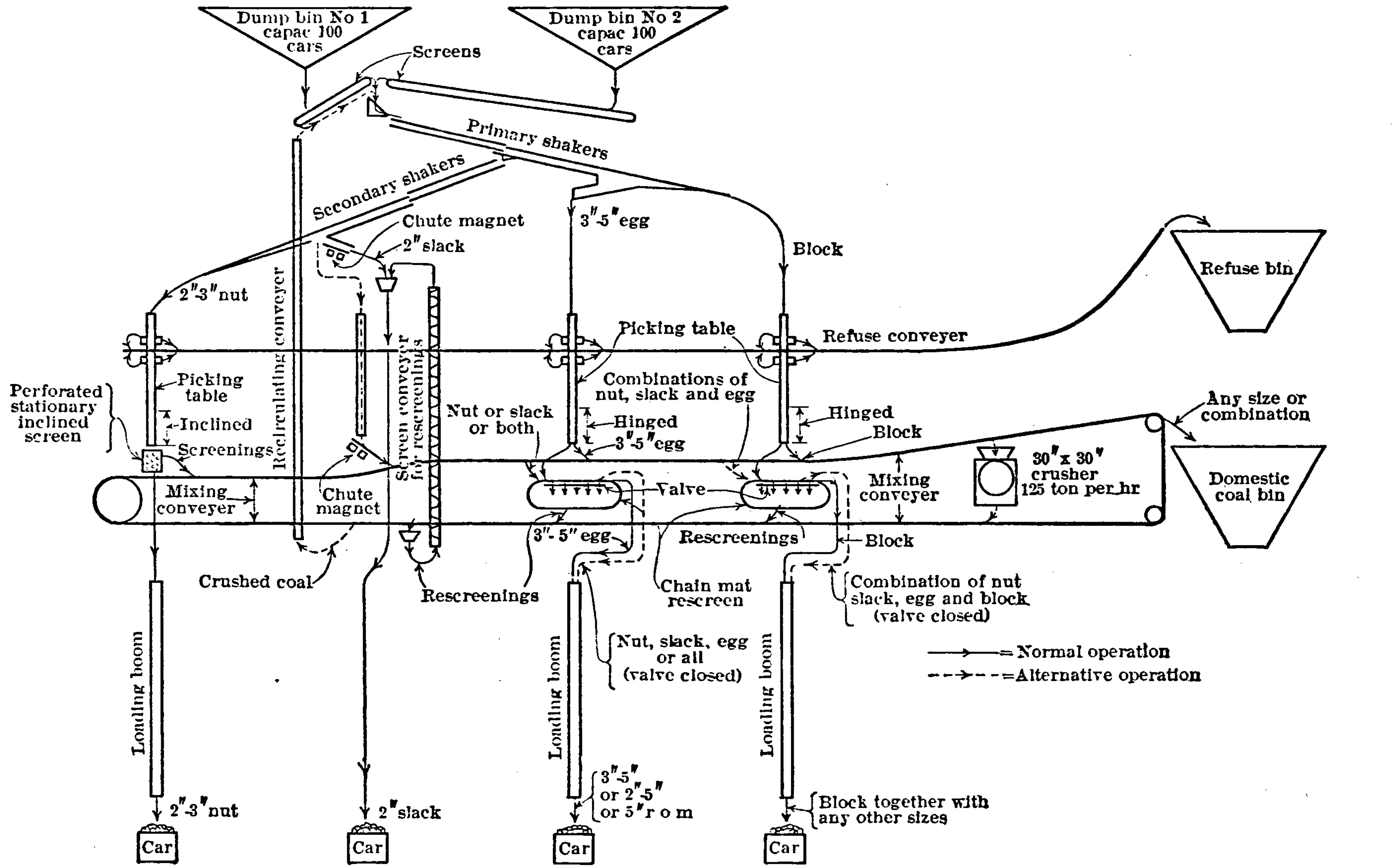
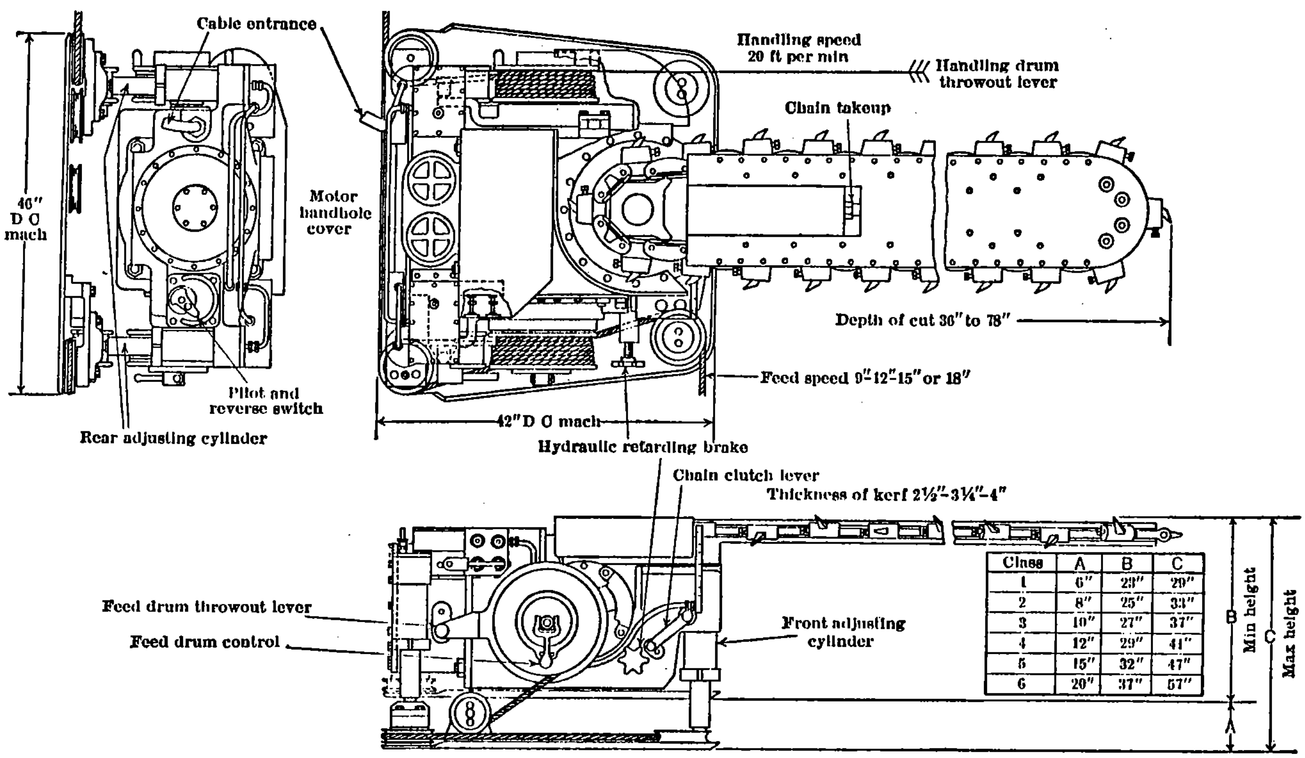


Fig 17. Tipple with Hand Picking (*Coal Age*, Jan, 1937)

TYPICAL BASIC COAL PREPARATION PLANT IN 1930's
(Similar to the original Friar Tuck Plant)

Figure 9



Class	A	B	C
1	6"	23"	29"
2	8"	25"	33"
3	10"	27"	37"
4	12"	29"	41"
5	15"	32"	47"
6	20"	37"	57"

Fig 12. Sullivan Floor-type, Turret, or Top Cutter (9B)

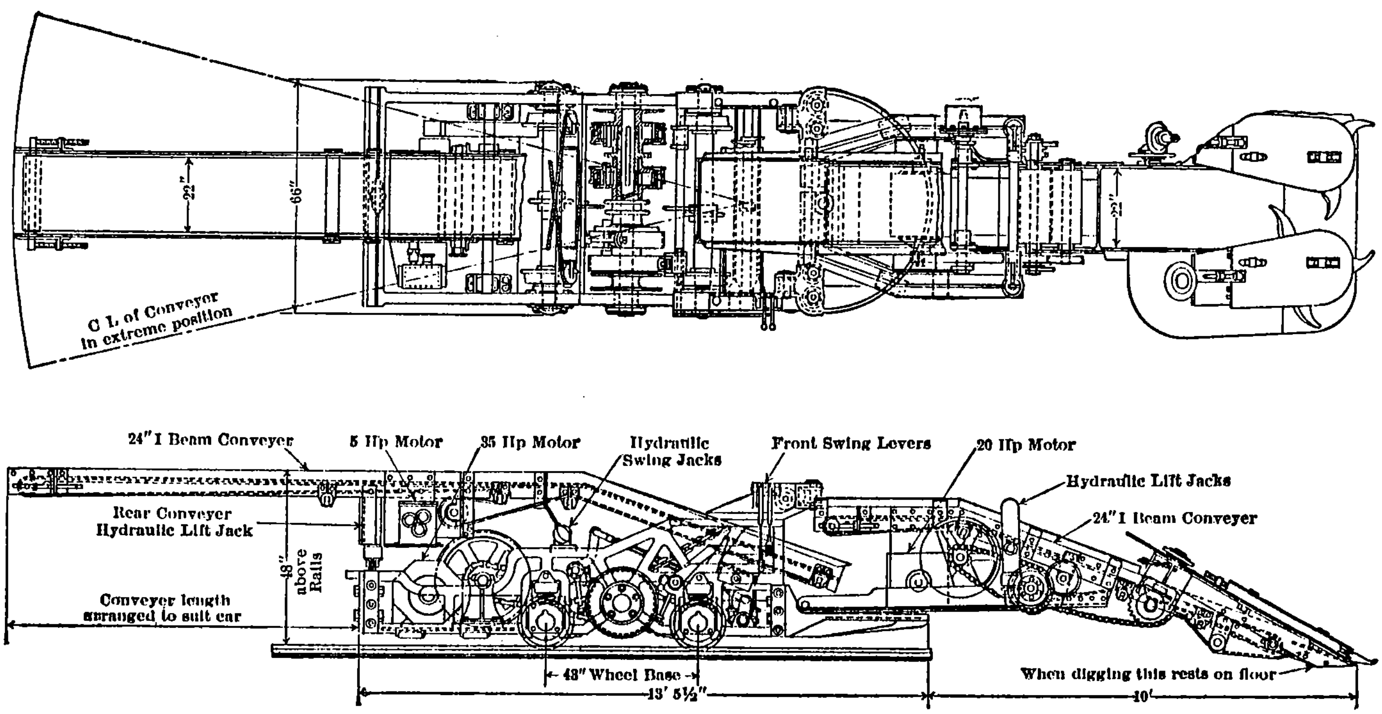


Fig 5. Clarkson Loader

TYPICAL UNDERGROUND MINING EQUIPMENT IN 1930's
(Similar to that used at New Hope Mine)

Figure 10

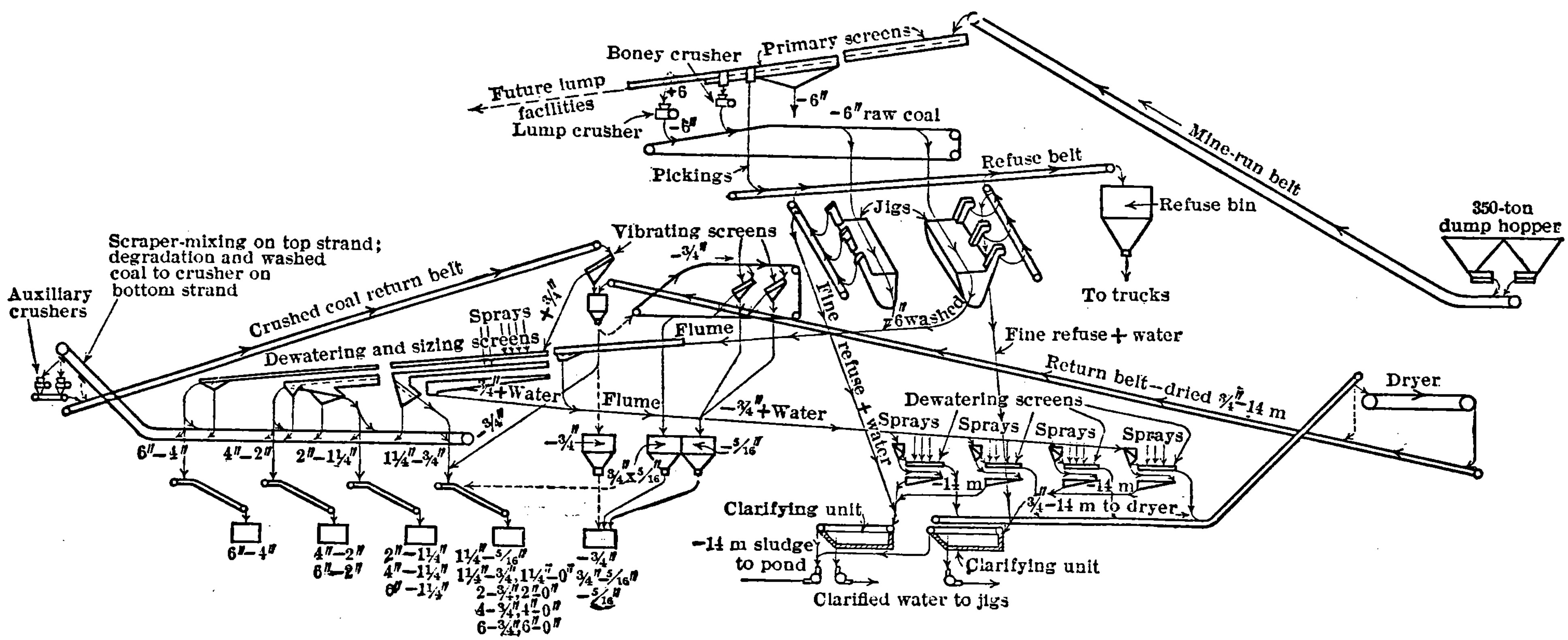
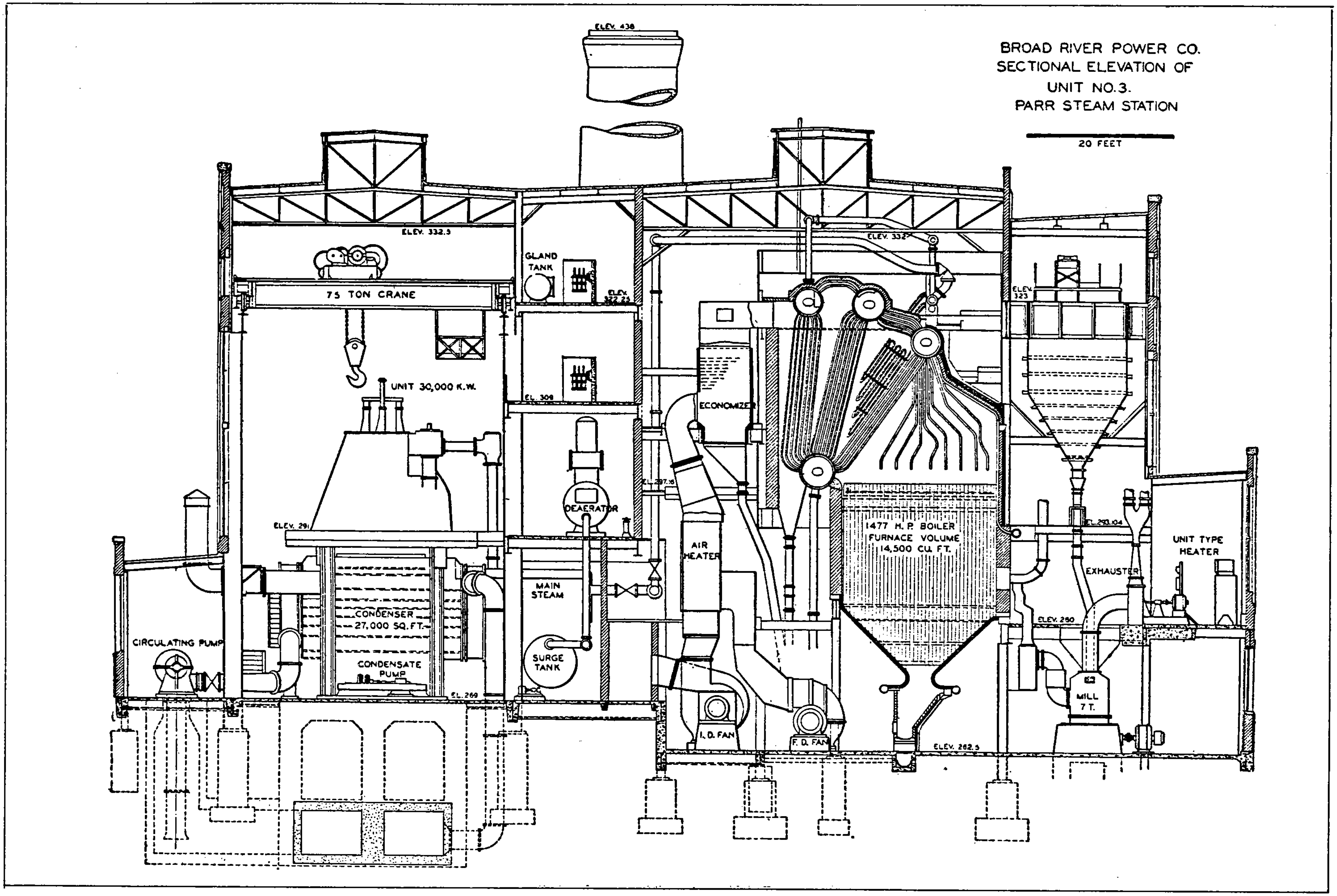


Fig 18. Baum Jig Plant

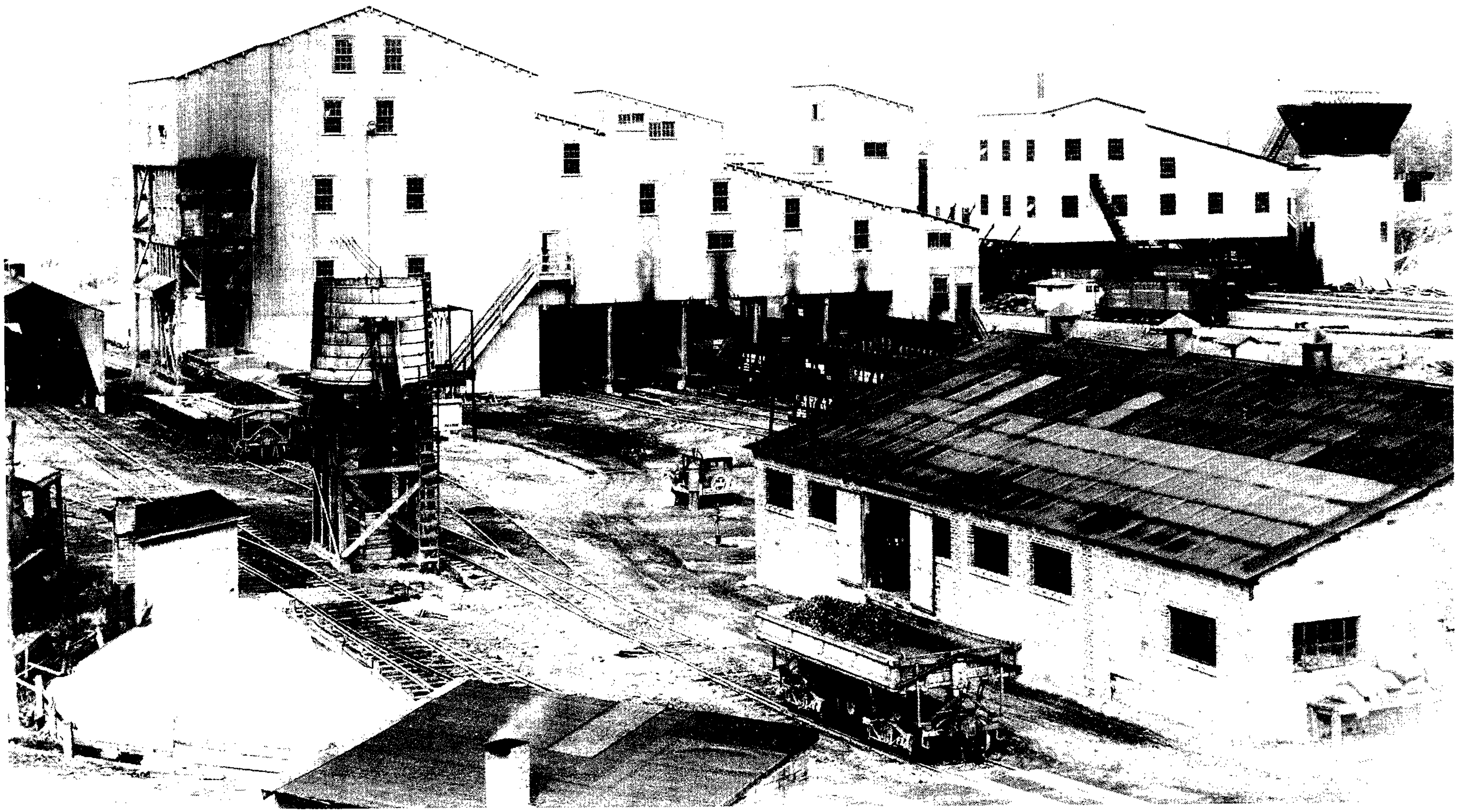
TYPICAL ADVANCED COAL PREPARATION PLANT IN 1930's
(Similar to the New Hope Plant)

Figure 11



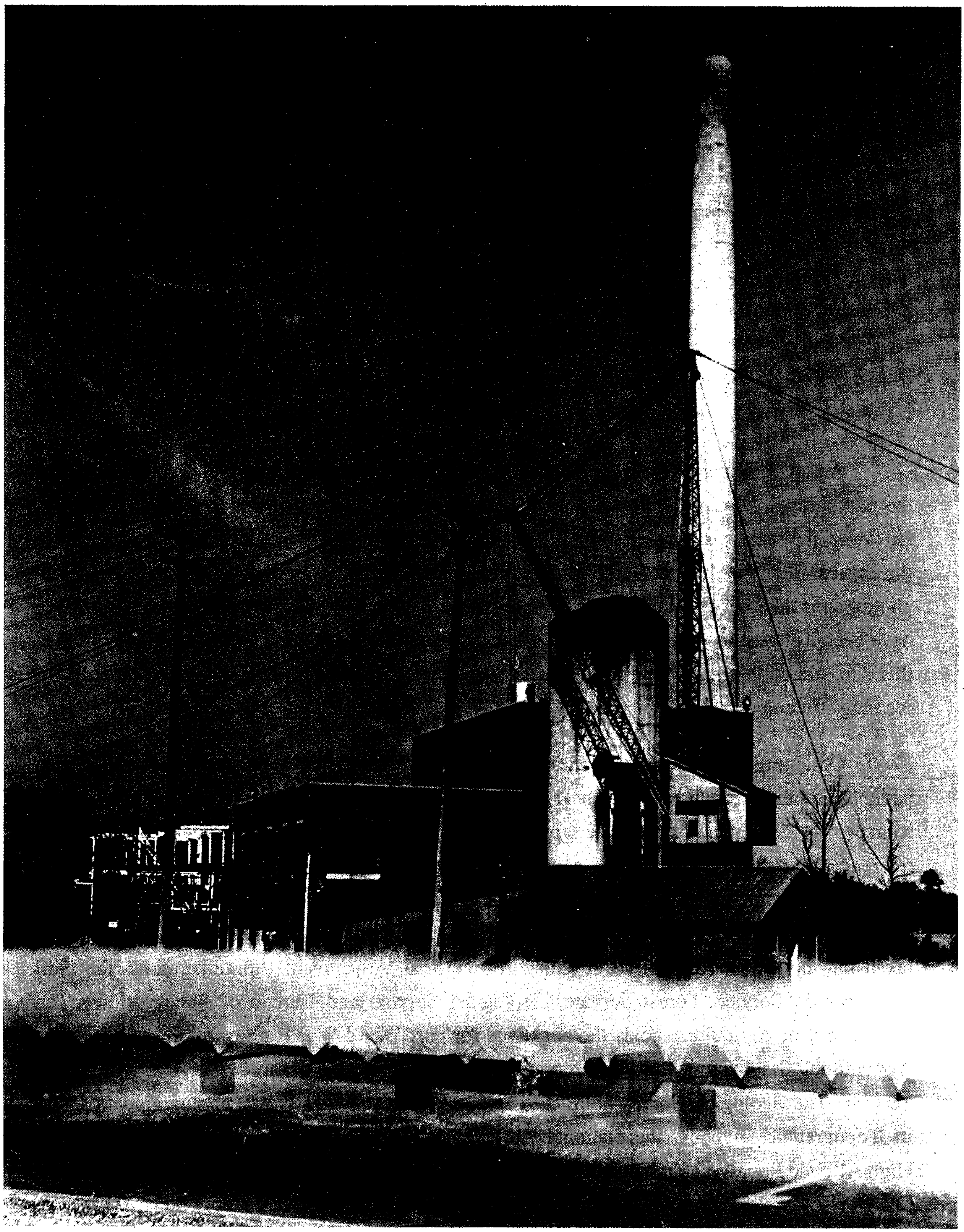
TYPICAL STEAM POWERED ELECTRIC GENERATING STATION IN 1930's
(Similar to Antioch Power Plant)

Figure 12



HISTORIC PHOTOGRAPH OF COAL MINING AND PREPARATION FACILITY IN 1930's
(Friar Tuck Plant in foreground and New Hope Plant in background)

Figure 13



HISTORIC PHOTOGRAPH OF ANTIOCH POWER PLANT
(Cooling pond is operating in foreground)

Figure 14

land grading has been done, as at this coal mining and preparation facility, measurements made on the photographs can be used to locate the structural remains of "missing buildings". Interpretation of building size and its relationship to other features shown in the photographs, allows a conjecture about its use. Since all aerial photographs are marked with the date (sometimes even to the minute) that they were taken, a complete collection of them can be used to establish approximate dates of building construction and abandonment. Knowing the building age and use gives an experienced person a good idea of its construction features and alerts him to possible demolition difficulties and environmental hazards. For example, the possibility of PCB contamination of the soil beneath the electrical transformers at the Antioch Power Plant noted from viewing the historic photograph in the previous section, was also independently noted from interpreting the aerial photographs.

Aerial photographs have long been used the prepare accurate topographic maps. Many of the photographs now available from government agencies were taken for exactly this purpose. However, the scale of the photograph is usually too small to provide useful topographic maps for the typical demolition site. Therefore, it is preferable to have special, large scale aerial photographs taken of the site vicinity, and to use these for preparation of a topographic map. We have been using a topographic map with a 1 inch = 200 feet scale and a 2 foot contour interval at this site, although sometimes it is more convenient to use the aerial photographs directly for locating structural remains in the field.

Obtaining a complete collection of aerial photographs for a site requires considerable patience, so the effort should begin as soon as possible. In most cases there are three major sources: the National Archives, the Department of Agriculture, and the Geological Survey (see reference list for addresses). Other sources for aerial photographs in limited areas include state highway departments and a few commercial aerial photographic contractors who maintain film libraries. The National Archives holds the negatives for the federal government photography taken in the period 1934-1942 by the Agricultural Adjustment Administration, the Soil Conservation Service, Forest Service, Geological Survey, and Bureau of Reclamation. The Department of Agriculture holds the negatives for post World War II photography taken by the Agricultural Stabilization and Conservation Service, Soil Conservation Service, and Forest Service. The Geological Survey holds the negatives for post World War II photography taken by its Topographic Division, the National Aeronautical and Space Administration, the National High Altitude Photography Program, and the National Aerial Photography Program.

At the time of your initial inquiry these federal government sources will encourage you to purchase a photo index so that you can identify the frames which cover the site, but they will occasionally do this search themselves, if given a topographic reference map showing the site. Although each of these sources offers a variety of photographic services from the negatives they hold, we have learned from several frustrating experiences that the large scale prints needed for demolition planning are best obtained from local photographic labs working directly with negatives. Therefore, we ordered only copy negatives from the government sources. Because of the small scale of government photography, the 9 x 9 inch copy negatives received can be easily trimmed to 4 x 5 inch size for use in locally available photographic equipment. We affixed yellow "Post-it-Notes" to the trimmed negatives for cropping them to cover only the site. The photographic lab then enlarged the cropped images from 8x to 30x to produce a set of prints on 8 1/2 x 11 inch paper at a uniform scale of about 1 inch = 225 feet. One of these aerial photograph prints, trimmed to 6 1/2 x 9 inch size, is shown in figure 15.

A list of the aerial photographic coverage which we obtained for our site is given in table 1. Notice that the list contains two photographs taken the same year, and even two photographs taken the same day. For several reasons we obtained every possible photograph of the site, including stereo pairs: 1) camera and film quality varies on each flight, 2) radial distortion varies with distance from the center of photo, 3) sun angle and atmospheric haze vary between flight lines or different flights, 4) sun reflection from water surfaces varies between frames and flight lines, 5) leaf season on deciduous trees affect interpretation in wooded areas, 6) ground moisture and season both affect interpretation in grassland areas, and 7) some site conditions change rapidly, such as vehicle locations and smoke stack emissions. Also notice that the list contains few photographs at scales smaller than 1:40,000. From our experience photographs at scales smaller than that have limited usefulness when studying a demolition site.

FIELD MEASUREMENTS

The first task is to locate and identify in the field the structural remains of buildings which are shown on maps or aerial photographs. In some cases this may be so obvious that it does not seem like a task at all. However, when an abandoned facility, like that on the site, has been mostly demolished and partially regraded with bulldozers, it takes a substantial amount of field work. Several reference points visible in the field must be correlated with points visible on maps or photographs. Buildings with presently visible basements or foundation walls make excellent reference points to locate the less obvious structural remains of other buildings. Road intersections, railroad crossings and hedge rows also make good reference points. Bends in surface water drainage channels and isolated trees are sometimes useful reference points, but

1949 AERIAL PHOTOGRAPH OF COAL MINING AND PREPARATION FACILITY

Figure 15

Table 1

AERIAL PHOTOGRAPHIC COVERAGE

OF FRIAR TUCK SITE

Date	Photo No.	Source	Film Type	Scale
6/28/37	QO-QP 4-340,341	AAA	B & W	1:20,000
6/28/37	QP-QO 4-304,305	AAA	B &W	1:20,000
5/31/49	QO-IF-177,178	ASCS	B & W	1:20,000
9/20/49	QP-3F-199,200,201,202	ASCS	B & W	1:20,000
9/25/52	VVAJM4-512,513	USGS	B & W	1:69,000
6/28/54	QO-1N-17,18	ASCS	B & w	1:20,000
6/6/58	QO-1V-38,39	ASCS	B & W	1:20,000
6/7/58	QP-IV-12,13,14	ASCS	B & W	1:20,000
4/16/62	VAJP2-48,49,50	USGS	B & W	1:24,000
7/20/66	QO-2GG-13,14, 15	ASCS	B & W	1:20,000
9/25/67	QP-1HH-283,284	ASCS	B & W	1:20,000
9/22/71	JSC-027,028	NASA	Color IR	1:63,000
8/10/72	02-0136	NASA	Color	1:20,000
9/30/74	18153-174-7,8,9	ASCS	B & W	1:40,000
4/21/75	18055-175-100,101	SCS	B & W	1:38,000
5/9/77	VEHQ2-94,95,96	USGS	B & W	1:80,000
10/30/80	18055-178-207,208	ASCS	B & W	1:40,000
5/15/84	NHAP132-51,52	NHAP	B & W	1:80,000
3/22/86	NHAP166-15,16	NHAP	B & W	1:80,000
3/22/86	NHAP247-22,23	NHAP	Color IR	1:58,000
8/19/87	NAPP229-29,30	NAPP	Color IR	1:40,000

their correlation with images in old photographs should be verified by tracing their appearance and location back through time by using a complete collection of aerial photographs. After a network of reference points is established, a measuring tape and compass can be used to find missing items in the field which are visible on old aerial photographs and maps.

For each structural remain located, it is necessary to make accurate measurements of the size and thickness of its component parts, such as foundation walls, floor slabs, and footings. The measurements previously obtained from aerial photographs commonly are not very accurate, especially when the footing of a building does not correspond to the size of its superstructure or roof, which is the only part visible in the photograph. Also, no aerial photograph measurement can give the thickness of structural members, such as foundation walls or slabs; yet this information is useful in estimating demolition costs.

When the structural remains are fully exposed, measuring tapes can be used to obtain the necessary dimensions. We found that a 200 foot fiberglass tape was ideal for measuring most of the larger dimensions. Its light weight was also an advantage when making measurements with the tape suspended in the air. For short dimensions we used a 25 foot steel tape with a 1 inch wide blade. The wide blade is an advantage when the loose end of the tape needs to be cantilevered in the air, pushed through debris, or pushed upward along high walls. Finally, the compass bearing of a building wall was obtained to confirm that the structural remain is oriented the same way as its assumed image in the aerial photograph (a simple method to check the accuracy of previous location and identification work). Also, a north arrow on the final scale drawing of the building helps to clarify later discussions when somebody in the field uses terminology like "the east wall of the building".

Measuring structural remains becomes more difficult when some of the members are buried beneath the ground surface. We made a 5 foot long probe rod with a welded "T" handle from 5/8 inch diameter steel bar. This was useful for probing the limits of partially buried concrete floor slabs. Foundation wall depths and footing sizes were determined by excavating 2 x 4 foot test pits up to 5 feet deep with a common round point shovel and a digging bar (available at some hardware stores if you are persistent). Based on our experience, a 2 x 4 foot test pit is the smallest size which a man can work in to excavate below 3 feet deep and to examine footing details. In the past, we have excavated a 2 x 4 foot test pit 13 feet deep to reach the bottom of a drilled pier footing, so the only limit to pit depth is your perseverance. After excavating the test pit and measuring the dimensions of the building substructure, we also logged the soil conditions exposed. Although soil conditions are not shown on the final scale drawings, this information is used for other phases of reclamation planning. For example, is the building located on a filled (mined) area, on the natural ground surface, or in an excavated area?

In some cases, one member of the structural remain conceals an important dimension of another member, and no reasonable amount of earth excavation can obtain this information. For example, a concrete floor slab commonly covers the top of the foundation wall, concealing its width. This makes it necessary to penetrate one of the structural members to get the missing dimension. Based on our past experience an electric hammer drill large enough to operate a 3/4 inch drill bit is a very satisfactory tool for probing the thickness of concrete or masonry members. Larger diameter drills have a greater chance of hitting steel reinforcing bars that stop the drilling, and smaller diameter drills are too slender to be available in the longer lengths sometimes required (3/4 inch diameter drills are available 18-24 inches long). Diamond coring drills, which are commonly used to drill rock, can also be used to probe the thickness of concrete or masonry members. However, diamond core bits are expensive, require a source of cooling water, and may be ruined it they contact steel reinforcement in concrete. A common electric drill with a 3/8 inch drill bit capacity is adequate to probe the thickness of steel pipes or structural members, and wood members.

Although the soundness of a concrete or masonry structural member can usually be determined by a hammer blow on its surface (similar to Terzaghi's method of testing rock soundness), complete penetration of the member is necessary to determine its reinforcement. This penetration must be considerably larger than a drilled hole, because steel reinforcement is commonly spaced 18-24 inches on center, or more. Electric demolition hammers are convenient to use, but they are painfully slow (including the 90 pound "Brutes"). Air operated demolition hammers have long been the standard tool in the construction industry, but they require trailer mounted 85-125 CFM air compressors to operate them. At this site we used a 12 pound sledge hammer to penetrate heavily reinforced slabs up to 6 inches thick. Although it may seem that swinging a 12 pound sledge is a lot of work, a lighter sledge hammer simply does not get the job done. A magnetic metal detector can usually determine the presence of steel reinforcement in concrete structural members. However, there is commonly a lot of scrap metal around an abandoned site, which could give a misleading reading on the instrument. Also, based on our experience, it is difficult for these instruments to determine the number and spacing of reinforcing bars; and it is impossible for them to determine the bar diameter.

OFFICE CALCULATONS AND RECOMMENDATIONS

The dimensions of structural remains measured in the field were used to make scale drawings, which are commonly called "as built" drawings in the construction industry. These dimensions were also used to calculate the surface area of walls and slabs, the lineal feet of footings, and the volume of in-place concrete. The calculated values for each member of the structural remain were included on the drawing. Whenever it was possible to determine the steel reinforcement in the concrete, this information was also included on the drawing. However, the soundness of the concrete was not included because its determination was subjective, and this value changed from one area to another in the same structure. We feel that it would be more appropriate for bidders to make their own evaluation of this factor during a site visit. The soil conditions were not included because we feel that they would not influence the demolition process to any significant extent. The cooling pond structure, shown in figures 16-21, is a typical example of the scale drawings made for contractors to bid on demolition work at the site.

Based on the demolition criteria found in the literature and our evaluation of possible land uses for this coal mine facility, we recommend that:

1) Structural remains projecting above existing grade, on grade, or within 2 feet below grade shall be removed.

2) Concrete slabs located deeper than 2 feet below existing grade shall be thoroughly broken up in-place.

3) Basements, pits, or other closed depressions below existing grade shall be cleaned of organic debris and backfilled with borrow material (soil) or select debris (concrete, brick, or stone rubble).

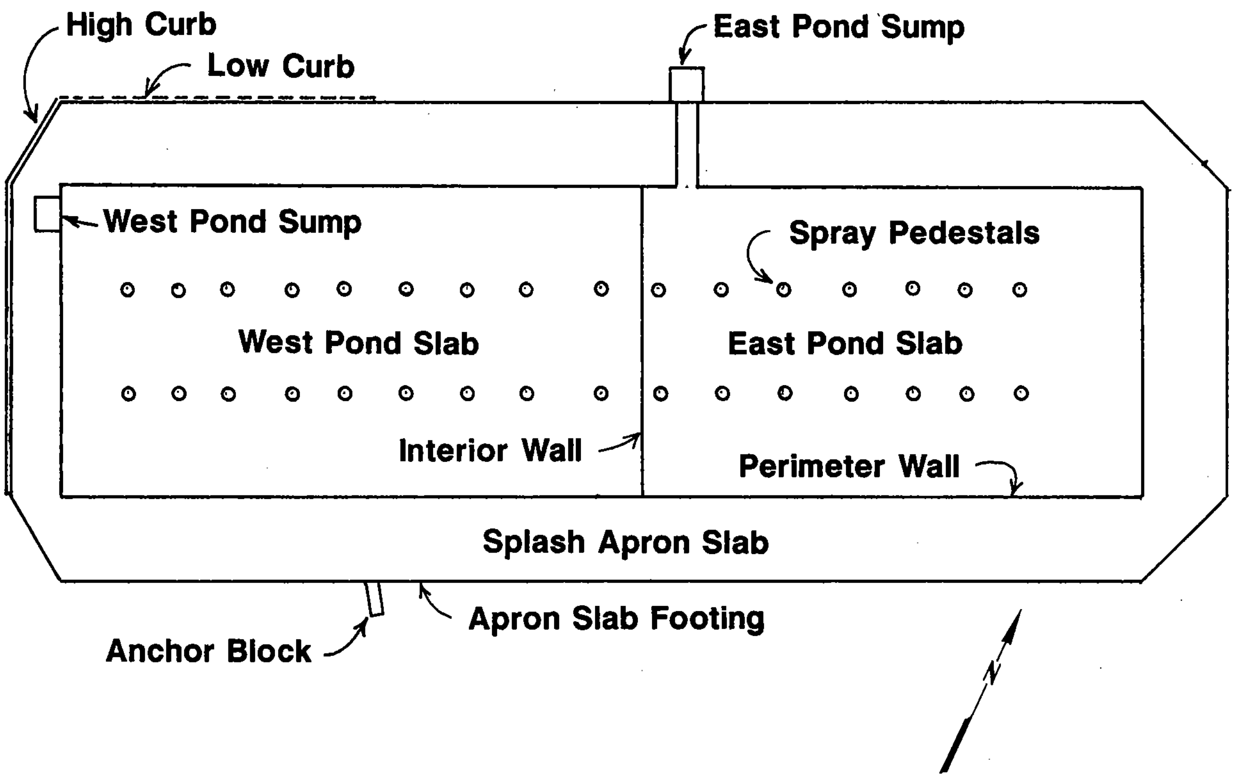
4) Select debris shall be covered with at least 2 feet of borrow material and sloped to drain surface water without excessive erosion.

5) Underground mine entrances (slopes) shall be sealed with a concrete or masonry bulkhead, and then backfilled to match surrounding grade.

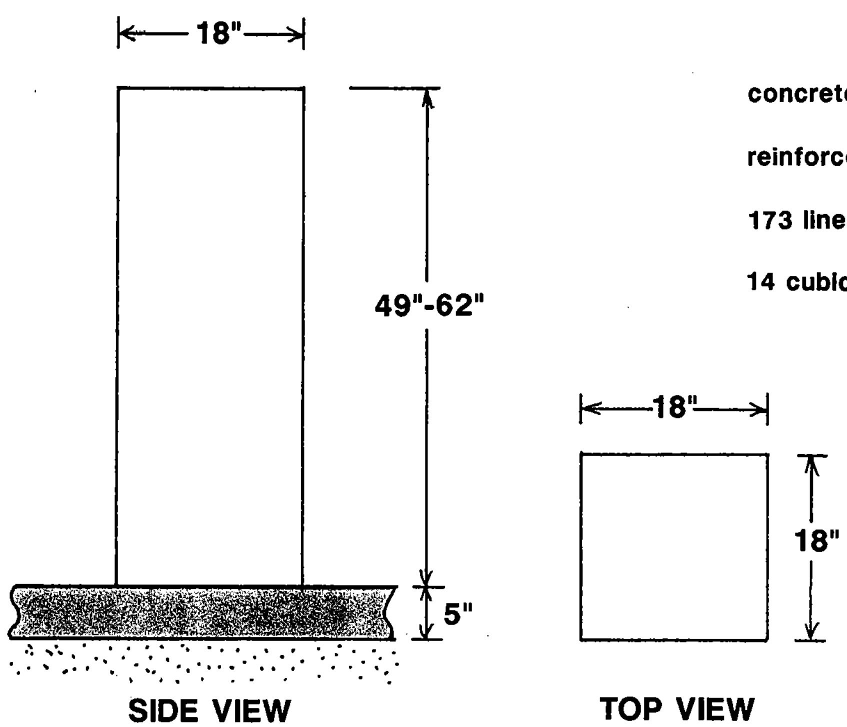
6) Culvert pipes and other pipes greater than 8 inches in diameter shall be removed.

7) Water wells shall be cleaned of soil or debris for a depth of 25 feet and refilled with clay or cement slurry; and the well casing shall be severed 2 feet below the ground surface and covered with a concrete plug.

COOLING POND PLAN



SPRAY PEDESTAL DETAIL



concrete: 18" square

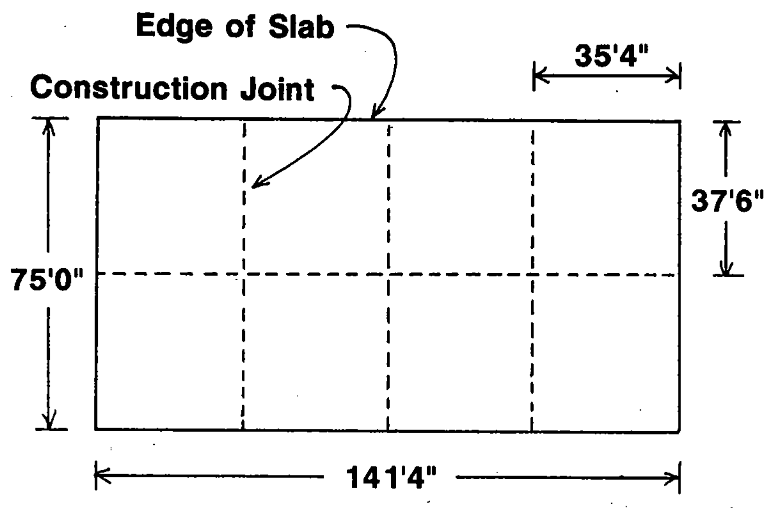
reinforcement: unknown

173 lineal feet

14 cubic yards

Figure 16

WEST POND SLAB DETAIL



concrete: 5" thick

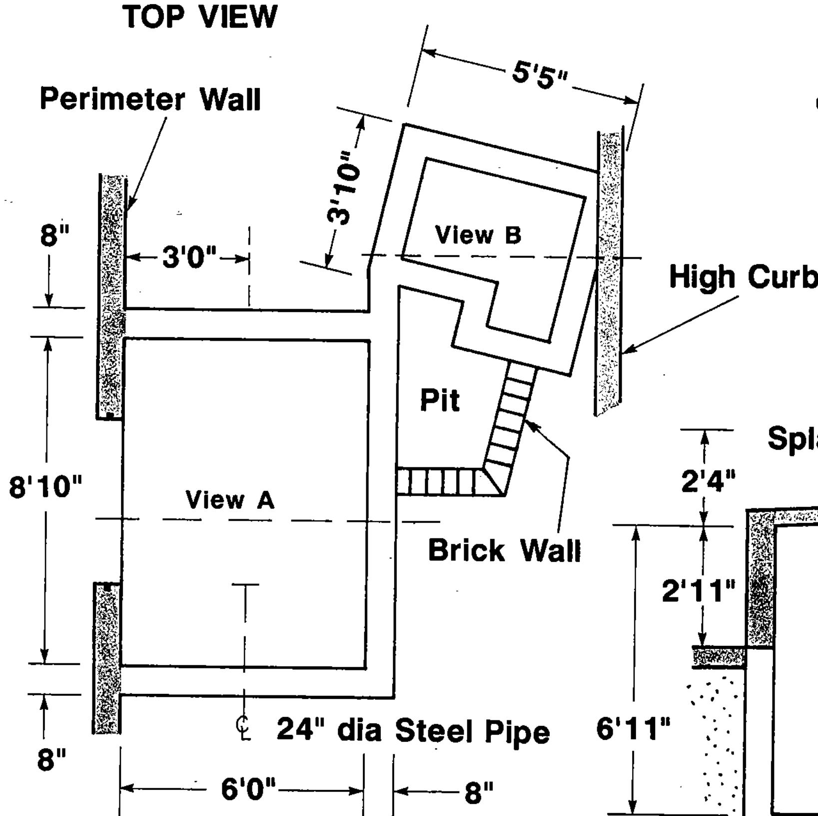
reinforcement: 10x10 ga 6x6" WWF

10,600 square feet

164 cubic yards

WEST POND SUMP DETAIL

TOP VIEW



concrete: 8" thick
reinforcement: unknown
341 square feet
8 cubic yards

SIDE VIEW

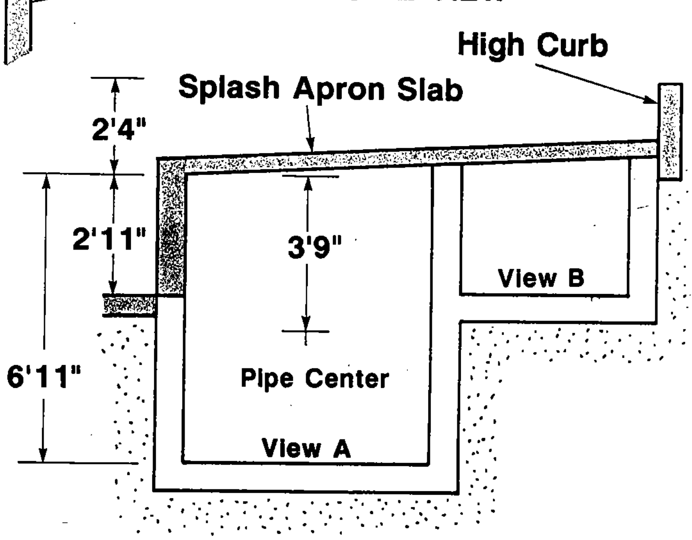
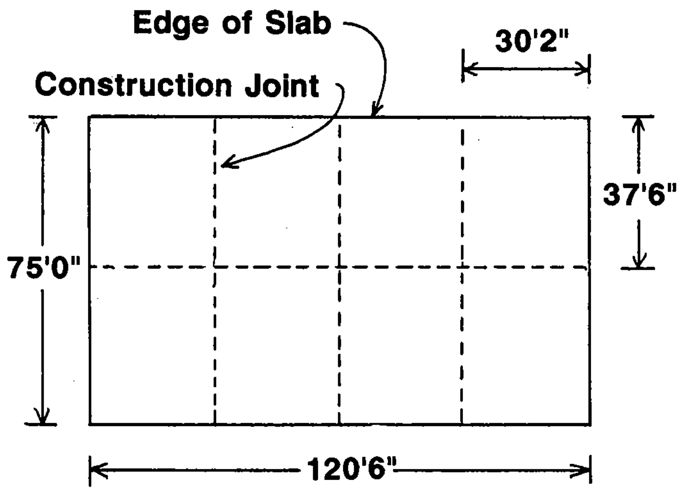


Figure 17

EAST POND SLAB DETAIL



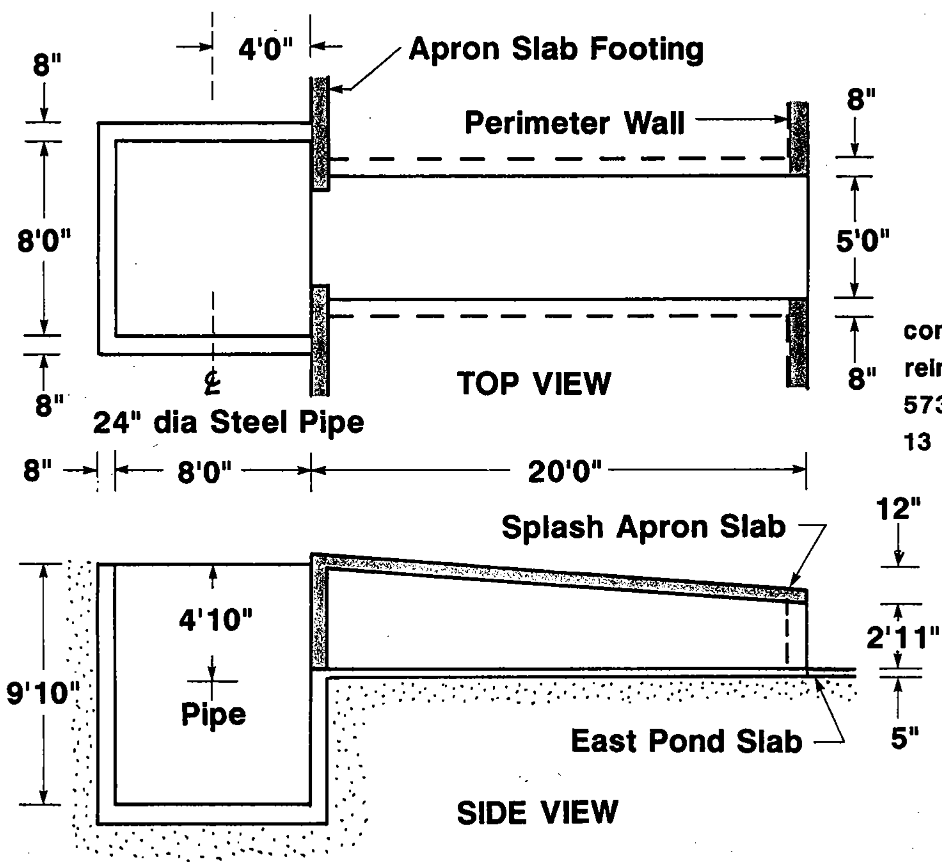
concrete: 5" thick

reinforcement: 10x10 ga 6x6" WWF

9,038 square feet

140 cubic yards

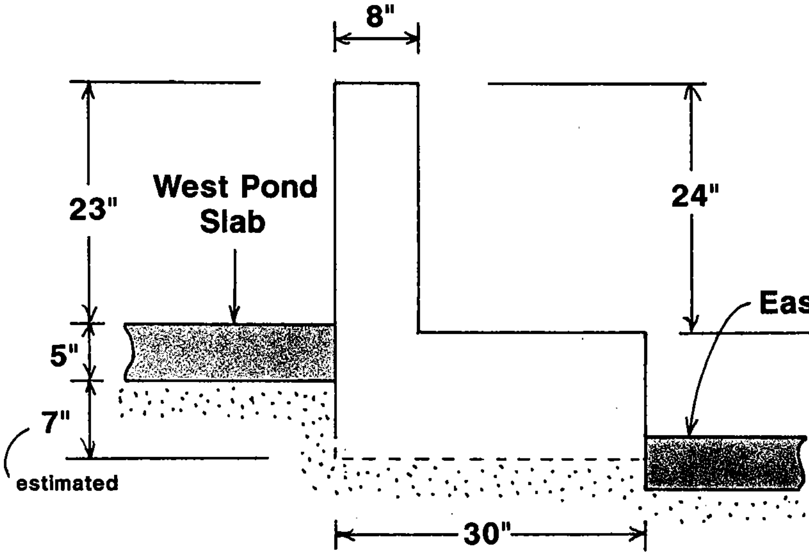
EAST POND SUMP DETAIL



concrete: 5" and 8" thick
reinforcement: unknown
573 square feet
13 cubic yards

Figure 18

INTERIOR WALL DETAIL



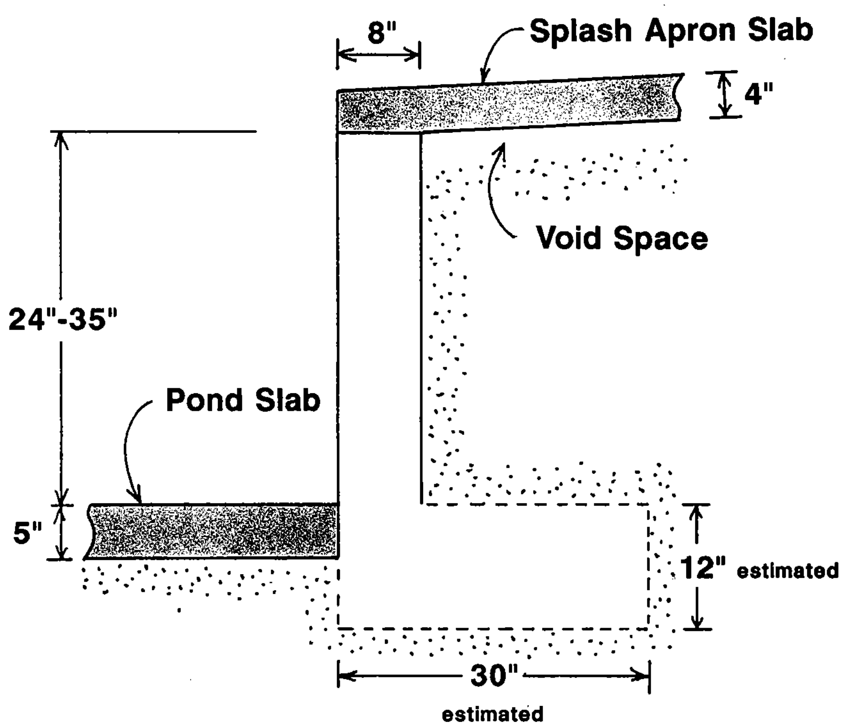
concrete: 8" and 12" thick

reinforcement: unknown

75 lineal feet

10 cubic yards

PERIMETER WALL DETAIL



concrete: 8" and 12" thick

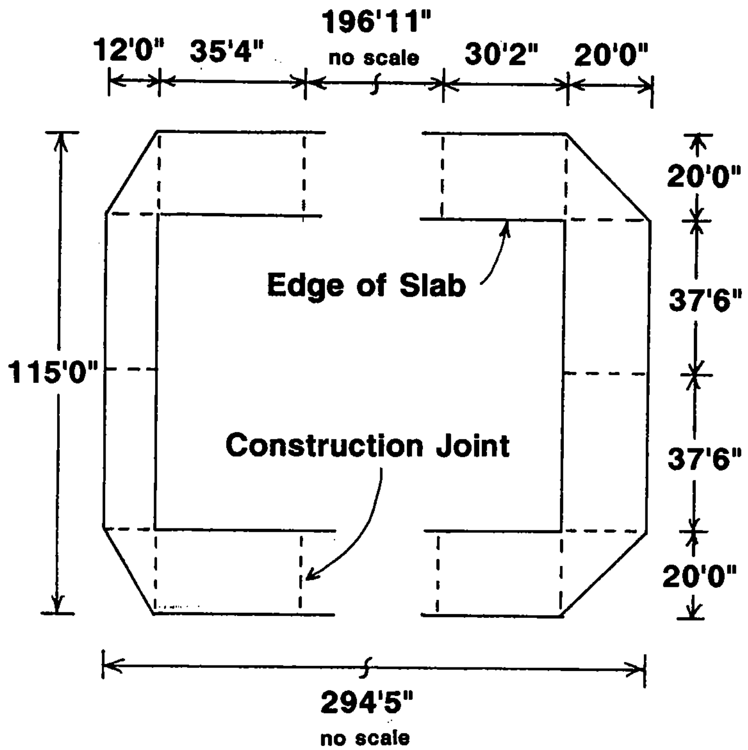
reinforcement: unknown

675 lineal feet

104 cubic yards

Figure 19

SPLASH APRON SLAB DETAIL



concrete: 4" thick

reinforcement:

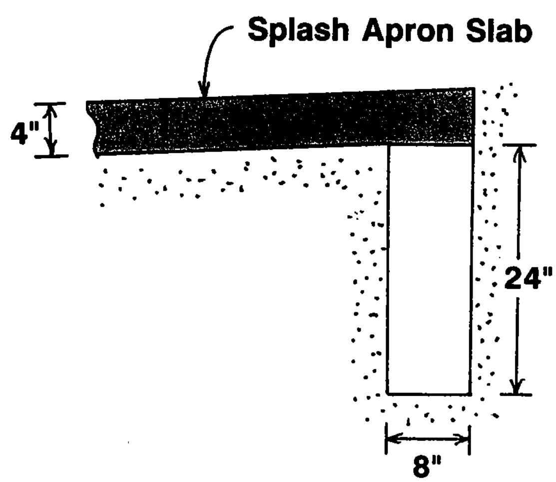
#2 at 24" OC each way

some 10x10 ga 6x6" WWF

13,537 square feet

165 cubic yards

APRON SLAB FOOTING DETAIL



concrete: 8" thick

reinforcement: unknown

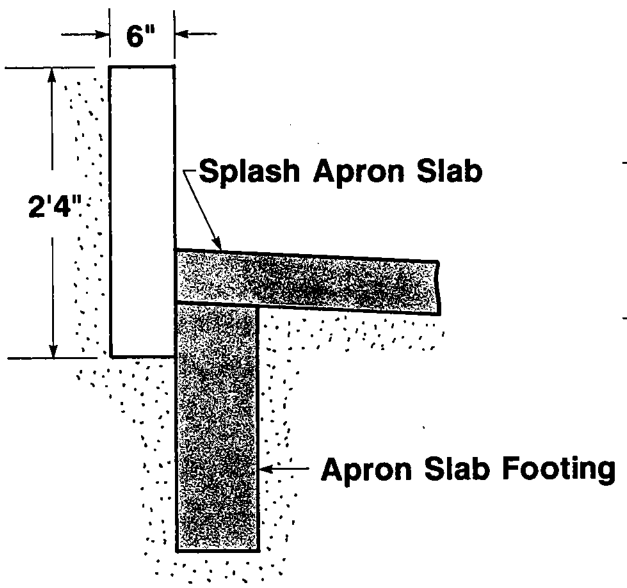
777 lineal feet

38 cubic yards

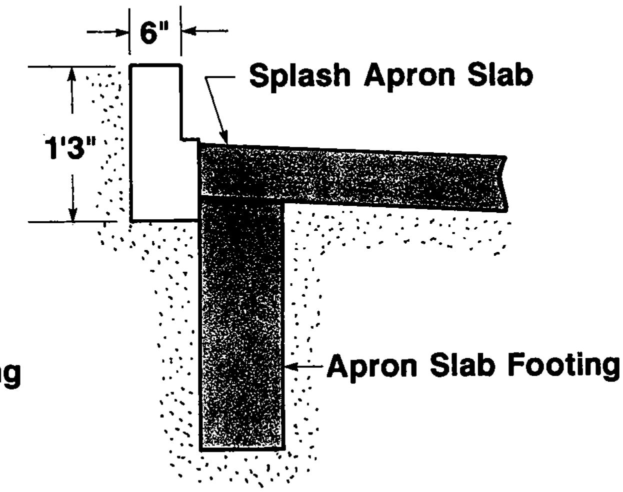
Figure 20

CURB DETAILS

concrete: 6" thick
reinforcement: unknown
174 lineal feet
6 cubic yards



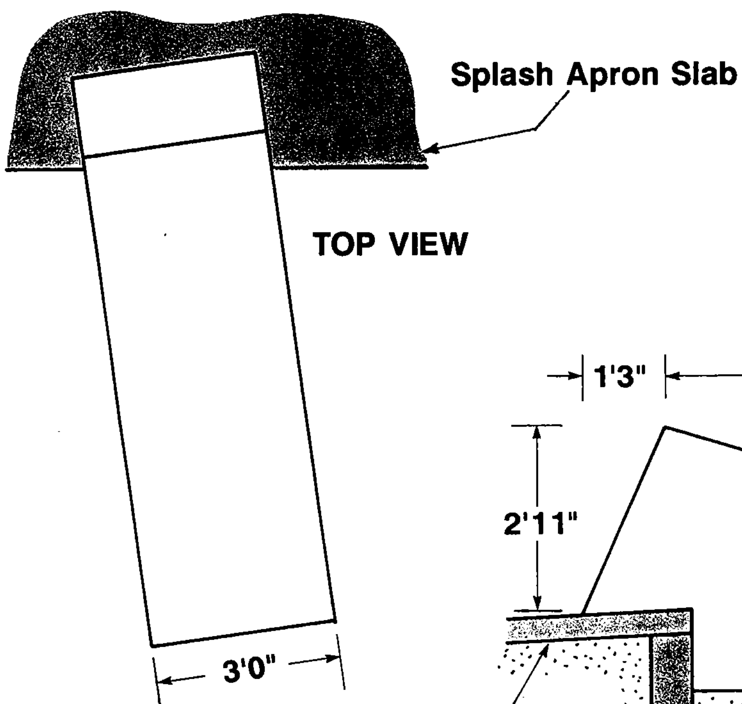
HIGH CURB



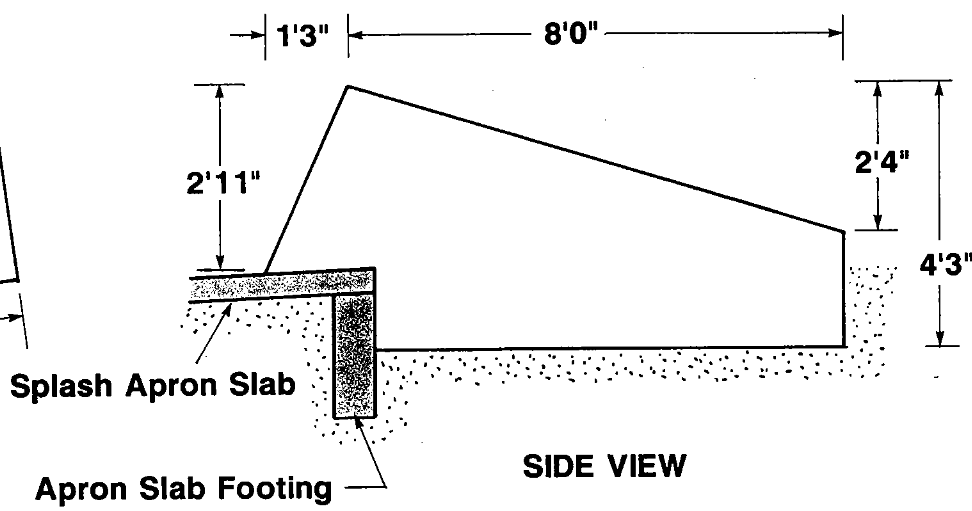
LOW CURB

ANCHOR BLOCK DETAIL

concrete: massive
reinforcement: unknown
3 cubic yards



TOP VIEW



SIDE VIEW

Figure 21

REFERENCES

Chapman, W.R. and Mott, R.A., 1928, The cleaning of coal: Chapman and Hill, London, England, 680 p.

Hudson Coal Company, 1932, The story of anthracite: Hudson Coal Company, 26 Liberty Street, New York, p. 111-218.

Indiana Department of Highways, 1985, Standard specifications: Indianapolis, Indiana, p. 63-70.

Indiana Department of Natural Resources, 1987, Final rules concerning the regulation of water well drilling: Indianapolis, Indiana, 10 p.

International Conference of Building Officials, 1985, Uniform code for the abatement of dangerous buildings: ICBO, Whittier, California, p. 12 -14.

Mitchell, D.R., 1950, Coal preparation: American Institute of Mining and Metallurgical Engineers, New York, 830 p.

Morse, F.T., 1932, Power plant engineering and design: Van Nostrand, New York, 813 p.

Office of Surface Mining, 1980, Abandoned mine land reclamation control technology handbook: U.S. Department of Interior, Washington, D.C., p. 9-1 to 9-50.

Peele, Robert and Church, J.A., 1941, Mining engineers' handbook, 3rd edition: John Wiley, New York, sections 16, 27, and 35.

Pledger, D.M., 1977, A complete guide to demolition: Construction Press, Ltd., London, England, p. 17-56.

Soil Conservation Service, 1988, Soil survey of Green County, Indiana: U.S. Department of Agriculture, Washington, D.C., 226 p.

Young, G.J., 1946, Elements of mining, 4th edition: McGraw-Hill, New York, p. 405-443.

AERIAL PHOTOGRAPH SOURCES

National Archives and Records Service
Cartographic Archives Division
General Services Administration
Washington, D.C. 20408 — (202) 756-6700

U.S. Department of Agriculture
Aerial Photography Field Office
2222 W. 2300 South
Salt Lake City, Utah 84119 — (801) 524-5856

U.S. Geological Survey
EROS Data Center
Sioux Falls, South Dakota 57198 — (605) 594-6511

SLOPE FAILURES ON HIGHWAY EMBANKMENTS HIGH IN SHRINK-SWELL CLAYS: PREVENTION AND REPAIR

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SLOPE FAILURES ON HIGHWAY EMBANKMENTS HIGH IN SHRINK-SWELL CLAYS: PREVENTION AND REPAIR

Scott F. Burns, William O. Hadley, Paul M. Griffin, Jack W. Mutchler, and Schaun M. Smith

ABSTRACT

It is well known that shrink-swell clays, known as smectites, can significantly reduce stability of embankment slopes. The question often asked is what amount of smectite is required to cause major stability problems since most soils contain some smectites, yet all soils are not problem soils. This paper addresses this particular phenomenon. In Louisiana, more than a hundred slope failures have occurred in embankments along the interstate highway system. It has been established that most of the failures have happened in soils high in smectites. From a study of the failed and non-failed slopes in the state, a model has been developed that predicts the chances of failure if a soil is used to construct an embankment. The procedure establishes the amount of shrink-swell clay that is acceptable and also the amount of smectite required to produce a problem soil. The model is easy to use and only requires Atterberg limits and particle size data of the soil to develop a prediction.

If a soil is determined to be a problem soil based on the predictive stability model, the embankment soil must be stabilized or the slope redesigned in order to prevent failure and/or repair the slope. Lime stabilization and geogrids are examined as soil stabilization techniques for use with traditional slope designs. Four design/rehabilitation methods are also presented that could be used with unaltered soils. The repair methods involving the installation of piling in the slope face or reconstructing the slope by pushing the failures back up the slope are not recommended.

INTRODUCTION

Slope instability is an enduring problem throughout the highway system of the United States. In Louisiana alone, more than 100 embankments have failed in the last ten years along the state's interstate highway system. More than sixty slope failures have occurred along I-10 between Lafayette and Lake Charles in the southern part of the state. In the northeast section of the state along I-20, over forty landslides have occurred in the past six years, primarily in Madison Parish in the Mississippi River flood plain.

The extent of the problem is manifest dramatically in the economic impact to the highway maintenance and remedial repair budgets. For the past ten years cumulative repair costs for landslides along I-10 have reached approximately $1.3 million and along the I-20 section, about $800,000. The projected cost of repair to the remaining slope failures exceeds $20 million. The solution to the problem of embankment slope instability and the resultant reduction of large capital expenditures lies in the understanding of why these embankments fail.

The objectives of this study were to establish the cause of these embankment failures along the interstate highway system and to recommend methods of repair/rehabilitation and new construction to prevent their occurrence on other slopes. The objectives were accomplished through: characterization of geotechnical and field properties of the failed and non-failed slopes along the highways; statistically comparing the characteristics of the two types of slopes to determine why some failed and others did not; making predictions as to future slope failures; and making recommendations about construction and repair of highway embankments. Application of these results and models are not restricted to Louisiana, but can be used any place where embankments are constructed.

It was determined that failures occurred most often in slopes constructed of soils high in smectite (i.e.shrink-swell clays composed of mainly montmorillonite). It is surmised that, control of slope stability depends on control of these clays.

This report presents the prediction model for slope failure developed in the project, as well as the different strategies proposed for construction and repair of smectite-rich embankments. In addition, information is provided for a project where sixteen slopes have been repaired using four soil stabilization methods. A more elaborate summary of the project is found in Burns et al, 1989.

STUDY AREA

Two sections of interstate highway were chosen as the study area. A section of I-10 between Lafayette and Lake Charles was selected as the southern study area because more than 60 failures had been recorded in this section (Figure 1). The portion of I-20 between Monroe and Tallulah was chosen as the northern study area. In this segment, the majority of the landslides had occurred in Madison Parish, at the eastern end of the I-20 transect. The slopes found in these two areas provided a good representative cross section of embankments built in the state on the many different parent materials.

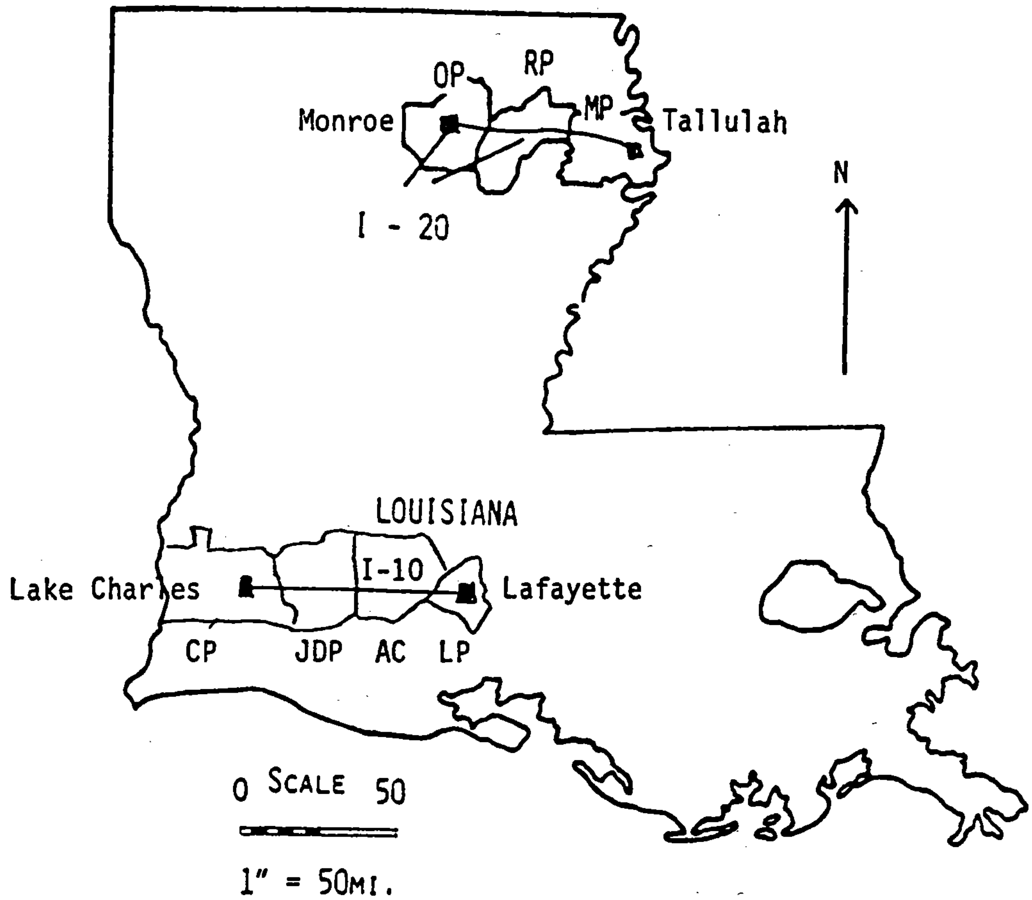

Figure 1, Location map of study areas within the state. The I-20 Study Area transect connects Monroe and Tallulah through Ouachita Parish (OP), Richland Parish (RP), and Madison Parish (MP). The I-10 Study Area transect connects Lake Charles and Lafayette through Calcasieu Parish (CP), Jefferson Davis Parish (JDP), Acadia Parish (AC), and Lafayette Parish (LP)

The climate can be characterized as consisting of mild winters and warm summers. The mean annual air temperature is 18^{o}C (66^{o}F) with 50-120 days during the summer with temperatures exceeding 32^{o}C (90^{o}F) and 30-65 days during the winter when the temperature falls below freezing (Newton, 1972; Weems et al, 1982). Winter cold spells are usually of a short duration, and droughts in the summer rarely last over a month (Weems et al, 1982). The mean annual precipitation is about 1300 mm (51 inches) in the north(Weems et al, 1982) and 1500 mm (59 inches) in the south (Clark et al, 1962). Large rainfall events in short periods of time, such as thunderstorms and hurricanes, are common.

The I-20 transect stretches 82 km (51 miles) and can be divided into three provinces. The eastern portion of the area is in Madison Parish which lies in the flood plain of the Mississippi River, and the alluvial sediments of the soils are fine-grained with high clay contents (Weems et al, 1982). In the middle of the study area, lying primarily in Richland Parish, is the Macon Ridge which is a terrace of braided stream deposits of the ancestral Mississippi River and is called the Mississippi Valley Train alluvium (Saucier, in press). The soils are sandier than in the rest of the study area. The eastern half of the Macon Ridge is veneered with a loess sheet. The western portion of the study area lies in the flood plain of the Ouachita River where the soils have a high clay content with some silts and gravels (Matthews et al, 1974). Terrain of the whole area is essentially flat.

The I-10 transect covers 114 km (71 miles) and crosses four parishes (from west to east): Calcasieu, Jefferson Davis, Acadia, and Lafayette. The terrain is basically flat because most of the study area lies on the upper Pleistocene Prairie Terrace. This surface consists predominantly of fine-grained clays, silts and argillaceous sands that have been deposited in a fluvial-deltaic environment (Autin et al, in press). At the eastern end in Lafayette Parish, the surface is covered with a veneer of loess from one to nine meters thick. The surface is also incised by four main rivers that drain the area. This modern alluvium, which is used in the making of some of the embankments, is much more clay rich than the Prairie Terrace deposits.

METHODS

Analysis of the embankment slopes in the study areas occurred in five steps. First, field work consisted of locating, mapping, and classifying slope failures. Samples of soils were taken from failed and non-failed slopes at 50 cm depths. Second, the samples were characterized in the laboratory for their sedimentological, geotechnical, mineralogical, and hydrogeological properties. The copious data generated is summarized in Burns et al, 1989. Third, computer models of the failures were developed using the field and laboratory data. The PCSTABL4 program developed at Purdue University was the main program used. Fourth, slope designs were developed using the computer models. Fifth, sixteen slopes were repaired in Madison Parish using four soil stabilization methods to establish a long term monitoring program to evaluate differing repair methods.

RESULTS

GENERAL RESULTS

A total of 242 embankments were examined along a combined 196 km (122 miles) section of I-10 and I-20 highways in Louisiana. A total of 99 slope failures were observed in the highway embankments along these sections. The failures were classified as slumps (32), slump-earthflows (44), and earthflows (23). The numbers in the parentheses represent the number of observed occurrences. The mean failure volume determined for all of the failures was 428 m^3 (15,105 ft^3.

All embankment failures had occurred in the time frame of 8-15 years after their construction. This time frame represents a contrast to that of the western United States (with rainfall of about 300 mm/yr or 12 inches/year), where embankments that were constructed 25-30 years ago are now beginning to fail (Terry Yarger, personal communication at the conference). From this information, it is possible that there is a "curing time" for all slopes, with embankments in higher rainfall areas undergoing a shorter time before failure.

Of the four parent materials composing the highway embankment slopes (modern alluvium, Prairie Terrace alluvium, Mississippi Valley Train alluvium, and loess), over 70% of the failures occurred in slopes constructed of modern alluvium. It is believed that the high amount of smectite in the modern alluvium soils caused most of the failures due to changes in soil status with moisture increases.

DEVELOPMENT OF A PREDICTIVE MODEL

Using T-tests, ANOVA tests, and risk model ranking, a predictive model was developed for the first 15 years after construction (Burns et al, 1989) (Table 1). The three characteristics that exhibited significant correlation with failed slopes were clay content, plasticity index, and liquid limit, all of which are related to the smectite contents of the soils. A fourth characteristic, termed net smectite, also had a

TABLE 1

SLOPE STABILITY RISK CATEGORIES FOR EMBANKMENT SOILS

GEOTECHNICAL TEST	HIGH RISK*	INTERMEDIATE RISK*	LOW RISK*
LIQUID LIMIT	> 54%	36 - 54%	< 36%
PLASTICITY LIMIT	> 29%	16 - 29%	< 16%
CLAY CONTENT	> 47%	32 - 47%	< 32%

* : HIGH RISK CATEGORY: 85-90% chance of failure in 8-15 years after construction

INTERMEDIATE RISK: 55-60% chance of failure in 8-15 years after construction

LOW RISK CATEGORY: < 5% chance of failure in 8-15 years after construction high correlation with slope failure, but it is considered to be too difficult to determine and was eliminated from further consideration. (It is discussed in depth in Burns et al,1989.) The predictive model was developed for three soil characteristics using risk model techniques.

Three risk categories were established from a risk model ranking of available slope failure information (Burns et al, 1989). High risk slopes are classified as those with a 85-90% chance of failure in 8-15 years after construction. Intermediate risk slopes are those with a 55-60% chance of failure in the same time frame. Low risk slopes are those with a chance of failure less than 5%. This model should work for any slopes constructed in climates similar to that of Louisiana. For other climates, the model could also be utilized, but the time period before failure should be adjusted to account for climatic differences. For drier climates, the time period to failure would be lengthened, while for wetter climates, the time to failure is expected to be shortened. Further experimental work is required to establish this time to failure/climate relationship.

The chances of slope failure in a 15 year period after construction are low for soils that fall within the low risk category. In this instance, traditional slope design methods can be used without modification. If the embankment soil is classified in the intermediate or high risk category, then the soil is considered a "problem expansive soil". In this situation, the chances of slope failure in the next 15 years after construction are 55-90%. Therefore, these soils must be stabilized or the slope design must be altered.

A difference between high and medium risk categories was developed primarily as an aid in establishing priorities in repair and indicating which soils are more prone to failure. If slopes have not failed, yet are sampled and the soil falls into the high risk category, then there is a much higher chance of failure during the next large rainfall event than a slope of medium risk category.

Before constructing an embankment, the Atterberg limits and clay contents should be determined for the embankment soil. Each of the results should be compared to the risk model (Table 1), and categorized as high, intermediate, or low risk. The overall sample risk category is the one that has the most examples in it (i.e. sample that has the liquid limit and clay content in the high risk category and the plasticity index in the intermediate would be considered high risk). In most cases, the Atterberg limits should provide sufficient information for classification, and clay content does not have to be run unless the two fall into different classifications.

SLOPE STABILITY MODEL FOR STUDY AREA

A slope stability model is presented here that adequately characterizes the embankment cross section as a three layer soil system. When applied to 31 existing slopes, it accurately predicted the stability state of 90% of the slopes. In the cross section (Figure 2), soil layer 1 is the top zone of vegetation and soil covering the slope and is considered to be 30 cm (12 inches) thick. Soil layer 2 is the embankment fill and is considered cohesive clay with a maximum cohesive strength of 175 psf. Soil layer 3 is the natural soil surface upon which the embankment has been constructed and is strictly cohesive clay with a minimum cohesion of 350 psf.

Two important conclusions were developed from the PCSTABL4 computer characterizations of the slopes. About 2/3 of the cases investigated resulted in failure surfaces extending well below the surrounding ground surface, i.e. deep seated failures. Also, the predicted failure surface originated well back of the top of the embankment slope approaching the edge of the shoulder. The designs of the embankment slopes should incorporate these two features. The deteriorated condition of the slopes in the field would not allow us to determine if the failures were deep seated or not. Based on this field data, it seemed that most of the failures were in the embankment soil and were not deep seated.

DESIGN AND REHABILITATION OF EMBANKMENTS

If a soil sample for a slope is determined to be in a high or intermediate risk category, the soil must be stabilized before a conventional construction method is used or a new design must be developed to account for the low strength clays. Soil stabilization can be accomplished by lime addition or geogrid emplacement. An additional design used in Mississippi that can be used when space is at a premium, is to cut the slope back, install drains, and then rebuild the slope using new soil material or geogrids (Mike Wright, personal communication at conference). Bottom ash (as discussed at the conference) and fly ash could also possibly be used to stabilize these problem soils. New embankment designs could be used to construct a new slope composed of intermediate/high risk soils or to reconstruct an older failed slope with the existing embankment soil that has not been stabilized by lime or geogrids. These new or rehabilitationdesigns could be based on the slope stability model presented in the last section.

The options considered in the selection and evaluation of an appropriate design will depend upon space available for the construction and economy of construction, as well as existing or projected soil strength. As a result, no rule of thumb can be offered concerning an appropriate design option. The

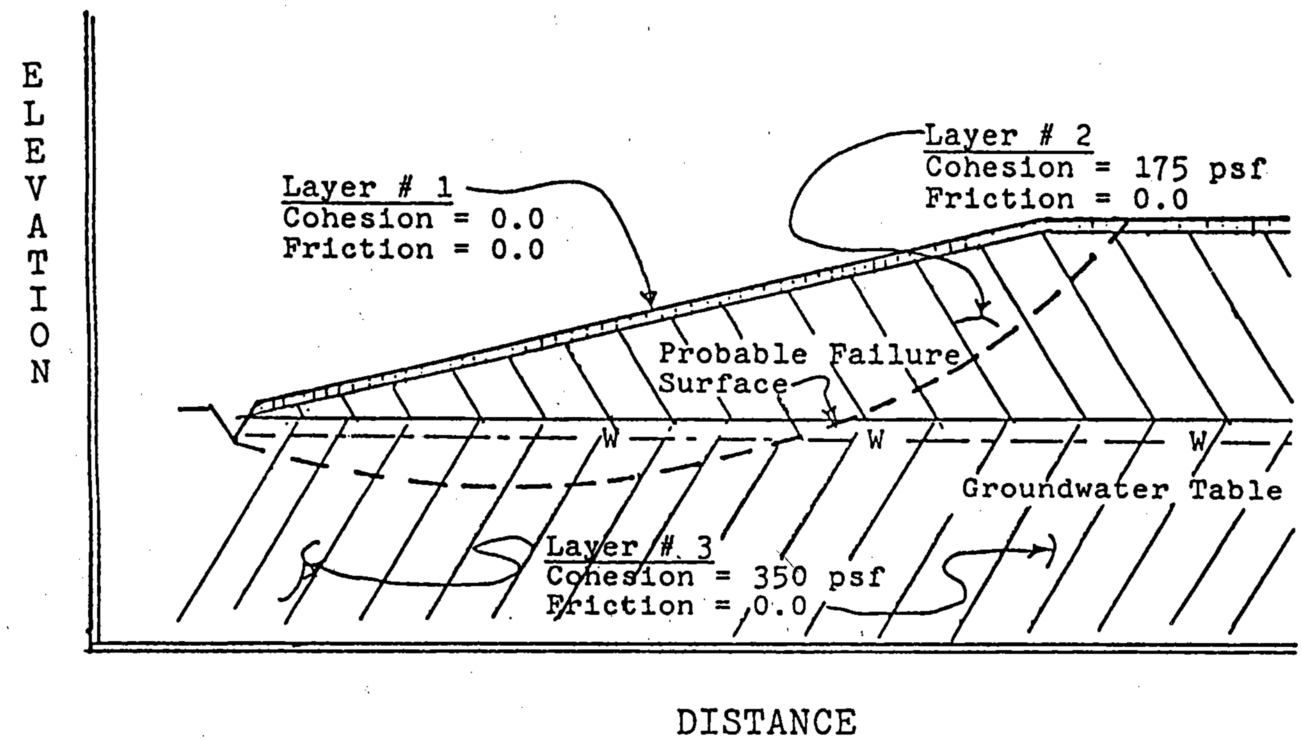

Figure 2 General Soil-Embankment Conditions
Slope Stability Model Cross Section

circumstances surrounding a particular job site would therefore determine to a great extent the design option or soil stabilization that would meet the project requirements.

1) <u>Constant embankment slope configuration</u>: The design nomograph for the constant embankment slope is presented in Figure 3. This choice would be utilized in those situations where acquisition of right-of-way is not a problem and low slope angles can be used. This method would be cheaper than the lime stabilization method as the natural soils would only be compacted, not limed. The input for this nomograph would include the height of the embankment, H, and the desired stability number (factor of safety). The intersection of a horizontal line through the stability number with a vertical line through the height, H, would yield the minimum acceptable embankment slope, S.

2) <u>Broken-back embankment configuration</u>: This slope configuration (Figure 4) is intended for cases where there is limited right-of-way or reconstruction of an existing embankment where lime stabilization is not wanted. The "broken back term" describes an embankment configuration consisting of two different slopes. The upper slope is the steeper of the two. This configuration was selected because it represented an opportunity to reduce the mass of earth captured within the failure surface and should result in a lesser driving force and corresponding higher stability number. The nomograph (Figure 4) is used in a fashion similar to Figure 3. A vertical line would be constructed from the height of the embankment, H, while a horizontal line would be constructed from the desired stability number (factor of safety). The intersection of these two lines would yield the combination of slopes S1 and S2 which meets the stability requirement.

A similar approach to the broken-back design is used in Mississippi where a berm is built at the bottom of the slope, essentially yielding a two slope system (Mike Wright, personal communication at conference).

3) <u>Broken-back embankment with stabilized soil layer</u>: In the first two design configurations, the failure surface dipped into the third soil layer. This third configuration was established to create a situation where a stabilized soil layer would limit the intrusion of the predicted failure surface into the bottom soil layer. In addition, the stabilized layer would substantially reduce the movement of soil moisture from the bottom layer into the upper embankment soil layer. It is more expensive than the above two methods and would require stabilization of the lower soil layer, probably with lime stabilization. The nomograph (Figure 5) is used in the same

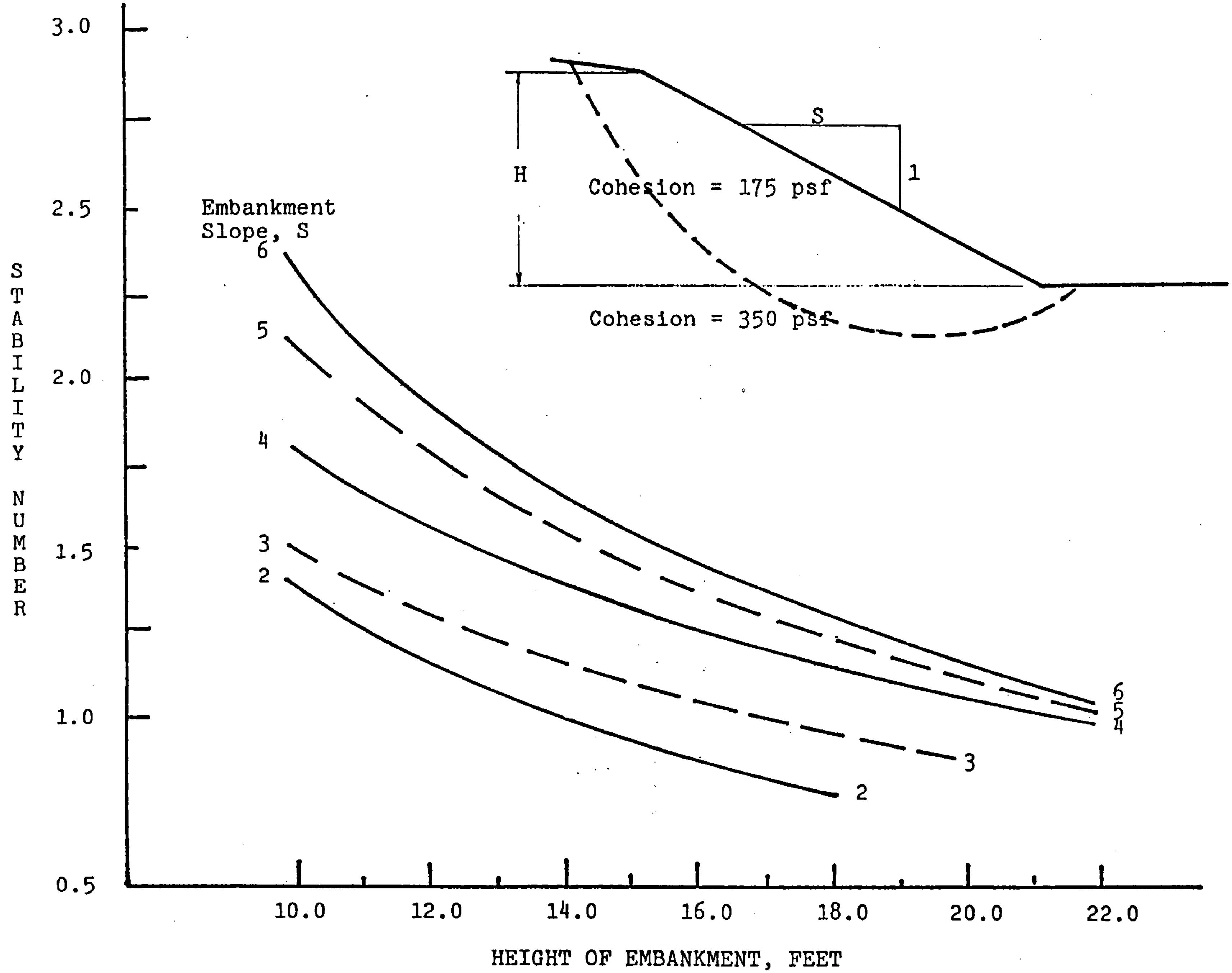

Figure 3: Stability Numbers for Constant Slope of Embankments and Embankment Height

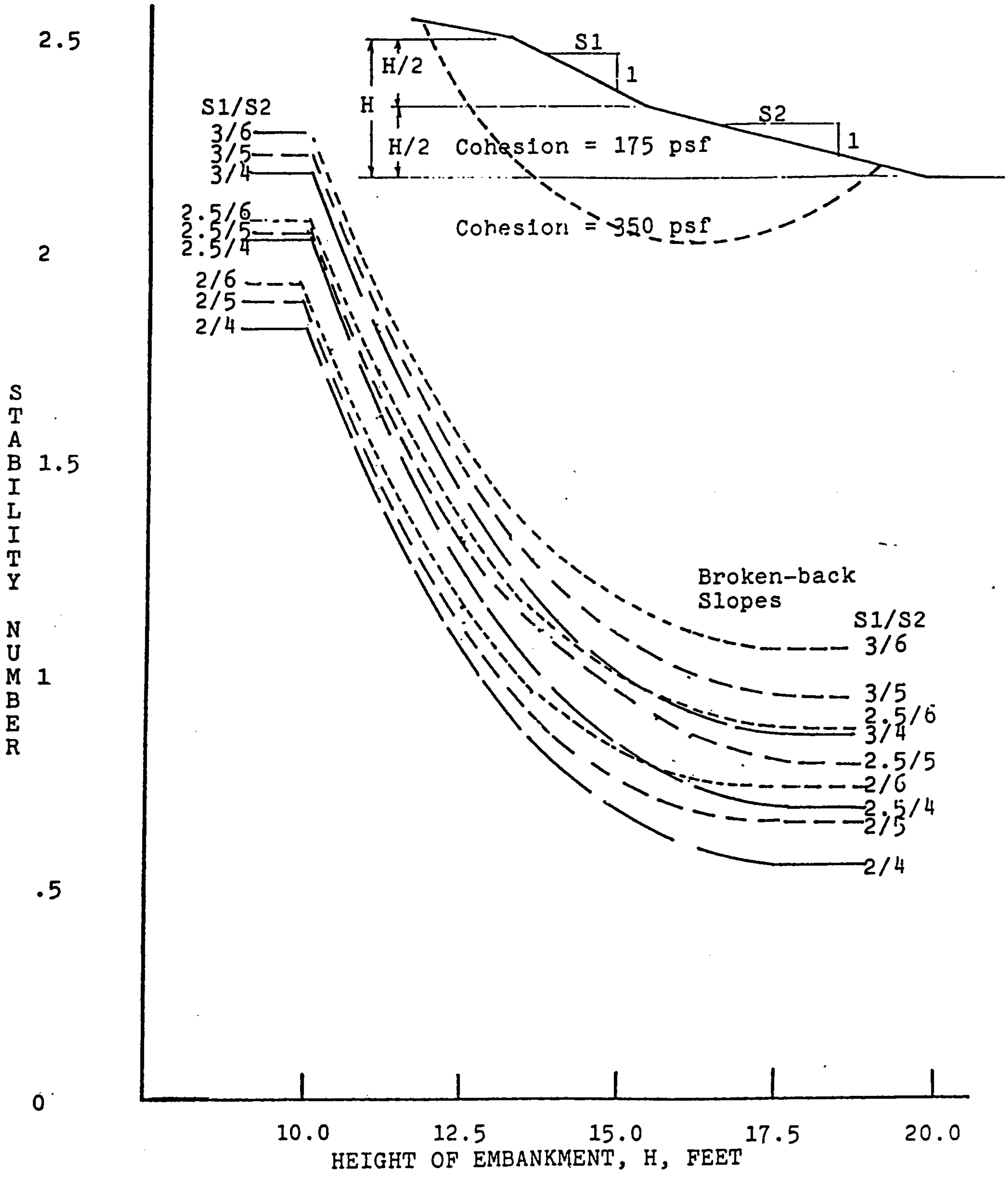

Figure 4: Stability Numbers for Broken Back Embankment Configuration with Varying Heights and Slopes

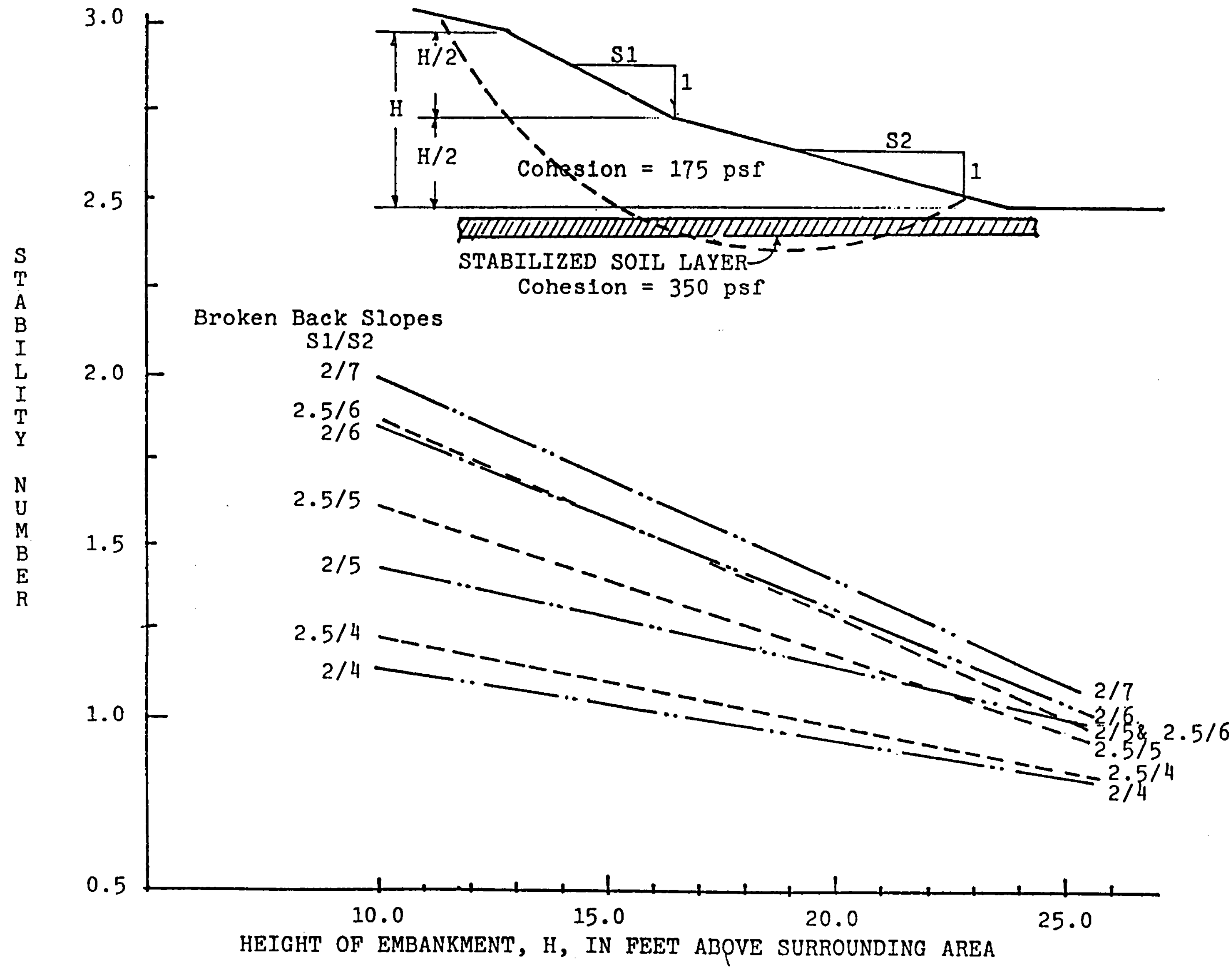

Figure 5 : Stability Numbers For Broken Back Embankment Configuration and Stabilized Soil Layer At Toe Of Slope; Consideration of Height, H, and Slopes, S1 & S2

fashion as the preceding one. The stabilized soil layer would have to have a minimum thickness of 45 cm (18 inches). Economics would, of course, play a major role in evaluating this option.

4) <u>Constant slope configuration with stable subgrade</u>: The fourth case involves the placement of a clay material as embankment over a stable subgrade with a cohesive strength of 700 psf or more. This would represent a case in which the in situ soil material would be stable and unaffected by available groundwater. Minimum embankment slopes for this situation can be obtained by the appropriate entry of the nomograph (Figure 6) with embankment height, H, and desired stability number (factor of safety).

Where soil moisture movements can be controlled and the soil cohesive strength can be established, the four nomographs can be used to establish minimum embankment slopes. Detailed directions on how to establish these minimum values and how to make corrections for increased cohesion in the soils are found in Burns et al, 1989. Another method of repair, using soil reinforcement ties is also elaborated on in the same volume.

5) <u>Construction/Design Recommendations</u>: These recommendations are offered for new construction or reconstruction of repaired slopes. 1) Do not use pilings to repair slopes or temporarily prevent slope movements where high or intermediate risk soils are used in the slopes. The pilings become conduits for moisture to enter the slope and actually increase the chances of slope instability. 2) Always provide ample drainage or cutoff ditches to keep water off of the slope. 3) Extend the revetment under the overpass around to the sides of the embankment. 4) Vegetate the slope with pampas grass (<u>Cortaderia selloana</u>) if it is available. This plant will not require mowing which will further stabilize the slope.

<u>EXPERIMENTAL REPAIR OF EMBANKMENTS, MADISON PARISH, LOUISIANA</u>

Sixteen slopes in Madison Parish that were classified in the "high risk" category were stabilized using four different techniques in 1986 to 1988. The four techniques were lime stabilization, lime injection, lime-fly ash injection, and geogrids. The purposes of the project were to observe the construction of the various embankment rehabilitation techniques and to provide slopes for long term observation. The slopes were selected at three overpasses and one box junction along I-20 in northeast Louisiana (Figure 7).

At the Tendal and Cow Bayou sites (Figure 7), the lime stabilization method was used. All four quadrants of each overpass had failed prior to embankment repair. At these sites,

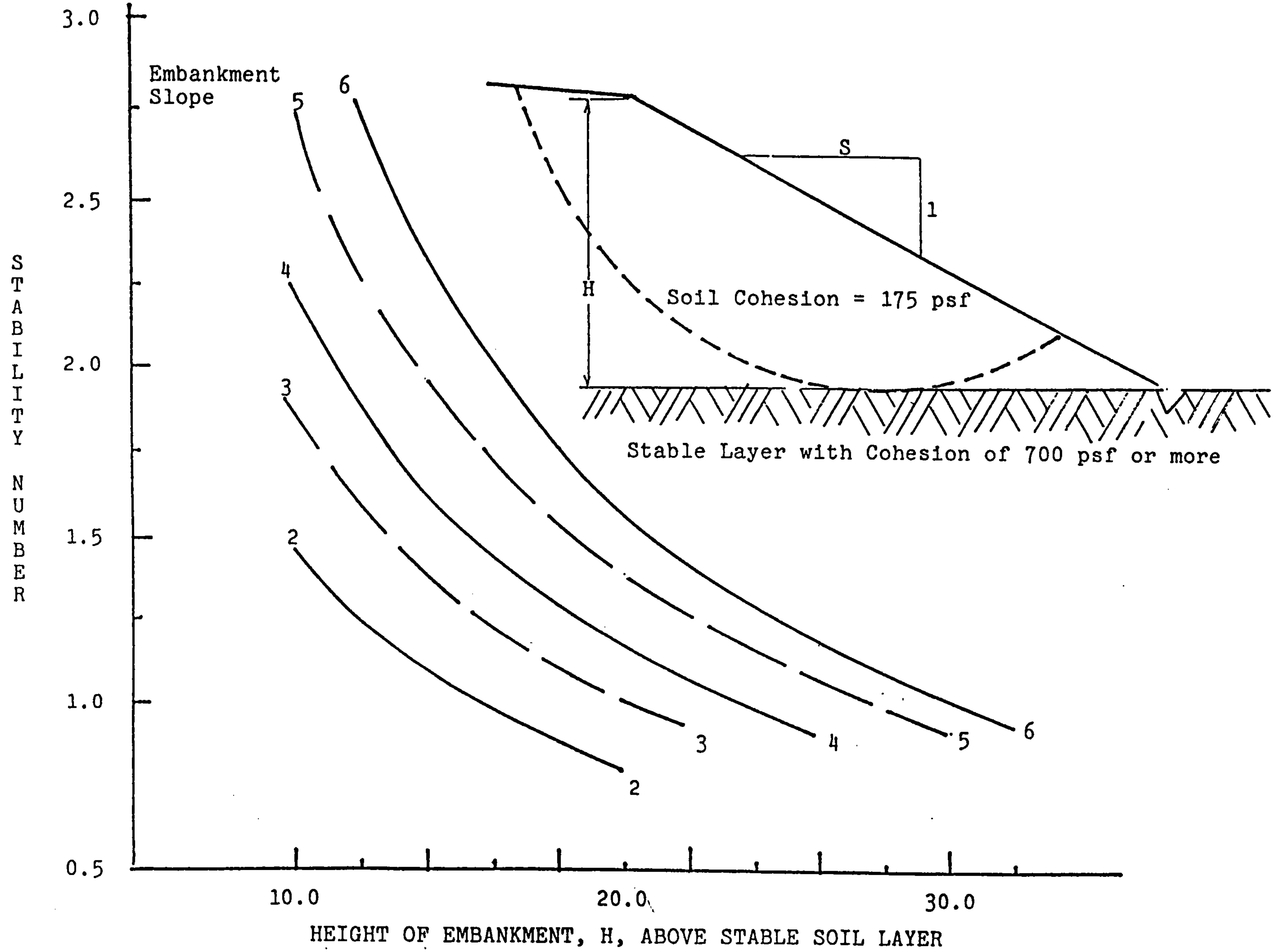

Figure 6: Stability Numbers for Embankment Founded on Stable Subgrade

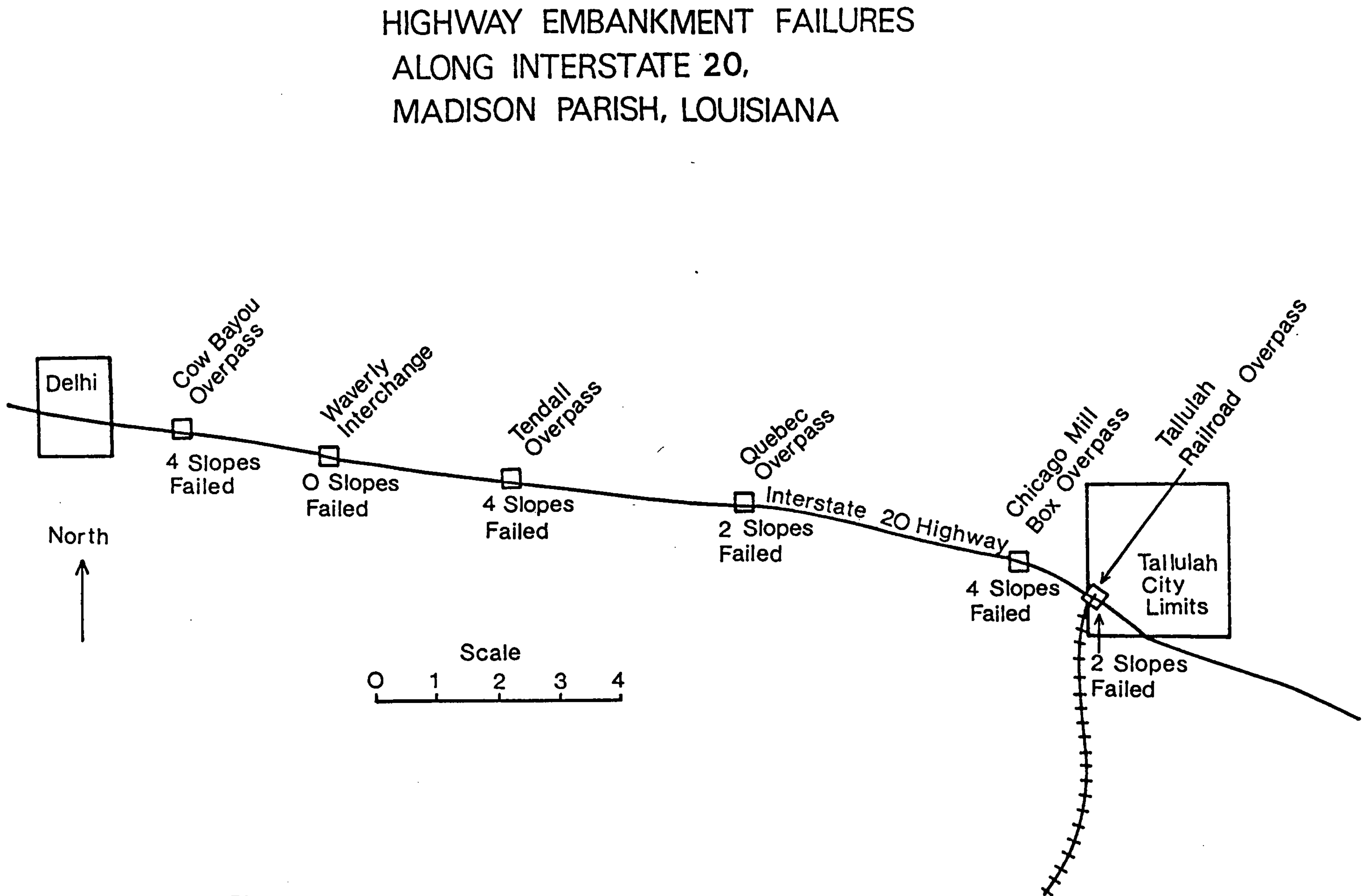

Figure 7: Sites of repaired slopes in Madison Parish

the slope material was stripped back from a 3:1 to a 2:1 slope and spread out onto a "table" at the slope bottom that was parallel to the slope. During this process, the wooden and metal pilings that had previously been driven into the slopes, attempting to stabilize them, were removed. Next, the "table soil" was mixed with hydrated lime. A predetermined amount of lime (most of the time about 12%) was spread across the table and then mixed into the top 23 cm (9 inches) of soil with a large mixing machine. Once the soil and lime were mixed, it was pushed back onto the slope with bulldozers and compacted in 23 cm (9 inch) lifts. The slope was restored to a 3:1 angle. Finally, grass was applied to the slopes. The cost of these restorations was approximately $35,000 per slope or $140,000 for the four slopes of each overpass. The lime treatment resulted in an immediate soil stability through the reduction of the plasticity index of the slope soils. The construction was considered a success and is further being monitored to ascertain acceptability of the technique for permanent embankment stabilization of slopes. Thornton and Kirkpatrick (1987) also used the lime stabilization method in Arkansas where it was successful, but was the most expensive technique they tried.

At the Quebec overpass (Figure 7), the two western quadrants had not failed and the eastern quadrants had multiple failures. The high pressure injection technique was used here, with lime being injected into the northern slopes and a mixture of lime-fly ash being injected into the southern slopes. This technique encountered many problems of machinery breakdowns, uneven distribution of the injected mixture, damage to the road surface from "boils" where the mixture passed through the surface, time delays, and cost over runs. After almost two years, the grass on the northwest slope has not come back because so much lime was injected. These injection techniques probably will not be repeated in the future because of the many construction problems encountered. Thornton and Kirkpatrick (1987) also rejected this technique in a project in Arkansas.

The geogrid technique was used to repair the slopes at the Chicago Mill site (Figure 7) where I-20 passes over a box junction. All four quadrants had failed. The soils had some of the highest liquid limits and smectite contents of all of the samples in the study. The sides of the slopes were stripped away similar to the Cow Bayou and Tendal sites to a 2:1 slope, and the pilings were removed. The Tensar SS2 plastic grid network was laid down between each of the compacted layers of soil in the slope reconstruction. This technique has been successful at other sites in other states, and if successful for these highly susceptible soils, it might be a good technique to use in the future. Thornton and Kirkpatrick (1987) also used this geogrid technique in Arkansas and the slopes are still stable.

CONCLUSIONS

The assurance of stable slopes apparently centers around control of the shrink-swell clays called smectites. In soils with high concentrations of these expansive clays, the embankment soils must be stabilized chemically or slopes must be redesigned to account for the low strength clays.

Before slope construction is undertaken, problem soils must be noted using the risk model. Although most soils contain shrink-swell clays, the model provides a technique for designating the problem soils. The soils that are classified as high and intermediate risk types must be altered using techniques such as lime stabilization or geogrids, or the slope design changed to fit the existing situation and requirements. No special attention is required for soils that are classified as low risk soils.

Where embankment failures have developed, their repair can be successfully completed using lime stabilization, geogrid stabilization, or slope redesign using the methods mentioned above. Most of the soils of those failed slopes are expected to fall within the high and intermediate risk categories.

All existing embankments should be tested to estimate the possibility of future failures. If most of the embankments are constructed of low risk soils, the maintenance should be minimal If the embankment soils fall within the high and intermediate categories, the maintenance funds for repair work should be designated in future budgets.

ACKNOWLEDGMENTS

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REFERENCES

Autin, W.J., Saucier, R.T., and Snead, J.I., "Introduction to Chapter on 'Quaternary Geology of the Lower Mississippi Valley'" In Morrison, R.B., ed. Quaternary Non-Glacial Geology of the Conterminous United States, Geological Society of America, Decade of North American Geology Series, in press.

Burns, S.F., Hadley, W.O., Mutchler, J.W., Smith, S.M., Siddiqui, A., and Hernandez, M.J. Development of Design Criteria for Prevention of Slope Failures, Report 210, Louisiana Transportation Research Center, Louisiana Department of Transportation and Development, Baton Rouge, 1989.

Clark, H.L., Haley, G.J., Hebert, E.J., Hollier, R.M. and Roy, A.J., Soil Survey of Acadia Parish, Louisiana, USDA Soil Conservation Service, 1962.

Mathews, D., Reynolds, E.F., Colvin, G.P., Weems, T.A., Ray, C.A., Seaholm, J.E. and Kilpatrick, W.W. Soil Survey of Ouachita Parish, Louisiana. USDA Soil Conservation Service, 1974.

Newton, M.B. Atlas of Louisiana, A Guide for Students. School of Geoscience, Louisiana State University, Miscellaneous Publication 72-1, Baton Rouge. 1972.

Saucier, R.T., "Geomorphology, stratigraphy and chronology of the Lower Mississippi Valley", In Morrison, R.B., ed., Quaternary Non-Glacial Geology of the Conterminous United States. Decade of North American Geology Series, Geological Society of America, in press.

Thornton, S.I. and Kirkpatrick, W.E., "Cures for slope failures in Arkansas", Proceedings of the 38th Annual Highway Geology Symposium, Pittsburgh, Pennsylvania, p. 124-126, 1987.

Weems, T.A., Reynolds, E.F., Venson, R.L., Allen, E.T. and Smith, T.J., Soil Survey of Madison Parish, Louisiana. USDA Soil Conservation Service, 1982.

DESIGN APPLICATIONS OF THE WELDED WIRE WALL

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John R. Selvage, P.E.

First developed by The Hilfiker Company ten years ago, more than 1500 Welded Wire Wall structures have been constructed through the United States. The Welded Wire Wall system has been applied under a variety of conditions and loadings, including: retaining walls, bridge abutments, slope failure repairs, soil stabilization, and erosion control. This paper reviews engineering applications of the Welded Wire Wall to specific project requirements.

INTRODUCTION

The Welded Wire Wall is a patented, reinforced soil system, consisting of welded wire fabric mats installed between successive lifts of backfill (figure 1.) Analysis of the Welded Wire Wall system assumes that the reinforced soil mass behaves as a coherent, gravity retaining wall. Conventional external stability criteria of overturning, sliding, bearing capacity and deep stability are applied to the reinforced mass. In addition, internal stability, involving tension in the longitudinal reinforcing elements and pullout of the reinforcing mats must be satisfied. Bishop and Anderson (1979) and Nelson (1985) discuss the analysis and design of Welded Wire Walls in detail.

While a complete review of the design procedure is not within the scope of this paper, a brief discussion of the current state-of-art analysis of internal stability is important.

The longitudinal wires of the reinforcing mats must be designed to resist lateral, internal earth pressures in tension. The tensile face applied to each longitudinal wire can be expressed as:

$$T = \sigma h \cdot W \cdot Z \qquad (1)$$

where T = Tensile load, lbs.
σh = Horizontal earth pressure, psf
W = Horizontal spacing of longitudinal wires, ft.
Z = Vertical spacing of the reinforcing mats, ft.

The horizontal earth pressure (σh) can be represented by:

$$\sigma h = K \cdot \gamma \cdot Y \qquad (2)$$

where K = Lateral earth pressure coefficient
γ = Unit weight of backfill, pcf
Y = Depth from top of wall to the reinforcing mat, ft.

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Based on instrumentation of an early Welded Wire Wall and laboratory testing, Bishop and Anderson (1979) first suggested at K value of 0.65 (at-rest) for design. Later work by Anderson et.al. (1985) involving instrumentation of a 55'-6" high Welded Wire Wall in Seattle, Washington, concluded that the value of K varies with depth: a K value of 0.65 for mat depths to 15 feet below the top of the wall; K varying linearly from 0.65 to 0.45 for mat depths of 15 to 20 feet; and a K value of 0.45 below 20 feet (figure 2.) Varying the value of K with depth as recommended by Anderson et.al. (1985) is the current design practice for Welded Wire Walls.

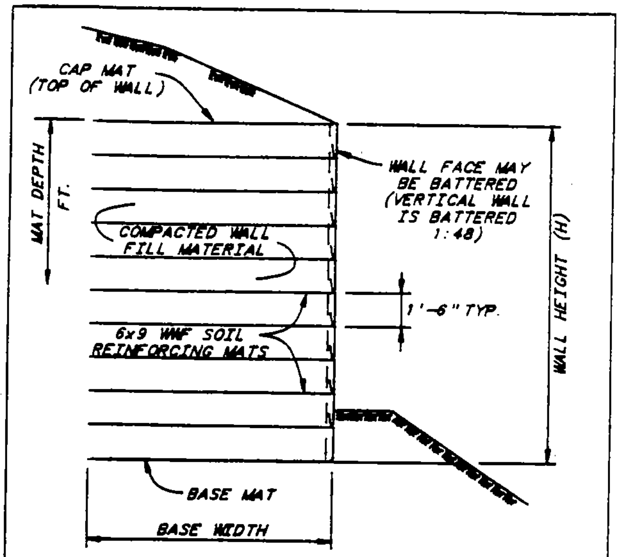


FIG. 1: WELDED WIRE WALL

The criteria governing pullout resistance within the reinforced soil mass of a Welded Wire Wall was also first suggested by Bishop and Anderson (1979). Assuming a potential Coulomb failure plane within the reinforced mass, reinforcing mats must extend far enough beyond the potential failure plane to prevent failure.

Subsequently, Anderson et.al. (1985) confirmed that the "coherent gravity structure" bi-linear failure surface presented by McKittrick (1978) for Reinforced Earth© walls was valid for structures using grid reinforcings (figure 2).

Further, Nielsen and Anderson (1983) and Hannon and Forsyth (1984) have shown the pullout of grid reinforcings to be directly related to the number of transverse wires behind the failure surface, the diameter of the transverse wires, friction on the longitudinal wires and overburden pressure.

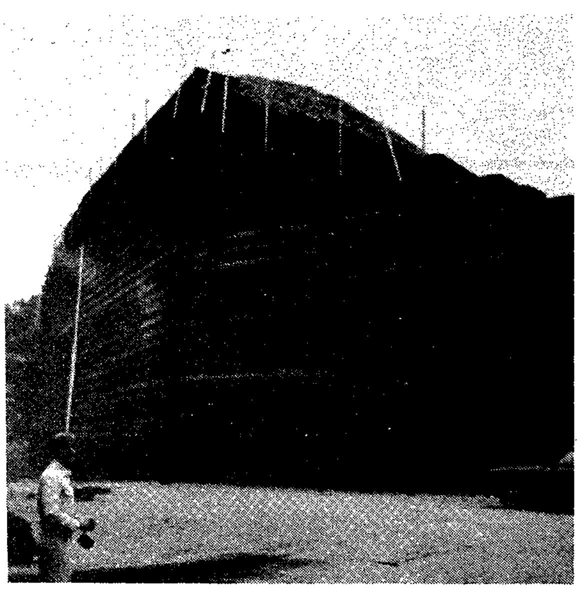
In the design of a Welded Wire Wall or any other reinforced soil wall system, five primary factors effect both internal and external stability and must be addressed:

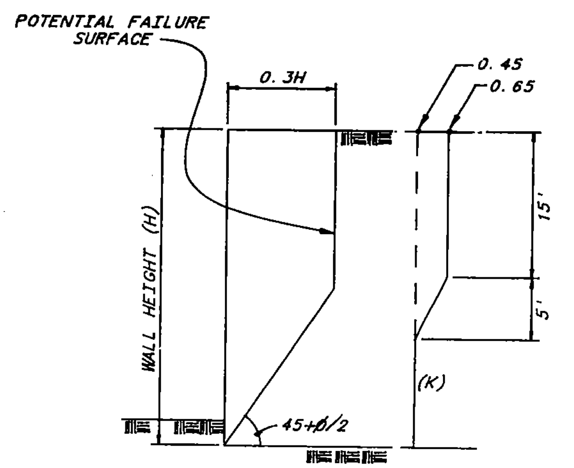
- °External Loadings
- °Foundation Conditions
- °Backfill Properties
- °Drainage
- °Geometric Constraints

An external loading for the purposes of this discussion is broadly defined as any force acting upon the reinforced soil mass and includes earth pressures, live load surcharges, line or point loadings, abutment or footing pressures, hydrostatic pressures, and seismic accelerations.

Foundation conditions must be reviewed for allowable bearing capacities, soil friction values, and settlement potential. In considering settlement, both total vertical displacement and differential settlement should be addressed. In the case of the Welded Wire Wall system which is inherently flexible, differential settlement in a plane perpendicular to the face of the wall is the most critical, as it may lead to outward leaning of the wall face and cracking in the fill at the back of the reinforcing mats. These conditions, if not extreme, are not necessarily a problem in themselves. However, outward lean of a wall face presents the appearance of a failed or failing system and cracking behind a wall provides a conduit for drainage and potentially the saturation of wall backfill.

The physical properties of the backfill within the reinforced soil mass are used in determining both the external and internal stability of a wall system. In-place unit weight, cohesion and friction values are necessary for calculating external stability factors of safety and bearing pressures. For internal stability, unit weight, cohesion, and angle of internal friction values are





(b) RELATIONSHIP BETWEEN POTENTIAL FAILURE SURFACE & K VALUE & DEPTH BELOW TOP OF WALL

a) RAINIER AVENUE WALL

FIG. 2

required to determine reinforcement stresses and pullout loads, as well as pullout resistance of the soil reinforcings. In addition, knowledge of the backfill gradation, plasticity index, pH and chloride content is used in determining both the service life and constructability of a structure.

Control of both surface and subsurface drainage is critical for a reinforced soil system to function properly. All reinforced soil systems rely upon the strength characteristics of the backfill for internal stability. Depending upon the backfill material selected, saturation, whether from surface or subsurface water can greatly reduce the strength of the backfill and result in failure. Also, in most applications, reinforced soil walls are not designed to resist hydrostatic pressures.

Geometric constraints may be defined as any condition which limits the space available for installation of soil reinforcements. Examples of constraints include existing structures or features which limit excavation, rock which is cost prohibitive to remove, drainage improvements, existing structures such as bridge abutment wingwalls to which a wall must conform and environmental encroachments.

The balance of this paper is a discussion of design applications of the Welded Wire Wall under three of the factors previously discussed: External loadings, geometric constraints, and backfill properties. In each case, actual projects will be used to illustrate the concepts described.

EXTERNAL LOADINGS

Chevron Resources' phosphate storage project and the U.S. Forest Service's (U.S.F.S.) Shoal Cove project represent applications of the Welded Wire Wall under nontypical external loadings. The Welded Wire Wall constructed at Chevron's Vernal, Utah, facility serves as a containment dike for phosphate storage (figure 3.) The wall, constructed in 1983, is 270 feet long with a maximum height of 45 feet. Because the volume of phosphate storage varies with time, the wall was designed as a conventional retaining wall at maximum storage capacity and as a free standing structure when storage is depleted (figure 4.)

For external stability, the critical condition exists at full storage, when the phosphate slopes upward at a 1.4:1 slope from the back of the wall. It was determined that the sloping phosphate exerts an equivalent fluid pressure loading of approximately 30 pounds per cubic foot (pcf.) Assuming minimum factors of safety of 2.0 and 1.5 for overturning and sliding respectively, a minimum base width to height (B/H) ratio of 0.5 was established for external stability. Resultant footing pressures were not a significant concern, as the wall was founded on native sandstone with a bearing capacity well in excess of the loads imposed by the wall.

In contrast, for internal stability the controlling condition occurs when storage is depleted and the wall is free standing, exposing the back of the wall. At the time the wall was designed, internal stability criteria assumed at-rest soil conditions for the full height of the wall and a Coulomb failure plane within the reinforced soil mass.

Assuming the back of the wall to be exposed and unsupported, a failure plane could theoretically propagate from either the front or rear of the reinforcing mats. In addition, backfill near the back of the wall required confinement both during and after construction. The design solution was to install soil reinforcing mats based on the minimum B/H ratio of 0.5 required for external stability and to add a facing at the back of the wall. With the Welded Wire Wall system, the facing is an integral part of the soil reinforcing mat. At the rear of the wall, 5'-0" long mats were installed, overlapping the main reinforcing mats a minimum of 4'-0" and the two were mechanically connected. In this manner, the backfill was effectively confined between the front and rear facings, preventing development of a failure plane in either direction (figure 4.) The longitudinal members of the reinforcing mats were sized to resist tensile loads based upon the at-rest earth coefficient, overburden pressures, and their tributary areas.

Final design considerations consisted of battering the front face at 1:6 (horizontal to vertical) to soften the effect of the wall's 45 foot height and of using only two mat lengths of 10 and 22 feet. Limiting the mat lengths minimized the irregularities in the alignment of the back of the wall, increased the constructability of the structure and provided a cleaner surface for operation.

As a part of the U.S.F.S. Shoal Cove project on the Ketchikan National Forest, Alaska, a 92 foot simple span bridge is to be constructed over Calamity Creek. The bridge abutments are to be supported on spread footings which will bear directly on Welded Wire Walls (figure 5.) At the time of the writing of this paper, materials for the Welded Wire Walls had been shipped to the job site; but construction had not yet started. Two features of this wall design required special consideration; the direct bearing of the abutment footings on the walls, and submergence of lower wall areas during high stream flows.

In cases where abutments bear directly on the reinforced soil mass and thereby resist both overturning and sliding, external stability seldom governs the final wall design. However, based on 50 year flood parameters established by the U.S.F.S., the

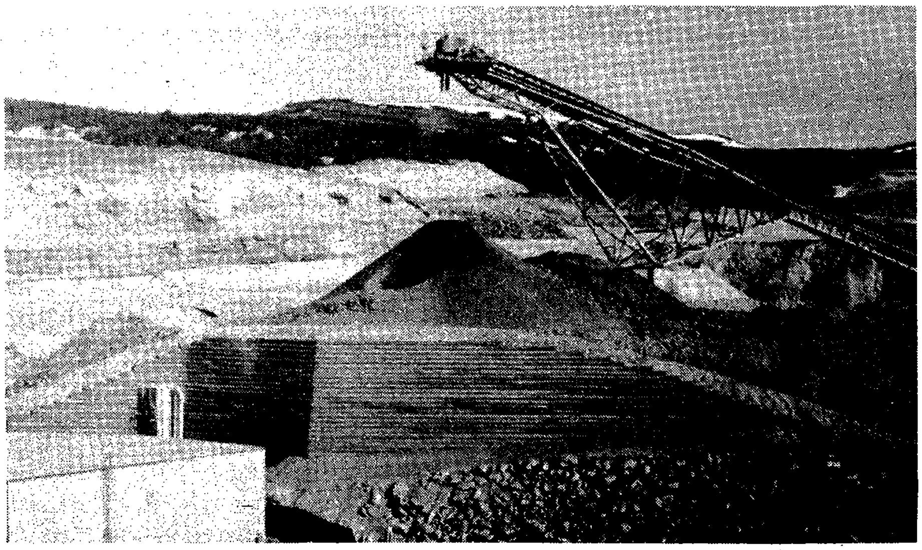


FIG.3: CHEVRON WALL

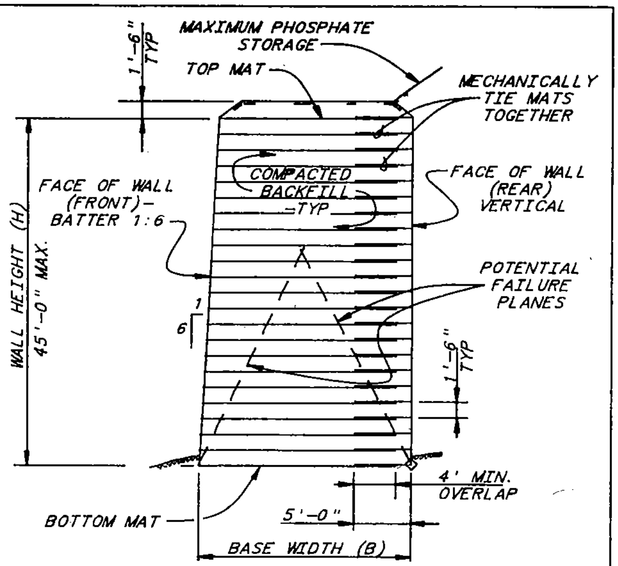


FIG. 4: CHEVRON WELDED WIRE WALL TYPICAL SECTION

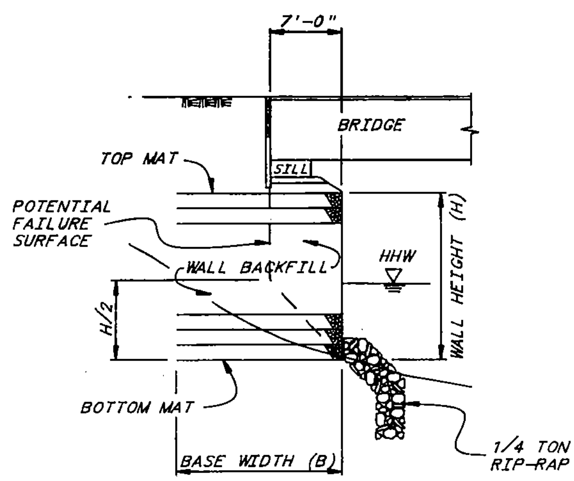


FIG. 5: CALAMITY CREEK BRIDGE ABUTMENT

lower 8 feet of one wall is subject to submergence. The current design procedure for walls which may experience partial submergence assumes that a 3 foot differential in water levels may develop between the back of the wall and the stream as water levels subside following a major storm event. The wall must be designed to satisfy external stability requirements under this condition as well as with applicable soil and live loads.

Designed in October 1986, the internal stability analysis is based on earth coefficients which vary with depth and a bi-linear failure plane modified as shown in figure 5. The design assumes that the failure plane will not develop directly below a footing supported by the wall, but will coincide with the back edge of the footing or remain at 0.3(H) whichever is the greater distance from the face of the wall. For a typical wall 16'-6" in height, the failure plane would be located approximately 5 feet behind the face. For Calamity Creek, due to the abutment footing, the distance from the face of the wall to the failure plane is increased to 7 feet.

In addition to the soil loadings on the reinforcing mats, the 3.0 ksf abutment footing pressure, equivalent to 25 feet of soil surcharge must be resisted by the reinforcings in tension and pullout. Equation (2) can be modified for the added abutment load and expressed as:

$$\sigma_h = K\ (\gamma \cdot Y + fb \cdot W\ / (W + Y1)) \tag{3}$$

Where fb = Abutment footing pressure, ksf
W = Abutment footing width, ft.
Y1 = Depth from bottom of abutment footing to the soil reinforcing mat

A vertical load dispersal of 1:2 (horizontal to vertical) is assumed for fb. Also, if the abutment footing is loaded eccentrically or a significant longitudinal bridge load exists, they must also be accounted for in the stress distribution.

To satisfy internal stability, which controlled the design, a 5 foot minimum reinforcing embedment beyond the failure plane and a minimum B/H ratio of 1.0 was required. Mat lengths equal to the height of the wall or 12 foot minimum, whichever was greater were specified.

To prevent loss of strength in the backfill during submergence and to minimize differential water levels at subsidence, a granular, free draining, non-plastic backfill with a maximum of 5 percent fine material (passing the No. 200 sieve) was specified. Also, as Calamity Creek is subject to salt water intrusion from

Shoal Cove, corrosion allowances were based on a 2.0 oz. per square foot zinc coating (ASTM A-123) and a chloride rich environment. Combined with the high stresses induced by the abutment footing, this resulted in selection of W9.5 x W4-6x9 WWF soil reinforcings as compared to W4.5 x W3.5-6x9 WWF that is typically used in walls of comparable height.

Additional design considerations included the location of the abutment footing and installation of special backfill at the face of the wall. To prevent loss of backfill through the face due to stream velocities, a layer of cobble is to be installed in the face of the wall (figure 6.) The cobble will serve two purposes; to act as an energy dissipater and to provide ultraviolet shielding for the filter fabric. Filter fabric behind the cobble will prevent loss of backfill to the stream. A 2'-9" setback from the face of the wall to the front of the abutment footing was established to ensure that the footing will bear fully and uniformly on compacted wall backfill.

GEOMETRIC CONSTRAINTS

The most common geometric constraints on the cross section of a soil reinforced wall are existence of bedrock, which is cost prohibitive to remove, and roadways, structures, or environmental features which restrict the excavation slope above a wall. Welded Wire Wall installations at the Geysers and Goleta in Northern California and Telluride, Colorado, represent projects in which constraints required special wall designs.

Constructed in late 1984 and 1985, Squaw Creek Road provides access to geothermal well sites at the Geysers in Sonoma County, California. Thirteen Welded Wire Walls totalling approximately 30,000 square feet were installed as part of the road construction. Of these walls, six were constructed in areas where excavation was controlled by existing rock. Due to orientation of bedding planes, rock excavation was limited to predetermined boundaries, beyond which existed the potential for slope failures extending several hundred feet above the proposed roadway.

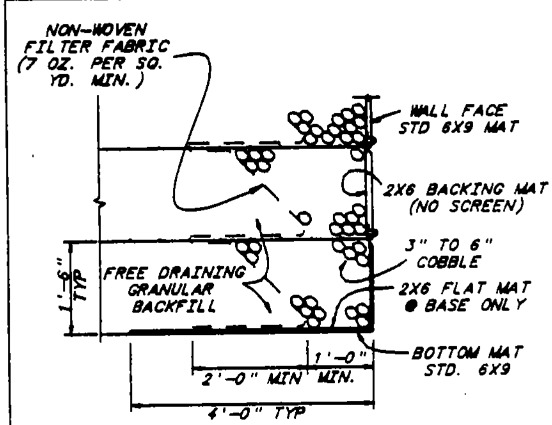


FIG. 6: COBBLE FACING

To accommodate the limited excavation, the wall design provided for mat lengths that decreased with depth below the top of the wall (figures 7 and 8.) The most critical of the walls was at Station 144+40 where a wall height of 39 feet was necessary and the bottom mats were limited to 13'-6".

Due to the configuration of the internal failure plane, it is possible to shorten mats near the bottom of a wall and still maintain adequate resistance to pullout. However, our current design procedure limits the minimum mat length to a B/H ratio of 0.35-0.40 and assumes that no wall loading is generated by material behind the wall from any area in which mat lengths do not produce to a B/H ratio of 0.50 or greater. In addition, the upper reinforcing mats must be lengthened such that the vertical resultant of soil pressure falls within the middle one-third of the wall. It should be noted that while many of the mechanisms involved in the design of reinforced soil structures are clearly understood, some design criteria are still empirical, based on observation of the performance of completed structures.

For the Squaw Creek Walls, rock bolting was specified for any rock which could not be excavated and which would potentially load the lower portions of the walls. Also, the lower reinforcing mats were manufactured longer than specified by the

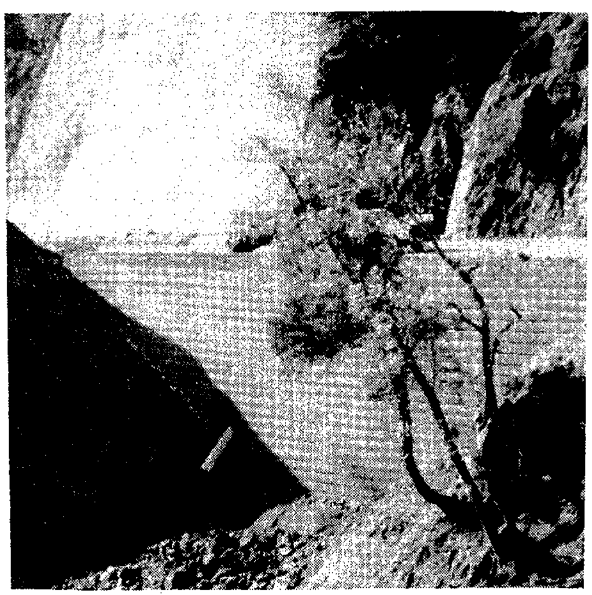


FIG.7: SQUAW CREEK WALL

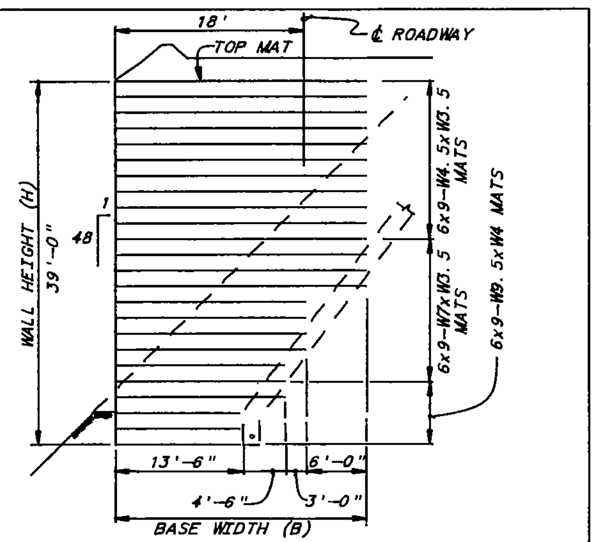


FIG. 8: SQUAW CREEK STA 144+40– TYPICAL WALL SECTION

design and field cut to conform to the exposed rock. This prevented development of a non-reinforced soil column behind the lower wall areas and the potential for loadings not accounted for in the design. In all cases, a minimum B/H ratio of 0.35 was maintained.

In contrast to Squaw Creek, the excavations at Goleta and Telluride were not limited by rock. At Southern California Edison's Casitos substation, which services the 220 KV Santa Clara-Goleta transmission line, a slope failure was threatening to undermine a transmission tower (figure 9.) Initially, a low timber pole wall, 5 to 6 feet in height, was installed. As the pole wall proved to be ineffective, the utility company decided to replace it with a Welded Wire Wall in 1984. The new wall was to be 120 feet long with a maximum height of 28'-6".

Edison's geologist defined the loadings to be resisted by the wall as a 50 pcf equivalent fluid pressure and a horizontal seismic acceleration of 0.10g. To provide adequate factors of safety for external stability, a minimum B/H ratio of 0.77 was required, with sliding the controlling criteria.

Approximately 40 feet of the length of the wall excavation was controlled by the transmission tower at the top of the slope and the substation at the toe of the wall. In this area the the bottom reinforcing mats were limited to lengths of 14'-3" to 16'-6". This provided a B/H ratio of roughly 0.53 and was insufficient for external stability. The solution, which to our knowledge was the first of its kind, was to construct a cast-in-place concrete footing and stem wall approximately 9 feet high as part of the Welded Wire Wall installation (figure 10.) A concrete key extending 5 feet below the bottom soil reinforcing mat was designed to develop passive soil pressures and provide an incremental resistance to sliding (figure 11.) The stem wall was constructed to a height, where based on mat lengths and remaining wall height above the top of the stem wall, B/H ratios were adequate to satisfy external stability.

Even in the most limiting case, mat lengths resulted in B/H ratios in excess of 0.5 and were sufficient to ensure internal stability. As such, the stem of the cast-in-place concrete "key wall" needed only to provide resistance to shear through the reinforced soil mass; varying from zero at the top of the stem to a maximum at the top of the concrete footing. The construction sequence specified required installation of the Welded Wire Wall to a height equal to the top of the stem, prior to casting the stem wall and thereby minimizing the transfer of internal soil stresses to the concrete structure.

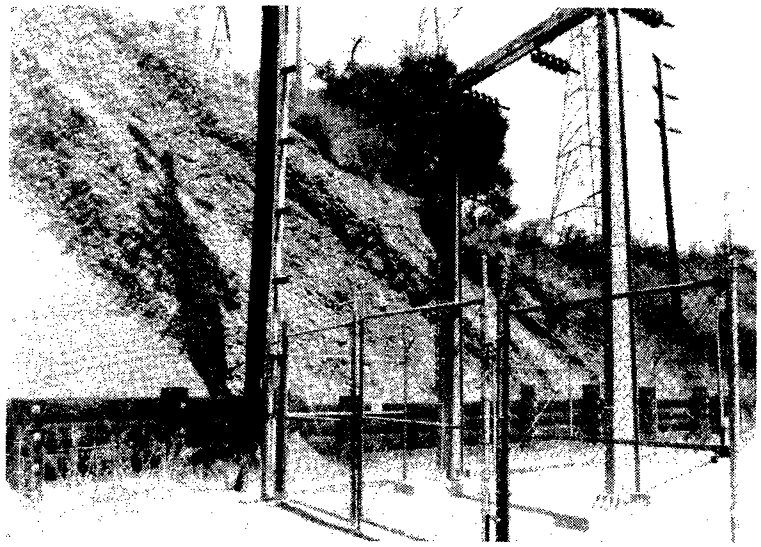


FIG.9 : CASITOS SUBSTATION

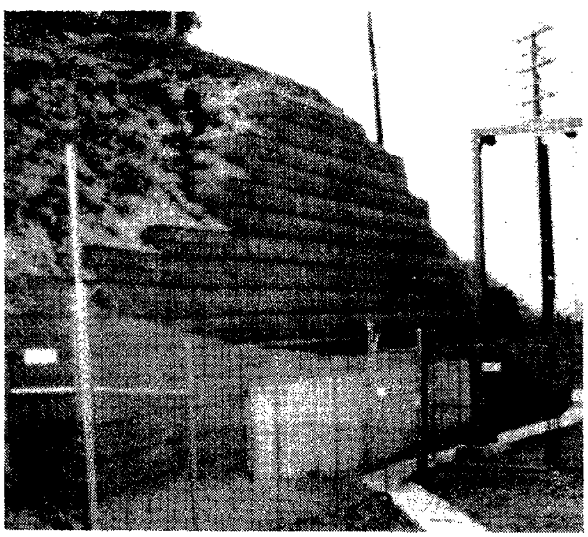


FIG.10 : GOLETA WALL

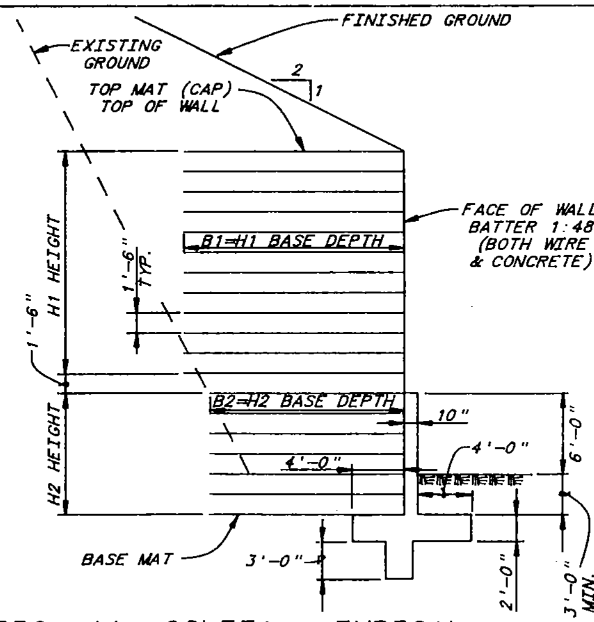


FIG. 11: GOLETA – TYPICAL WALL SECTION

Similar to the Goleta project, five of the sixteen Welded Wire Walls constructed at the Telluride Ski Area in Colorado during 1984 and 1985 included the use of concrete "key walls." Also similar to Goleta, these walls were subject to active soil pressures for their full heights. However, at Telluride, excavation limits resulted in B/H ratios as low as 0.36; which when combined with full height soil loadings were not adequate to maintain internal stability. As such, the entire reinforced soil area could not be considered to act as a single coherent mass.

As a result, the "key wall" stems were designed not only to provide additional resistance to sliding, but also for the surcharge of the wall above (figure 12.) The design provided for the transfer of horizontal shear and vertical bearing pressures from the portion of the Welded Wire Wall above the stem wall to the concrete structure and the portion of the Welded Wire Wall below.

It should be noted that the design of the Telluride walls was also effected by the architectural treatment of wall face. Following construction of each Welded Wire Wall and installation of a concrete leveling course at the base of each wall, native rock was dry stacked to form an architectural facing (figure 13.) To accommodate the stability of the facing, the Welded Wire Walls were battered at 1:6.

BACKFILL PROPERTIES

To date, the main body of research and testing related to reinforced soil retaining wall systems has assumed a granular, cohesionless backfill within the limits of the soil reinforcings. As such, nearly all reinforced soil walls constructed in the United States have used ostensibly cohesionless backfill, and little information exists on the use of backfill having both frictional and cohesive properties. By neglecting the use of cohesive material, backfill for many wall projects must be imported, often at considerable cost. In some instances, the high cost of imported backfill has increased the cost of reinforced soil walls to such a magnitude that their inclusion in a project is not economically feasible.

The Bay Area Teleport project, located in Niles Canyon, California, is an example in point. Eight Welded Wire Walls were included in the 1986 construction of the access road that services the satellite communications facility upon which the project is based (figure 14.) The project geotechnical consultant identified the on-site material as silty clay to clayey silt, with an internal angle of friction of 24 degrees and cohesion of 250 psf. It was also determined that imported backfill was cost prohibitive and native soils would have to be used as backfill for the Welded Wire Walls.

Design criteria including lateral earth pressures, coefficients of friction, passive soil pressures, bearing capacities, and drainage were presented in the project geotechnical report, Terratech (1985). Because of the clay content of the on-site soils, at-rest soil coefficients were recommended.

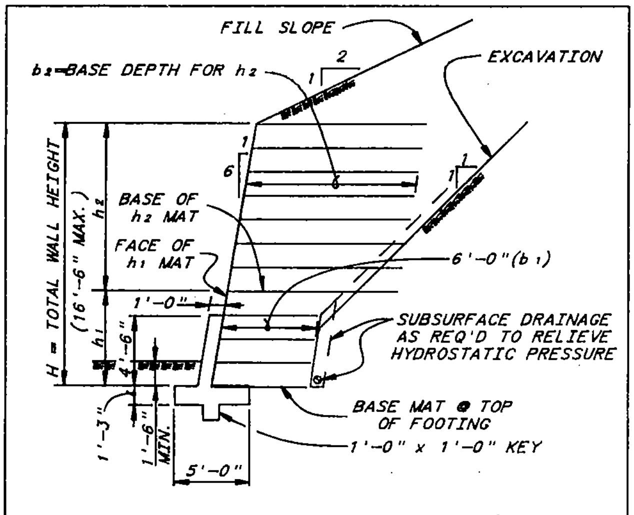


FIG. 12: TELLURIDE WALL w/CONC. KEY – TYP. SECTION

Design of the walls for external stability did not differ from those procedures set forth earlier and need not be discussed further. However, due to the use of cohesive backfill, analysis of the internal stability of the walls required three significant departures from standard design procedures.

FIG.13: TELLURIDE WALL

The first area of concern related to the selection of the correct coefficient of lateral earth pressure (K) for internal stability. Anderson et.al. (1985) State:

> "As shown on Figures 12-18 for wall heights (above the mats) up to about 10 feet, the values of K range between about 0.6 and 0.8. This range of K values corresponds to the at-rest condition for sand compacted in layers. For all heights above about 10 feet, the K values appear to decrease....
>
> Figure 26 strongly suggests that for wall heights greater than about 20 feet, the value of K is much less than the value of 0.65 suggested by Bishop and Anderson (1979). The envelope of the K values shown on Figure 26 shows that for wall heights greater than 20 feet, a K value of about 0.45 should be adequate for design. For walls less than 20 feet high, at-rest values of K in the range of 0.6 to 0.8 is appropriate for design."

Because of the characteristics of the project backfill material, a conservative K value of 0.80, non-variable with depth, was selected for use in design. This is in contrast to the K values which vary with depth from 0.65 to 0.45, typically used with cohesionless backfill.

Secondly, the empirical equations currently used to model sliding shear resistance between soil and reinforcements (pullout) are based on cohesionless backfill. However, in the work of Nielsen and Anderson (1983), laboratory pullout test results were compared to theoretical values produced by the Terzaghi-Buisman bearing capacity equation. By treating each transverse wire of the soil reinforcing mat as an individual footing bearing on the backfill, it was demonstrated that pullout resistance could be accurately modeled as:

$$Fp = Nt \cdot dt \cdot (Nc \cdot C + Nq \cdot \sigma v) \qquad (4)$$

Where: Fp = Pullout Resistance, lbs.
Nt = No. of transverse wires behind the failure plane
dt = Diameter of transverse wires, ft.
Nc, Nq = Bearing capacity factors
C = Cohesion, psf
σv = Vertical overburden pressure, psf

Because the design approach is considered experimental, a factor of safety in pullout of 3.0 was selected for each reinforcing mat, based on equations (1) and (4).

FIG.14: BAY AREA TELEPORT WALL

The third design departure was related to constructability and performance. In constructing a Welded Wire Wall, as each reinforcing mat is installed, the backfill immediately behind the face must be placed after the next mat above has been installed. This requires placing the backfill through the 6" x 9" WWF mesh openings. Compaction is limited to rodding and light, hand operated compaction equipment due to the flexible nature of the wall system's wire facing. Because this type of compaction effort is inappropriate for cohesive soils, a clean 3/4-inch rock was specified for use as backfill at the face of the wall (figure 15). This ensured that highly compressible pockets of uncompacted soil and the potential for substantial consolidation and serious distortion of the wall face were not built into the wall. In addition, the top two mats were extended 3'-0" beyond the lower mats to isolate any cracking which might occur at the back of the mats from the majority of the reinforced soil mass. In the event cracking did occur, due to consolidation or yielding, surface drainage would not be channeled directly to the wall backfill, possibly resulting in saturation and loss of strength.

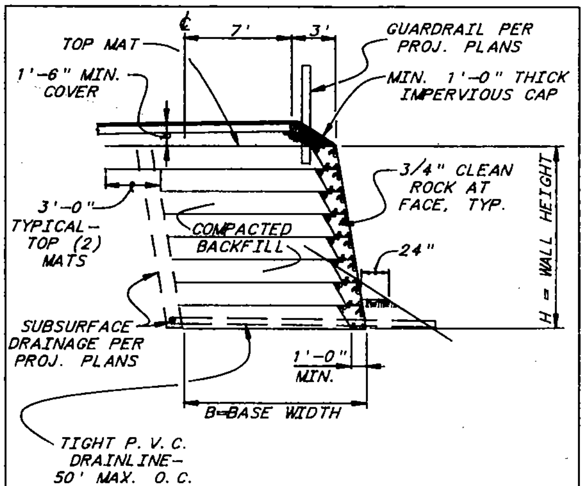


FIG. 15: BAY AREA TELEPORT TYPICAL WALL SECTION

CONCLUSION

It has been thoroughly demonstrated through research and project application that the Welded Wire Wall and other reinforced soil systems act as a coherent mass subject to external stability analysis similar to conventional gravity retaining walls. In addition, the interaction between backfill and soil reinforcings must be considered to ensure the internal stability of the reinforced soil mass. As has been discussed herein, the Welded Wire Wall can be readily adapted to a variety of unusual conditions created by varying external loadings, geometric constraints, and backfill properties. Further, in response to visual exposure, severe environmental conditions and structural considerations, the Welded Wire Wall has lead to the development of other grid reinforcing systems which offer a variety of wall facings: durable, pre-cast concrete panels; cast-in-place concrete, allowing architectural variations; and airblown mortar. As the relationships between soil and reinforcings continue to be better understood, the Welded Wire Wall and its companion systems are being applied to situations previously thought to be inappropriate.

About the Authors: John R. Selvage and K. Jeff Nelson are principals in the consulting engineering firm of Selvage, Heber, Nelson & Associates. Both registered civil engineers, they have been directly involved since 1979 with the Hilfiker Company and Utah State University in the development of the Welded Wire Wall, RSE, and Eureka Wall systems. To date they have designed more than 2 million square feet of these wall systems for projects throughout the United States.

REFERENCES

1. Bishop, Jerold A. and Loren R. Anderson. 1979 "Performance of a Welded Wire Retaining Wall." Report to The Hilfiker Company. Utah State University, Logan, Utah

2. Nelson, K. Jeff, 1985. "Design Procedures: Reinforces Soil Embankment and Welded Wire Walls." Report to The Hilfiker Company, Eureka, California

3. Anderson, Loren R., Kevin D. Sharp, B. Louise Woodward, and Rene F. Winward, 1985. "Performance of the Rainer Avenue Welded Wire Retaining Wall, Seattle, Washington - Draft." Report to The Hilfiker Company and Washington State Department of Transportation, Utah State University, Logan, Utah, and CH2M Hill Consulting Engineers, Seattle, Washington

4. McKittrick, David P. 1978 "Reinforced Earth : Application of Theory and Research to Practice." Keynote Address presented at the Symposium on Soil Reinforcing and Stabilising Techniques sponsored by the New South Wales Institute of Technology and the University of New South Wales, Sydney, Australia

5. Nielsen, Mark R. and Loren R. Anderson. 1984 "Pullout Resistance of Wire Mats Embedded in Soil." Report to The Hilfiker Company, Utah State University, Logan, Utah

6. Hannon, Joseph B. and Raymond A. Forsyth. 1984 "Performance of an Earthwork Reinforcement System Constructed with Low Quality Backfill." Report presented at the 63rd Annual Meeting of the Transportation Research Board, Washington, D.C., Sponsored by California Department of Transportation, Sacramento, California

7. Terratech, Inc., 1985 "Geologic and Geotechnical Investigation, Bay Area Teleport Facility, Niles Canyon, California." Report to the Fantozzi Company, San Jose, California

INTEGRATED SLOPE STABILITY ANALYSIS

USING

MICROCOMPUTERS

by

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INTRODUCTION

Prior to the general introduction of personal computers to the work environment, users had to rely on a central mini or mainframe computer that was strictly controlled by the data-processing personnel. The restricted access to such computers encouraged development of programs that could operate on a batch-basis and relied upon rigorously formatted input data. Often, there would be a substantial turn-around time period before the users obtained their output and were able to determine whether the computer run was successful. If errors were discovered, the cycle of data preparation, submission and retrieval was repeated until the problem was finally solved. However, personal computers have provided users with direct access to a CPU and they have discovered that computers are indeed very efficient and useful.

An exposure to a vast amount of personal computer software has resulted in higher user requirements and expectations. Unfortunately, much of the engineering software still reflects the mentality of the batch-processing environment. Many such programs fail to take advantage of a PC-environment and do not provide a user friendly interface, screen graphics, interactive error checking of data and help in interpreting the output.

This paper presents details about the philosophy behind the development of a typical user-friendly interface for a slope stability program, STABL, that was originally developed for mainframe computers at Purdue University (Siegel, 1975, Boutrup, 1977). Also, some guidelines are presented concerning the "minimum" features that a PC user expects from new software for engineering analysis.

THE USER-FRIENDLY INTERFACE

A "user-friendly" program is one that uses the computer hardware to improve the productivity of the program user by taking advantage of several human factors. For example, human memory and intelligence precludes providing repetitive simple instructions for every task; visual stimulation through screen graphics; and perhaps the use of "beeps" to warn of errors are typical cases of using human factor characteristics.

Generally, a user-friendly program should be easy to use for the non-familiar, provide natural instinctive guidance and trap unintentional errors. Once guidelines for a user-friendly program have been outlined, they will have to be modified, and re-modified, until a consistent interface is produced to meet the specifications. Often limits are imposed on the "friendliness", as many pages of computer code may be required to achieve minor objectives. In the end, a compromise is reached between the perceived requirements and the capability of the program development team to provide a product that combines efficiency of code with compactness and consistency.

Although there are no rigid guidelines for development of reliable interfaces, the following principles should be addressed (Hippolyte, 1986) :

1. A program is part of a larger system,
2. Virtually all user input to a program is initiated by output from the program,
3. The interactions of a program fall into two groups: those that concern the state of the program, and those that represent some portion of the data.

The program will always be part of a larger system and will function within the limitations imposed by operating software and hardware. For example the program may operate under the "umbrella" of the disk operating system (DOS) in case of personal computers. The interface will logically influence the user's behavior via the program's behavior. Hippolyte (1986) considers this essential as "the user cannot be expected to behave in a proper way". By following the final principle, the user is kept informed about the current status, i.e. is more data to be entered or whether data manipulations are being performed in the background. This ensures that the user is not left wondering about the activities of the computer. Consistency is incorporated into the program to ensure that portions of the program do not contradict each other and subsequently confuse the user. If a menu-based interface is used, all menus <u>should</u> operate in similar ways treating any errors and providing useful help in an appropriate fashion without any ambiguities.

All these tasks have to anticipated for a successful user-friendly interface. Ideally, the person in charge of the development should be very familiar with the analytical program, be aware of the program's limitations and be capable of "knowing" the potential users. This knowledge about end-users is essential for the development of successful and marketable products. It is also the most difficult aspect of software development.

PC-DOS or MS-DOS Environment

There are many operating systems available for use with desktop personal computers with the most prevalent being the one developed by Microsoft Corporation and available as either IBM PC-DOS or MS-DOS. These two are very similar and the minor differences are just special enhancements made by the computer manufacturer to take advantage of unique hardware features. The items discussed in this paper concern these dominant operating systems and will be subsequently referred to as the "DOS" environment.

Under the DOS environment, software should be capable of providing the following "basic" features :

1. Rapid installation of software,
2. Interactive data entry,
3. Full screen editing of previously entered data,
4. Full screen graphics support for checking input data,
5. DOS file manipulation (e.g. renaming, copying, deleting) without exiting the main program,

With the above features in mind, the slope stability program, STABL, was used as a prototype model for possible conversion to a "truly" PC-based user-friendly program. Such a program would completely integrate the data preparation and checking phases with the actual slope stability analysis. Details of the prototype program that has been developed using the DOS environment and the pertinent characteristics of human-computer interaction are presented below.

STABL INTERFACE

The first prototype of the STABL user-friendly interface was developed to aid unfamiliar users in the data preparation phase only (Sharma, 1988). This early version was developed as a substitute for a full-screen editor and did provide an instinctive interface and screen graphics to aid the user in assembling a potentially error-free input data file. Subsequently, the program was modified through several iterations and finally combined with the main STABL program.

The current prototype, XSTABL ver. 1.2, now forms an integrated program that provides many screen graphics options and also permits the user to generate "report-quality" plots on IBM graphics compatible printers, HP LaserJet II or the HP Plotter (see Figures A1 - A3 in the Appendix). Thus the potential user can create or edit the input data, perform an analysis and finally print the slope profile and critical failure surface on a conventional dot matrix printer. The remaining part of this paper discusses a few of the unique features of XSTABL and shows the extent of potential improvements that can be made to mainframe computer programs ported to a DOS environment.

DESCRIPTION OF XSTABL

XSTABL consists of a menu driven interface that provides the user with several options, that are selected by moving the highlight bar to the required option and pressing the "ENTER" key to make the appropriate selection. This menu approach of option/selection has been shown to be the most successful feature in PC-based programs, and so has been adopted for use in XSTABL. The program consists of two major parts that are connected by the main menu, as illustrated in Figure 1.

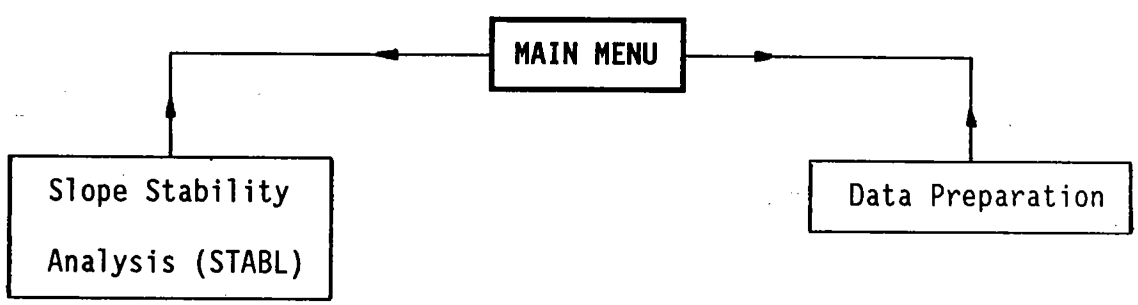

Figure 1, Overall Structure of XSTABL

The interface consists of a series of "menus" and "tables". There are two main menus and five sub-menus where the user is asked to make a selection out of the displayed options. The tables are used for data entry where the user provides "real" information, such as slope geometry or soil properties, to the program.

Main Menu

The main menu is illustrated in Figure 2, below. This is the first menu that a user will encounter upon executing XSTABL. From this menu, one can select any of the tasks listed by moving the highlighted bar to the appropriate option and pressing the ENTER key. In this particular menu the option: "ENTER OR EDIT DATA" is currently highlighted, and ready for possible selection. Also, a short description of each highlighted option is provided below the menu-box for the convenience of the user.

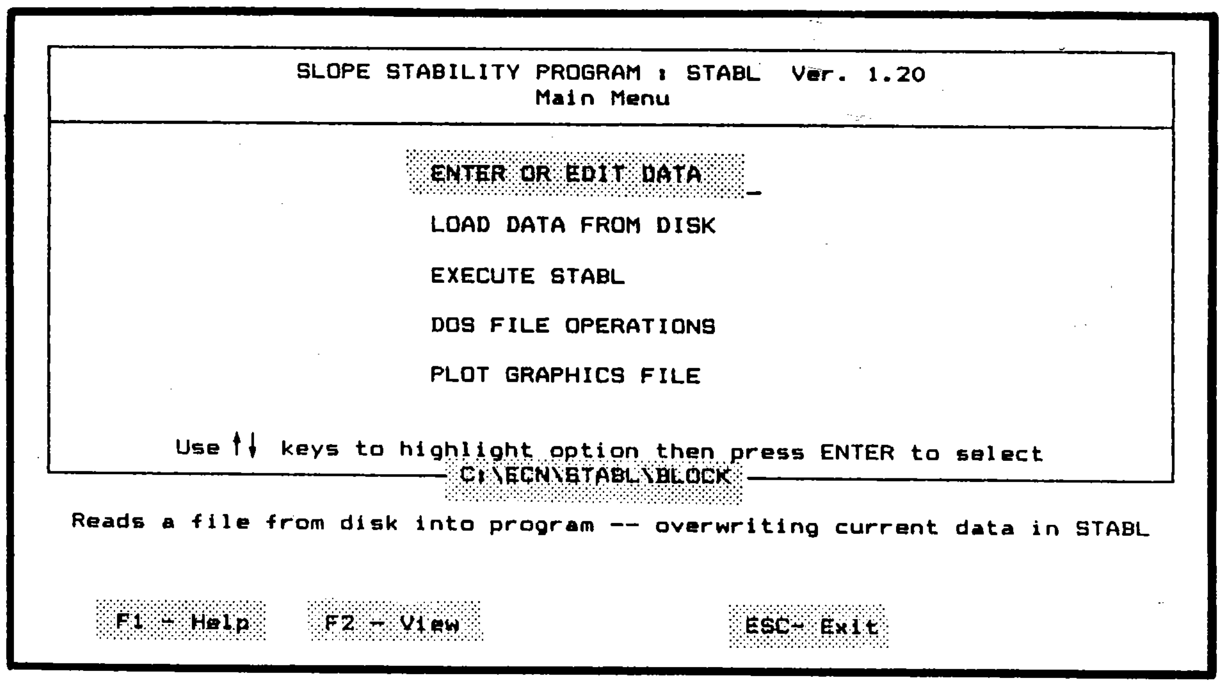

Figure 2, The Main Menu in XSTABL

The menu has been loaded with an existing data file: "C:\ECN\STABL\BLOCK", and this filename is displayed in the center of the lower menu-box boundary in Figure 2. Messages that provide additional services are placed in the lower portion of the screen. In figure 2, these messages indicate that (1) Help is available by pressing F1, (2) The input slope geometry may be Viewed by pressing F2, and (3) Press ESC to Exit from menu. In reality, these messages provide an index to remind the user about these specific informational keys. Additional helpful features, such as a listing of filenames, are offered to the user according to the context of the information required by the selected option.

Enter or Edit Data -- MENU

If the user selects the first option from the main menu shown in Figure 2, the screen clears, and a new menu is written to the screen, as shown in Figure 3. This menu provides an interface to the many options that are available for performing the slope stability analysis. Most of the options are the same as those discussed by Siegel (1975). However, a few new options have been introduced to complement the user-friendly nature of XSTABL. Generally, this is the menu where the user selects an option and then proceeds to provide numeric input data that will be later used for the actual slope stability analysis. This menu requires the use of cursor keys to move both horizontally and vertically through the many available options.

Typically, it starts with the "PROFILE" option and lets the user select to provide input for either the surface or sub-surface boundaries. If the user presses the right cursor key, the "SOIL" option is highlighted and a new set of input data selections become available. The screen updates for these selections are very fast and so the user will not be inconvenienced by any delays. For the sake of brevity, Figure 3 illustrates all the available options and sub-options on one figure. The lowest portion of the figure does however show a complete screen for the last option: "LOADS/LIMITS".

Again, a short description of each currently highlighted option is presented below the menu-box. The filename of the current data-file and the index to the informational keys is also included in this screen, just as in the main menu. By pressing the ESC key, the user may exit this menu and return to the main menu. During this phase, the user will be prompted for a

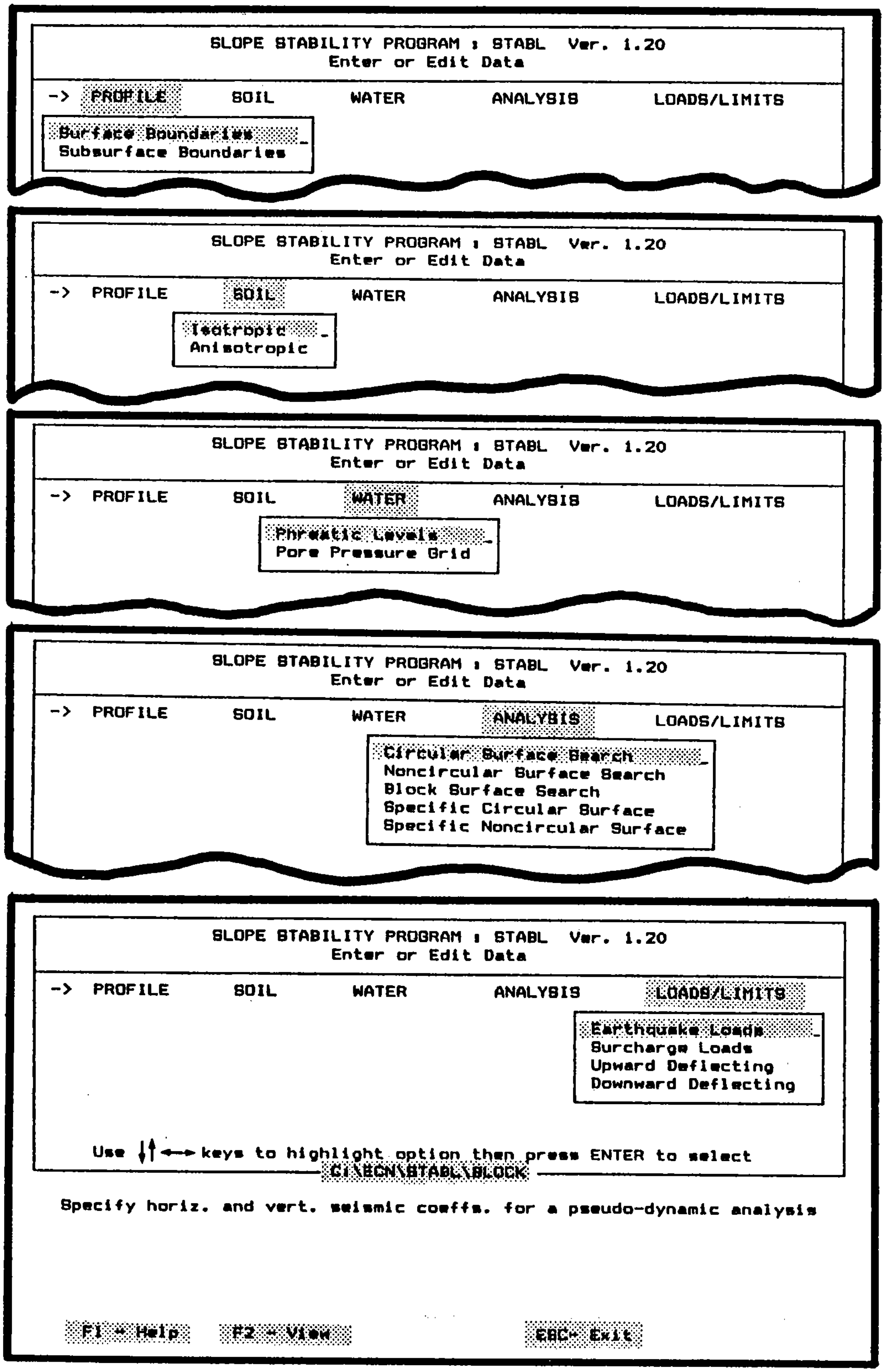

Figure 3, Different options that may be selected from the "Enter or Edit" menu.

filename. Any entered data will then be written to assigned disk filename.

If any of the options are selected in this particular menu, the user is then prompted for numeric data for developing an input file for use with the slope stability analysis portion of XSTABL. For example, if the user selects the "Isotropic" option (under the title SOIL in Fig. 5), the screen clears and a table is presented to the user, as shown in Figure 4. This particular table has a title (second-line) of: "Soil Property Data". Before printing this table, some data had already been entered, as shown.

Data can be entered into these tables and subsequently edited by using the cursor control keys and several other commands that allow more rapid movements through large amounts of data. Also, new lines may be inserted, or lines may be deleted from these tables. In fact, each table is very similar to a full-screen editor but does provide "intelligent" data entry. For the other options, similar tables are constructed on the screen to allow the user to enter the appropriate data. These tables (almost) preclude the use of a "User-Manual" due to descriptive table headings and instinctive sequences of data entry.

PROGRAM STABL - INPUT ROUTINE
Soil Property Data

SOIL TYPE	WEIGHT MOIST (PCF)	WEIGHT SAT (PCF)	STRENGTH C (PSF)	STRENGTH Ø (DEG)	PORE PRESSURE Ru	PORE PRESSURE CONST. (PSF)	WATER SURFACE INDEX
1	116.4	124.2	500.0	14.00	0.000	0.0	1
2	116.4	116.4	0.0	0.00	0.000	0.0	1

Editing

F1 - Help   ESC- Exit

Figure 4, Table for entering Soil Property Data

SUMMARY AND CONCLUSIONS

A user-friendly interface has been developed for the slope stability analysis program: STABL. The interface allows the user to create, edit and review input data before performing the slope stability analysis. This combination of data preparation and analysis should provide a more efficient environment for slope stability assessment using a personal computer. Also, the opportunity to view the geometry of the slope on the screen is a tremendous help in reducing input errors and improving the productivity of the geotechnical engineer.

The development of the user interface for the STABL program has shown that "user-friendly" programs can be created for the personal computer environment. In the future, we can expect that many other popular geotechnical programs will also be converted to "user-friendly" versions.

REFERENCES

Boutrup. E. (1977), Computerized Slope Stability Analysis for Indiana Highways, Volumes 1 and 2, Report JHRP-77-25,26, Purdue University, West Lafayette, Indiana.

Hippolyte, M.S. (1986), Designing the User Interface, in Applied C, edited by Bonnie Derman, Van Nostrand Reinhold Co., New York, Chapter 2.

Sharma, S. (1988), STBLIN : An Interactive Data Preparation Program for Slope Stability Program STABL, Idaho Transportation Dept., Boise, Idaho, Contract 128 - K028, October, 34 pages.

Siegel, R.A. (1975), STABL User Manual, Report JHRP-75-9, Purdue University, West Lafayette, Indiana.

APPENDIX

The XSTABL program is available from the author and requires the following hardware :

1. An IBM-PC, or near compatible, with either two floppy disk drives or a hard disk (preferred) and one floppy disk drive and at least 640 kb RAM using DOS 2.0 or later version.

2. A floating point coprocessor (80x87) must be installed in the computer!

3. An EGA or CGA graphics adaptor with an appropriate monitor.

4. An IBM graphics printer, HP LaserJet II or a HP Plotter for output.

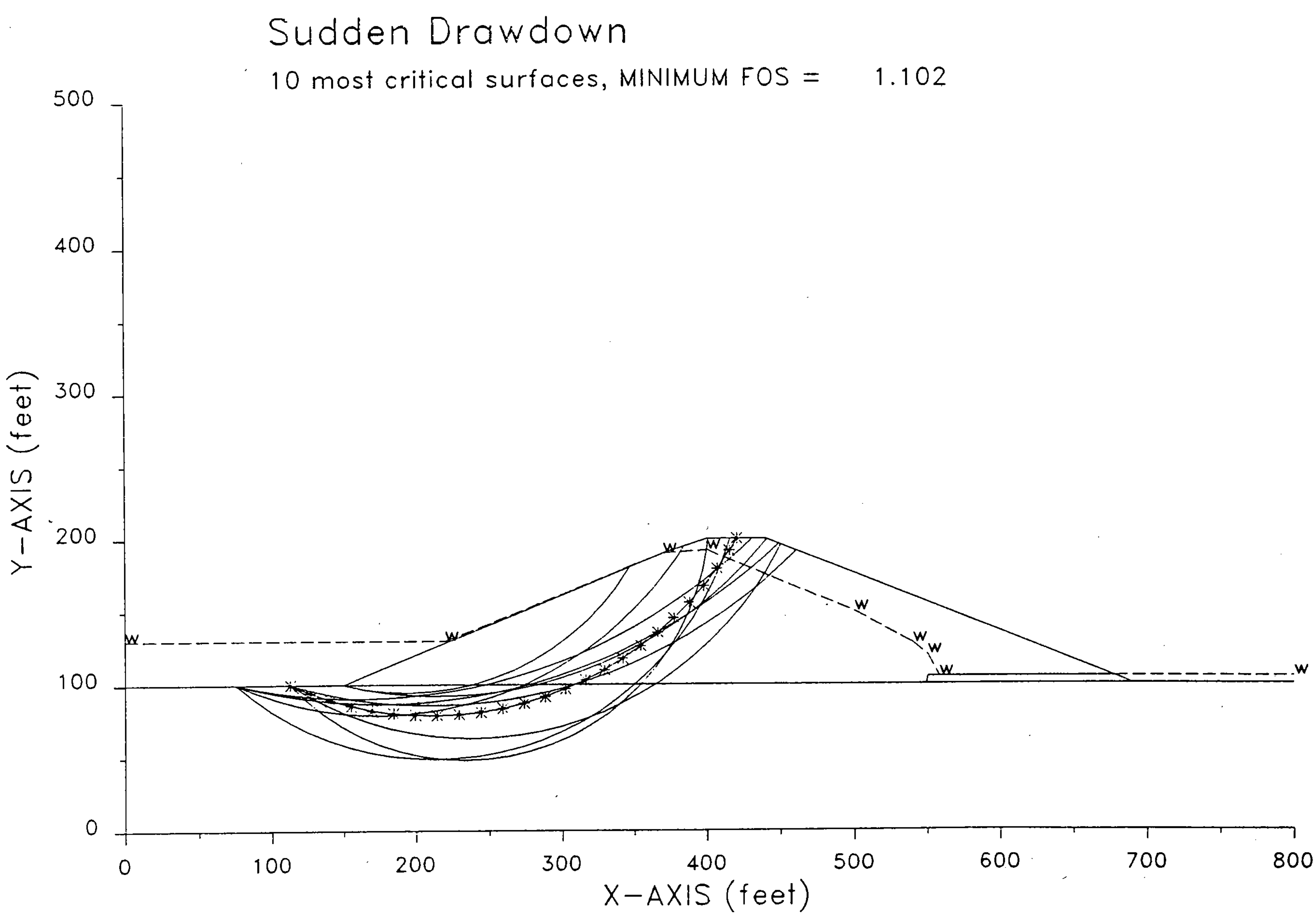

Figure A1, XSTABL output plotted on a HP LaserJet II printer in about 4 minutes

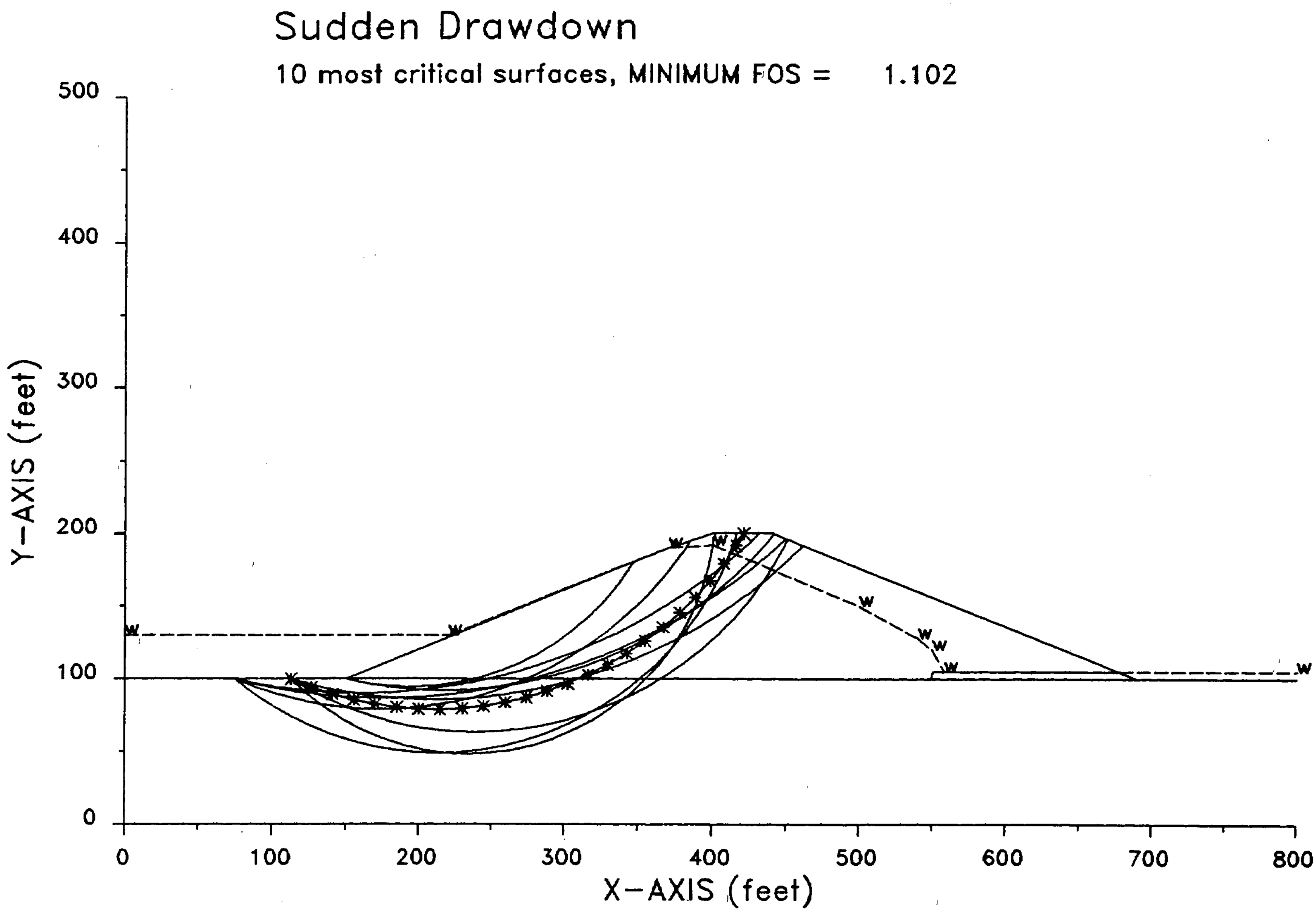

Figure A2, XSTABL output plotted at HIGH resolution on an EPSON FX-85 printer in about 18 minutes

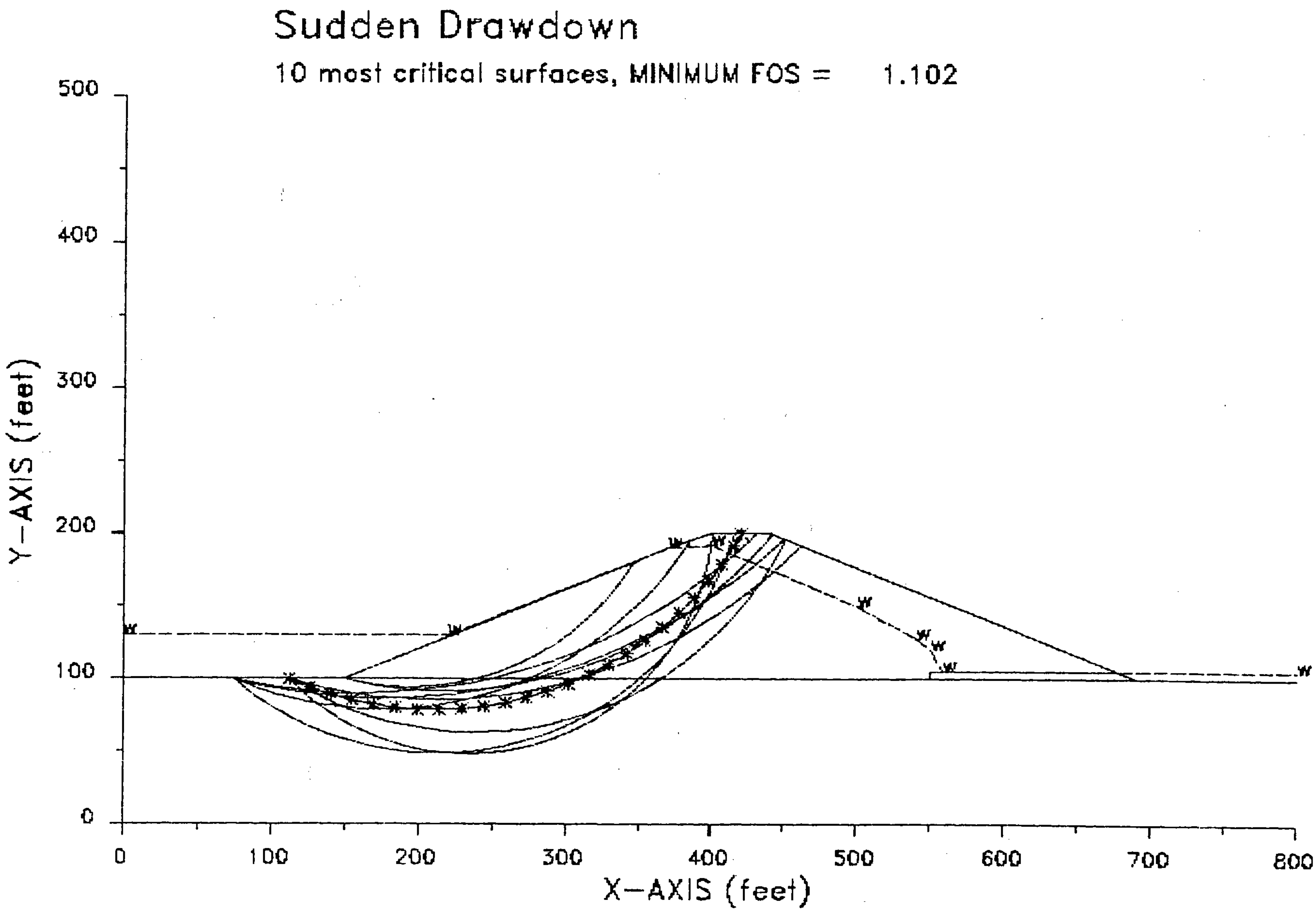

Figure A3, XSTABL output plotted at LOW resolution on an EPSON FX-85 printer in about 4 minutes

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"HONDURAS HIGHWAY GEOLOGY"

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PRESENTED AT
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TABLE OF CONTENTS

		No.
I)-	I N T R O D U C T I O N	1
II)-	HONDURAS' GOVERNMENT	2
III)-	GEOLOGIC AND STRUCTURAL PROVINCES OF CENTRAL AMERICA	4
IV)-	STRATIGRAPHIC COLUMN	4
V)-	HIGHWAY SYSTEM	8
VI)-	C O N C L U S I O N S	9

I. INTRODUCTION

Honduras' Geology has been little understood in the past. The Honduras' Government has no geological agency as such. However, the Instituto Geográfico Nacional, our Mapping Agency is in charge of only printing the geological quadrangles at a scale of 1:50,000. The country has been divided in 291 quadrangles of 15' X 10' minutes equal to aprox. 486 Km^2., from which only 20 quadrangles have been printed as geological maps. Most of there geological maps have been produced by U.S. Students within their Masters or ph.D. thesis work. The great mayority of young researchers came from the University of Texas at Austin during late 60's y early 70's.

This land has been boosted as a minning country, however, little was done by the minning companies to map geological terreins.

Geological mapping was mostly centered along the Tegucigalpa-North Coast main highway, known as CA-5. Perhaps, this was done due to the fact that this Highway was been built then and it offered better mobility from one place to another.

In the middle '70 there was a boom in highway construction. Very little and perhaps not at all, geological Studies were done during the highway planning Stage and durin the construction. Thus, Honduras highways are marked by any thinkable failures, that had to do with no geological planning during the construction Stage. On the other hand, if it were not because of these highways, we as geologist, would not have the oportunity to Study important outcrops cut by the highways from which we enhanced our Stratigraphic Column idea. Even though our Civil Engineers and construction people showed very little interest during the planning and construction Stage to use the geological concept in road design, they are now very eager to use the geological model to perform highway repairs and maintenance work after many geotecnical failures.

II. HONDURAS' GOVERNMENT

The Honduras Government thru its Ministry of Comunication, Public Works and Transportation (SECOPT) under the Direction of Ing. ALEJANDRO CASTRO RUIZ, has been encouraging it various practical Bureaus such as Highways; Highways and airports Maintenance; Public Works and Urban planning, to develop geological techniques to aid them in the concept of design, construction and maintenance of public works. See Figure No. 1.

An intricate inconvenience appears due to the fact that only fifteen Hondurean geologists exist. Most of them are hired by the National Electric Company (ENEE) on their Hidrological proyects. Two geologists work with the Minning Bureau of the Ministry of Natural Resources. Only one geologist performs related duties and research at SECOPT. We are in a great need for practical and experienced professionals to aid us in this particular technical area such as applied Highway Geology.

The Highway and Airports Maintenance Bureau, Directed by Ing. ANGEL NUÑEZ CERRITOS, has recently created the Geological Investigations

Department (DIG), which has undertaken the Technical goal pursued by the Ministry of S.E.C.O.P.T.

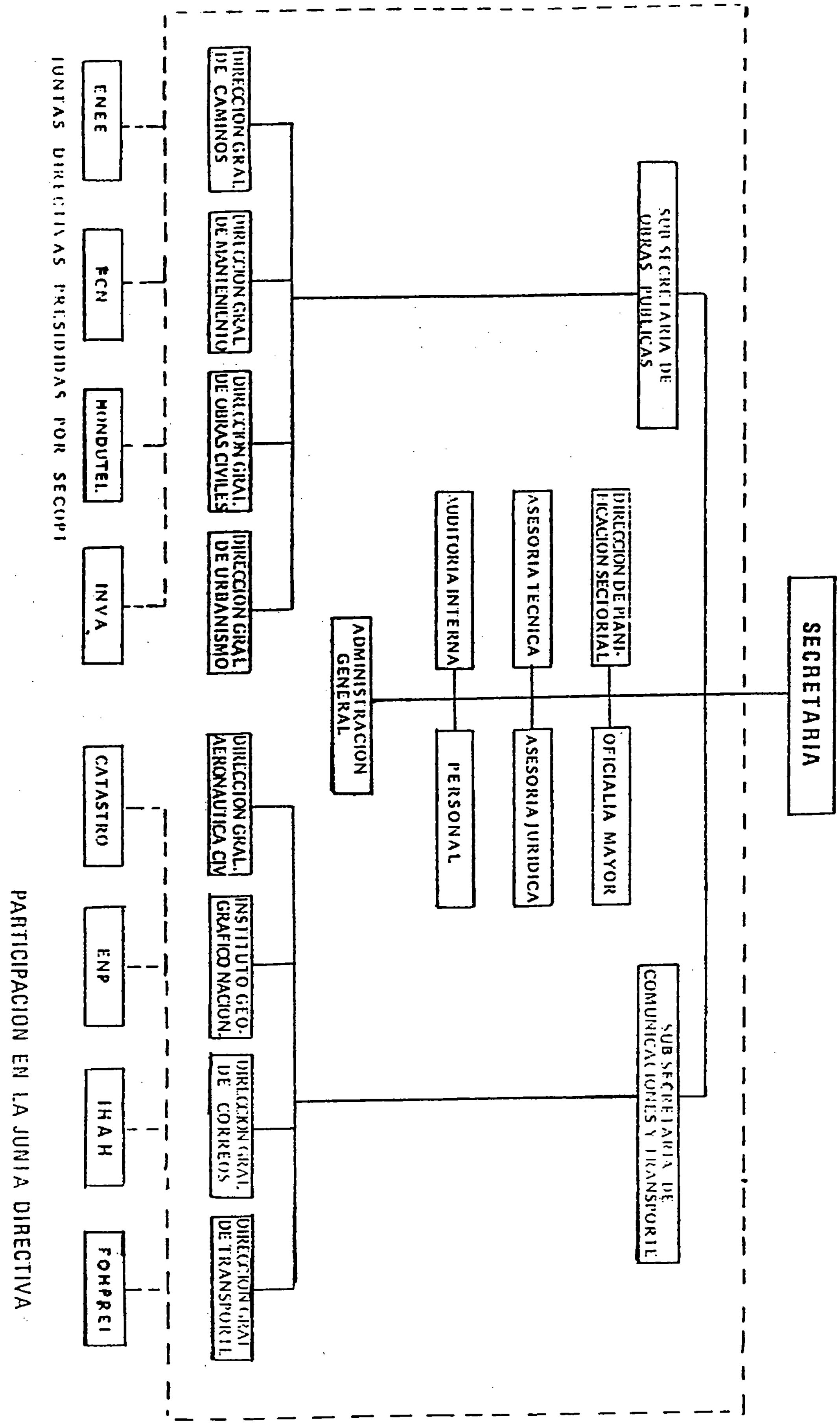
The objective of D.I.G. is to elaborate the diagnostics, analysis and/or geologic studies that refer to problems or failures in our highway system and public infraestructure works. The purpose of D.I.G. is to give technical assistance within the geological sciences applied to Civil Engenering with the aim that the various sections of the Bureau can give practical and true Technical data for a better Highway maintenance and the repair of Highway and bridge failures.

See Figure No. 1

REPUBLICA DE HONDURAS

SECRETARIA DE COMUNICACIONES, OBRAS PUBLICAS Y TRANSPORTE

ORGANIGRAMA 1987



III. GEOLOGIC AND STRUCTURAL PROVINCES OF CENTRAL AMERICA

Honduras' Geology will be briefly discussed, See Figure No. 2. The country is politically located in Central America. In our geologic world the country is within the Caribbean plate and bordered to the north with a Late Cretaceous-Early Cenozoic thrust-fold belt and the Motagua-Polochic Fault zone. Descriptions are made southwards of the Motagua fault zone were we find the nuclear axis and a site of cretaceos (?) Granite Batholits Known as morphotectonic unit of the Northern cordillera. The early cretaceous folded carbonate shelf is known as the Central Cordillera. It in turn is flanked by a late cretaceous batholith axis known as the Southern cordillera.
Southwards we find the Cenozoic pacific volcanic chain known as the morphotectonic unit of volcanic ranges and plateaus, the pacific volcanic chain and the pacific coastal plain. These provinces are then followed by the axis of the America Basin in the Pacific Ocean.

IV. STRATIGRAPHIC COLUMN: See Figure No. 3.

The Paleozoic is marked by undifferentiated age, by metamorphosed Sedimentary and Igneous rocks such as sericitic and graphite schists, philites, marble, meta-sediments and granitic plutons, kown as Cacaguapa shists. An unconformity and an erosive period tops The paleozoic rocks. The Mesozoic age has been marked by continental erosion and deposition coupled with marine transgression and regression. The triassic is represented by thin bedded, gradational beds of clastic and carbonate rocks of a floodplain or shallow marine enviroment. The Jurassic rocks are - mostly conglomerates, red beds (shales and siltstones) arkosic sandstones granitic plutons, originated probably from alluvial fans, fluvial, lacustrine, sanddunes, deltas and swamp enviroments. Both Triassic and Jurassic up to the middle lower Cretaceous is known as the Honduras Group. An unconformity lays on top of Jurassic times. Marine transgression marks most of the Cretaceous were the thick bedded limestones of the Yojoa Group of Aptian-Albian age from a Back reef enviroment (Cantarranas fm.) and shallow marine enviroment of the Atima Formation.
The upper Cretaceous is marked by continental, marine and Lacustrine - enviroment where Valle de Angeles Group was deposited consisting of red beds such as mudstones, shales, siltstones and sandstones; these are our most problematic rocks for perfect "mass movements" due to their lithology and the terrain morphology that has been formed by them. During Cenomanian times, a marine tectonic deposition formed the Jaitique and Esquias Formations in the middle of the Valle de Angeles Group and they consist in thin bedded to massive biomicritic limestones, limy claystones and siltstones and the Jaitique fm, is locally capped by a gypsum lithosome.

Some granitic plutonism appeared through-out uppercretaceous times and a tectonic uplift is marked by an erosive period and an unconformity. The Valle de Angeles Group covers middle Honduras from West to East along the early cretaceous and folded carbonate shelf.

The Cenozoic age is represented by intense vulcanism that laid extrusive rocks such basic lava flows of the Matagalpa formation during late Eocene-early Oligocene times. Uplift and erosion marked most of Paleocene-late Oligocene and early Miocene times. Another intense tectonic uplift during Miocene-Lower Pliocene times and related vulcanism laid extrusive alkaline flows, tuffs and ignimbrites of the Padre MIGUEL - Group. The Quaternary age is marked mostly by tectonic uplift and strike slip movements parallel to the Motagua-Polochic fault zone and the Guayape fault zone; this event deposited alluvium, coluvium, basaltic and riolitic flows. Tertiary age is represented, partly by the southern Cordillera and the volcanic ranges and plateaus morphotectonic unit.

The recent age is represented by alluvium, coluvium, landslide debris of the Gracias fm. of fluvial and floodplain deposits.

GEOLOGIC & STRUCTURAL PROVINCES OF CENTRAL AMERICA

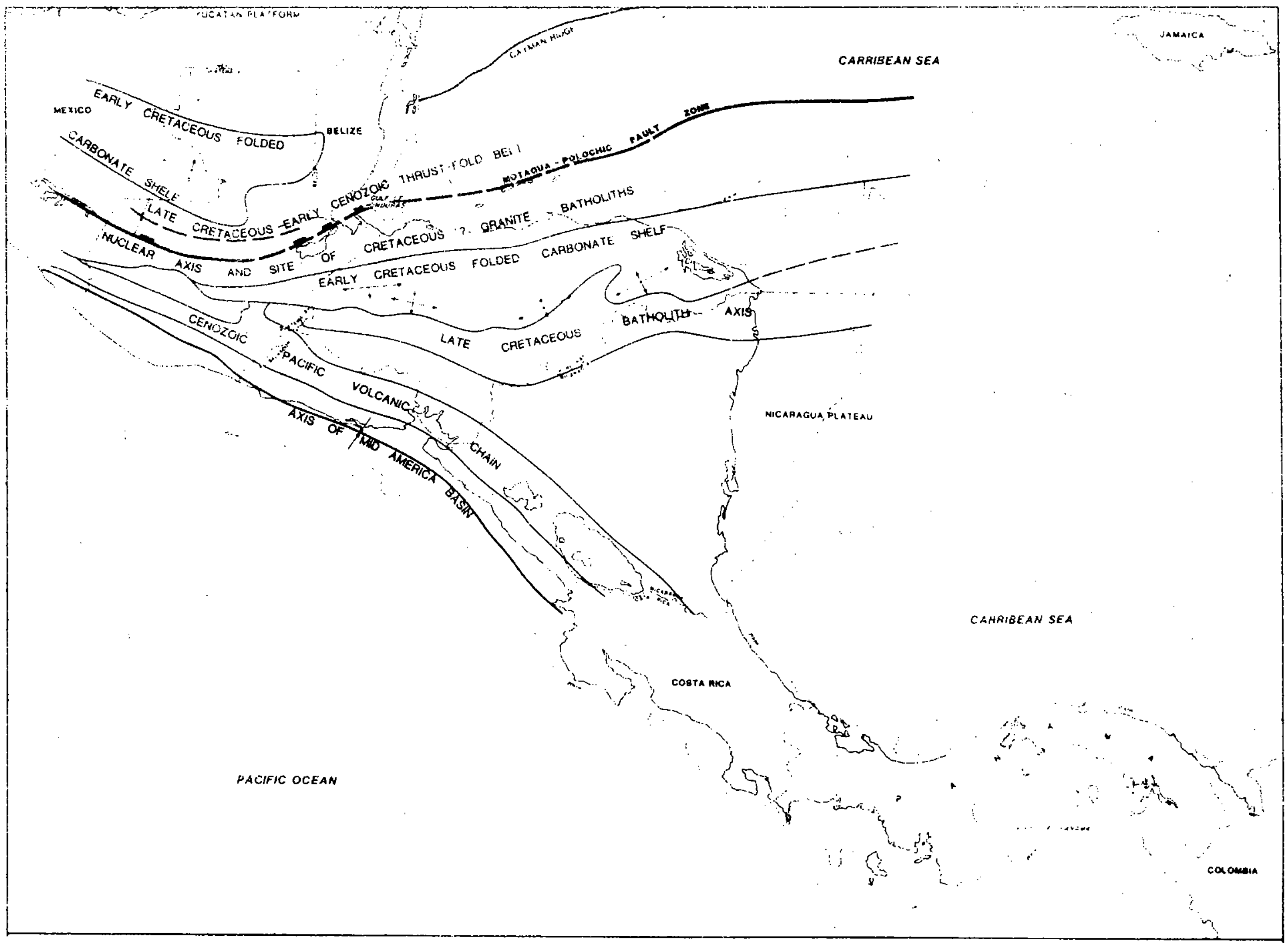
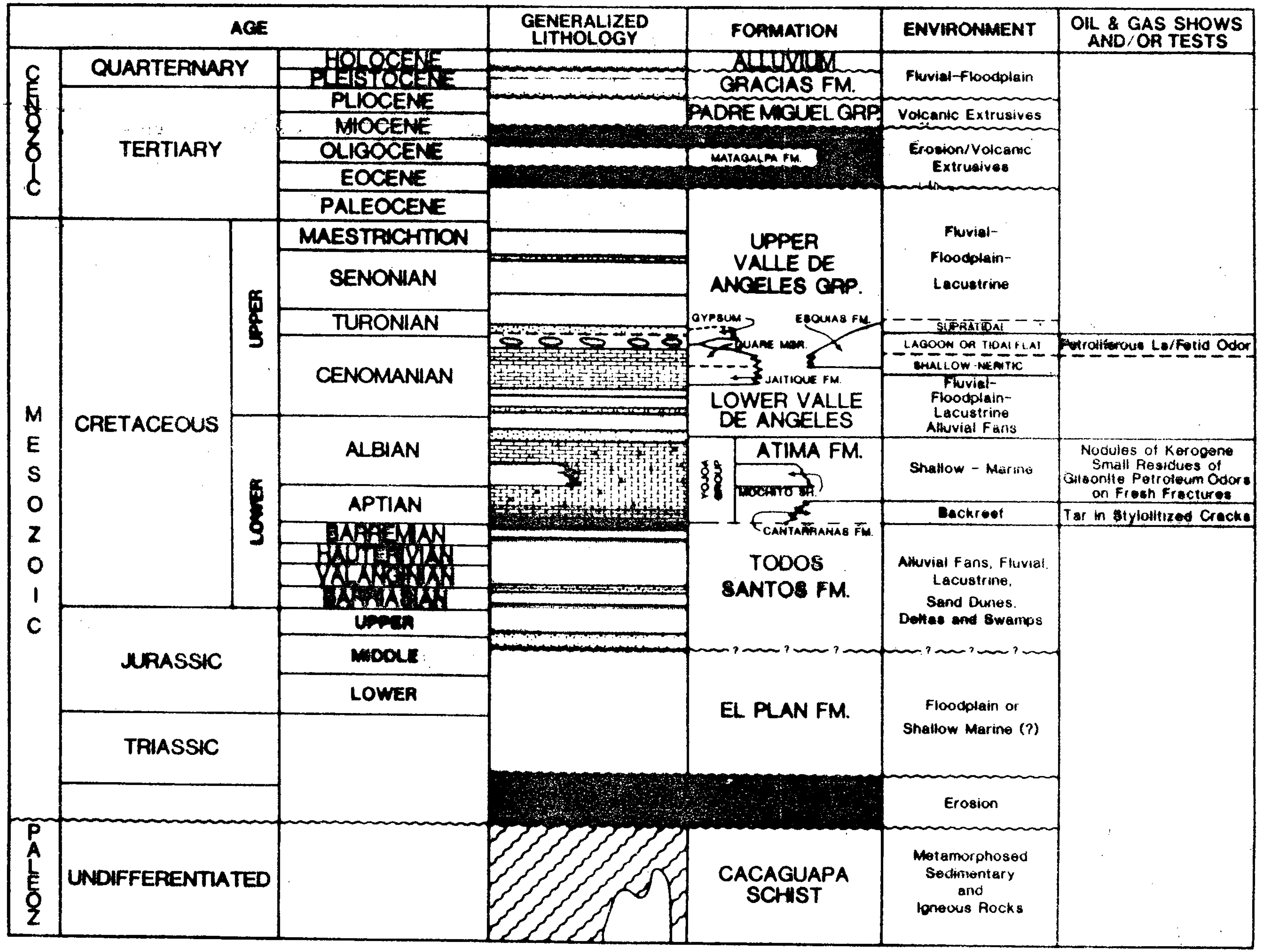


figure No. 3

GENERALIZED STRATIGRAPHIC COLUMN - HONDURAS



V. HIGHWAY SYSTEM

The Bureau of Highway and Airport Maintenance has divided the country in eight (8) Districts. Each one has the responsability of maintaining and repairing the highway within them. Besides the Districts, the Bureau has three (3) divisions of Asphalt which provides the necesary materials to repair paved roads, See Figure No. 4.

In the North and South zones of the country, the Bureau has one pro---- cessing plant of asphalt aggregate concrete and a trituration plant, respectively. The following map shows the territorial distribution of each District. As we con see, each District has been spatially arranged around political territorial division. Thus, some Districts have much work having to do with landslide, fill colapses and related mass movements. For example, the western highway CA-4 goes from District Nº 5 to District 7 and the geotecnical highway failures appear in red beds further away from the main office; therefore, this highway failures are not easily repaired, CA-4 was built between 1969 and 1971, and it is heavily damaged.

It is a task undertaken by D.I.G. to survey the influence of geologic related phenomena and produce highway geologic maps applied to the design, maintenance and repairment of the Highways.

A specific highway being actually built right now on The South-West fringe of District Nº 5, has presented many landslides due to the red beds and mudstones. The highway designs, plans for construction did not take into consideration lithological aspects at all. The specifications for slope gradient not to exceed 6% has constructed deep cuts (more than 20-40 meters), thus, they have created man made mass movements. This particular Highway from Santa Barbara to Pito Solo, near lake Yojoa was design only using the topographic map and did not consider the geologic map already published which shows the three big landslides that The Highway went trough. In one of them, the Highway cuts the toe of a landslide and more than 180,000 m^3 of loose material has been removed; now the Highway was - moved 30 meters side ways and into a soft silty clay strata.

Most of our Highway failures are caused by underground water, which saturates the silty well compacted base and sub-base and creates fill colapses and fill slides.

The heavy rainfall and deforestation has created large runoff which turn forms mud and silt flows. These sediments in turn fill the lateral highway drainage systems and some cases the culvert also fails.

Another serious problem is the heavy erosion of the surface of highways that are not yet paved. The Highway Supervision only makes sure that the road design being built by the Contractor is within the specifications. There-for , no provisions are made for cut slopes and fill slopes to protect them from erosion.

The north Highway system have many problems related to erosion of bridge foundations. The Highway was located in a flood plain created by aluvial fans from high velocity rivers that come out from the mountains. The Annual meandering of these small current rivers have heavily damaged the bridge foundations and in most cases they colapse.

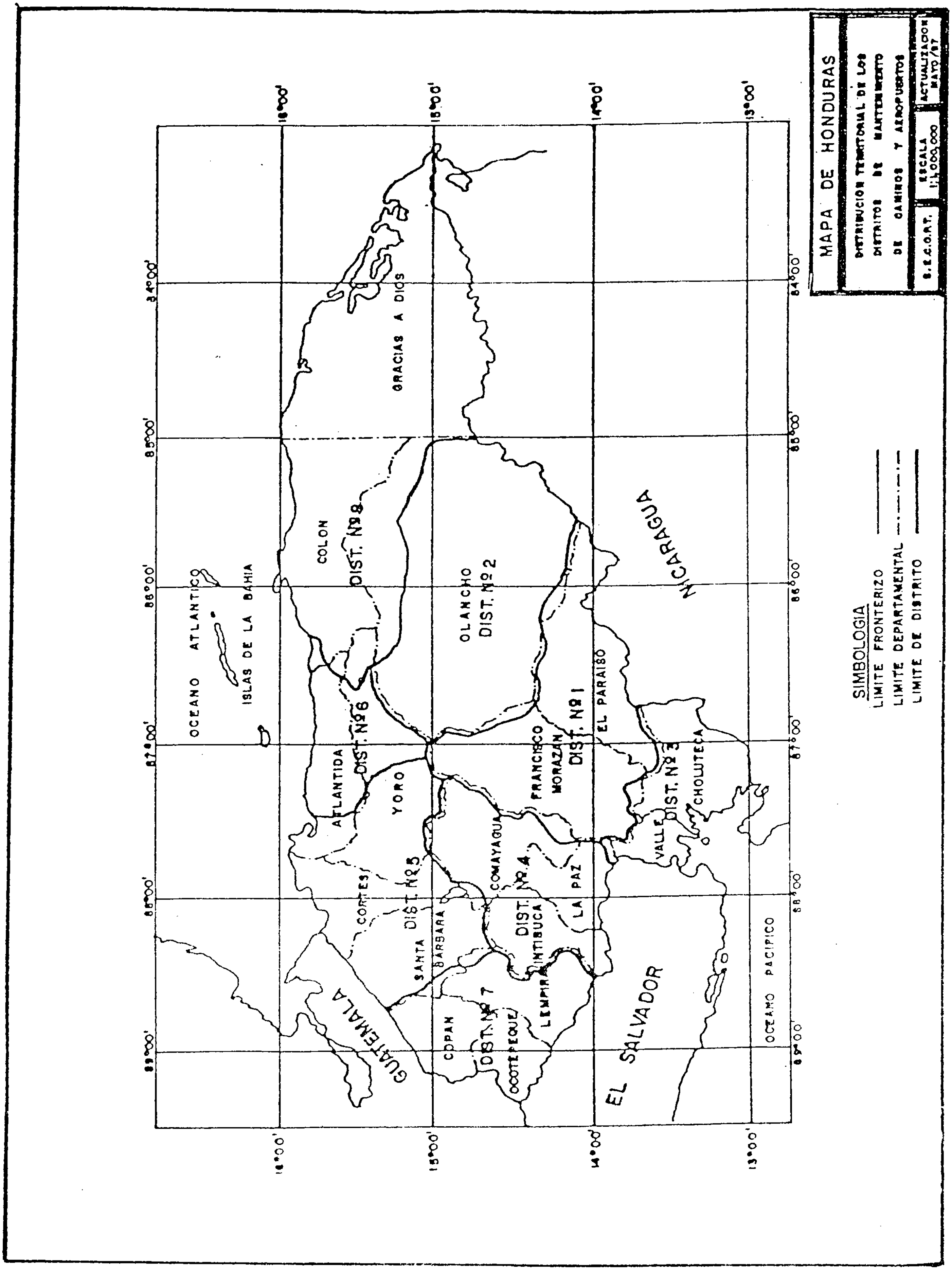
District Nº 2 highway system have problems in rupture of the paved surface and heavy culvert destruction. In november, 1979, the the road was cut due to the culvert obstruction, ten people died when their Bus fell into the crevase.

The volcanic tuffs and ignimbrites are far more better construction rocks. However, within the volcanic event some ash fell in water and has been poorly consolidated, this creates silty mud flows, rock fall and coluvium slides.
Therefore, failures such as landslides, fill and slope colapses, erosion of bridge foundation, bridge colapses, pavement destruction and almost any thinkable geotechnical highway failure related to geomorfological, lithological and geological aspects are represented in the Honduras' Highway system.

VI.- CONCLUSIONS

The Honduras' Highway system has a total net of 9,602.0 Km (1989), Under the responsability of the Bureau of Highway and Airport Maintenance. Other Highway such as the ones being built by the Highway Bureau, Forest Service, Natural Resources and Agriculture, Coffee Farmers, Rural roads and hand labor roads are not included in the SECOPT maintenance program. The Highway system is being expanded and we have a great need for experience and expertise in applied geological techniques for highway maintenance and repair.
Only with a qualified Highway system can make a country develop itself.

Figure No. 4


MAPA DE HONDURAS
DISTRIBUCION TERRITORIAL DE LOS DISTRITOS DE MANTENIMIENTO DE CAMINOS Y AEROPUERTOS
S.E.C.O.P.T.
ESCALA 1:1,000,000
ACTUALIZACION MAYO/87
SIMBOLOGIA
LIMITE FRONTERIZO
LIMITE DEPARTAMENTAL
LIMITE DE DISTRITO
OCEANO ATLANTICO
ISLAS DE LA BAHIA
GRACIAS A DIOS
COLON
DIST. Nº 8
OLANCHO
DIST. Nº 2
NICARAGUA
EL PARAISO
DIST. Nº 1
FRANCISCO MORAZAN
CHOLUTECA
DIST. Nº 3
VALLE
LA PAZ
COMAYAGUA
DIST. Nº 4
INTIBUCA
LEMPIRA
DIST. Nº 7
COPAN
OCOTEPEQUE
SANTA BARBARA
CORTES
DIST. Nº 5
YORO
ATLANTIDA
DIST. Nº 6
GUATEMALA
EL SALVADOR
OCEANO PACIFICO
16°00'
15°00'
14°00'
13°00'
84°00'
85°00'
86°00'
87°00'
88°00'
89°00'

40th ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

BIRMINGHAM, ALABAMA MAY 17 TO 19, 1989

CONTRACT SPECIFICATION OPTIONS FOR RETAINING WALL DESIGN AND CONSTRUCTION: DISCUSSION OF CONSTRUCTION ALTERNATES AND CASE HISTORIES

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CONTRACT SPECIFICATION OPTIONS FOR RETAINING WALLS
JOHN R. WOLOSICK

ABSTRACT

This paper discusses contract specification options for anchored (tied-back) retaining walls and compares Closed Specifications with these options. Closed Specifications are associated with designs that are generated by highway departments or consultants in which the complete retaining wall structures, including the ground anchors, are designed. Contract Specification options discussed and compared to Closed Specifications include Open Specifications and Performance Specifications. In addition, contract specification features and details of two recent case histories of wall construction are presented and discussed. One of the cases was bid using Closed Specifications and the other was bid with a Modified Performance Specification.

INTRODUCTION

For many years, anchored or tied-back retaining walls were used for temporary earth retaining systems. However, for the last 15 years, these walls have been used successfully for permanent retaining walls. A permanently anchored retaining wall should be considered for use when a wall is required for a cut situation; the wall required is 15 feet or more in height; or where temporary shoring is required for the construction of a reinforced concrete cantilever wall or mechanically stabilized embankment wall. In these situations, a permanently anchored wall will usually be the most cost effective solution because "top-down" construction can be utilized. For those unfamiliar with anchored wall construction, Figures 1, 2, and 3 show a typical construction sequence for an anchored retaining wall.

Permanent ground anchor (tieback) design and construction are rapidly changing technologies. Since the first permanent anchor installation in the United States in 1969, tendon corrosion details have improved, drilling and installation techniques have improved, and anchor loads have increased. Also, the range of geologic strata that are considered suitable for anchorage has increased to include materials that until a few years ago were considered to be very difficult, if not impossible, to utilize.

Simultaneously, there has been a growth in awareness and confidence in the use of permanent ground anchors by the engineering community. Even though the techniques of anchor construction have improved considerably, certain aspects of anchor construction still remain an art and not a science. The long term performance of permanent anchors is directly related to installation procedures. Engineers and owners must recognize that anchor construction is an art form, that the technology is changing and growing rapidly, and that specialty contractors in this area have their own customized equipment and construction procedures.

Contract Specifications should be developed to take advantage of these realities. This is the basis for the construction alternate concept which allows qualified specialty contractors to provide design and construction services while still remaining in the competitive bidding arena.

CONTRACT SPECIFICATION OPTIONS

This paper discusses options to the Contract Specifications which allow the Owner to take full advantage of the construction alternate concept for anchored or tied-back retaining walls. Contract Specification options available to owners include:

1. Closed Specifications
2. Open Specifications
3. Performance Specifications
4. Modified Performance Specifications
    A. Pre-bid Design
    B. Post-bid Design

These types of Contract Specifications are described and compared below.

CLOSED SPECIFICATIONS

Highway departments or consultants in the United States sometimes design the complete anchored retaining wall structure including the ground anchors. In addition, they commonly specify the type of anchor, capacity, and the installation and testing procedures. The anchor contractor is only required to submit material certifications and install the anchors in accordance with the specifications. These types of specifications are known as Closed Specifications. Closed Specifications do not enable the experienced anchor contractor to utilize his design expertise or the construction methods best suited to his equipment. Closed Specifications also encourage unqualified contractors to bid on the project.

This type of specification does not guarantee low prices. In fact, higher prices, change orders, and delays often result when an incorrect anchor system or drilling method is specified. Closed Specifications should be utilized only in unusual circumstances where the engineer has reason to limit methods and technique. Ground anchor design and construction techniques are evolutionary in nature, and it is important not to stop innovation by using rigid specifications. The specifications cannot replace the professional experience and conscientiousness of the experienced contractor's personnel at all levels.

OPEN SPECIFICATIONS

Open Specifications permit the project scope, design, and construction to be performed by the specialty contractor. This method is most often utilized for securing bids on temporary ground anchor projects. The responsibility for the design, construction, and performance of the completed structure is placed solely on the specialty contractor. This method allows the contractor to select the most economical anchor system by utilizing his design and construction expertise and methods. Change orders are then kept to a minimum. Since Open Specifications place the total responsibility for the retaining wall on the specialty contractor, it is imperative that only experienced and responsible contractors be permitted to participate in the bidding process. A strict prequalification process is required to eliminate unqualified contractors.

PERFORMANCE SPECIFICATIONS

Performance Specifications provide for various amounts of the design to be performed by the anchor contractor and the owner's design engineer. The engineer typically establishes the scope of work and the design loadings as well as the type, size, and location of the proposed structure. The engineer also specifies the type of corrosion protection required, the anchor testing procedures, and any instrumentation or monitoring requirements. The method of constructing the tieback anchor, the tendon type, the sizing of the anchor bond length, and the location of the anchor are determined by the specialty contractor. This method allows the anchor contractor to provide an economical design while satisfying the engineer's design requirements. Performance Specifications allow the responsibility for the work to be shared between the design engineer and the anchor contractor. While this method is slightly more restrictive than Open Specifications, it is still flexible enough to allow for innovation and cost savings. The engineer maintains control over the design input while the contractor, within certain parameters, is free to vary the spacings, sizes, and types of various structural elements. This procedure results in a safe and economical design and construction process.

MODIFIED PERFORMANCE SPECIFICATIONS

Modified Performance Specifications are utilized in the construction alternate concept. The modification is associated with the time frame during which the design is performed. These modifications include Pre-bid designs and Post-bid designs.

PRE-BID DESIGNS

The contract documents for Pre-Bid designs are prepared to allow for various retaining wall alternates. The typical wall alternates can include standard cantilevered reinforced concrete retaining walls, gravity walls, or permanently anchored walls. The engineer provides the conceptual type, size, and location of the required walls (see Figures 4 and 5). In addition, required information is identified such as:

1. Prequalification requirements (Table 1).
2. Geotechnical reports and data.
3. Design parameters.
4. Site limitations such as right-of-way.
5. Design submittal requirements.
6. Material specifications.
7. Instrumentation requirements.
8. Testing requirements.
9. Acceptance criteria.
10. Method of payment.

This information is usually made available sixty (60) to ninety (90) days prior to the bid date. Prequalified specialty contractors prepare and submit final design calculations and construction drawings for the engineer's review. Once the designs are approved, the specialty contractors are placed on a list of "prequalified specialty contractors with approved designs." This list is then distributed to the General Contractors. General Contractors receive bids only from the prequalified specialty contractors. Often the specialty contractors provide design drawings that are included in the bid documents. These drawings illustrate the proposed construction and allow the General Contractor to understand and coordinate his other project tasks with the retaining wall construction. Once the contract is awarded, the specialty contractor typically subcontracts his services to the General Contractor.

POST-BID DESIGNS

The contract documents for Post-Bid designs are also prepared to allow for the bidding of various retaining wall alternates. The major difference in this method is the timing of the design and the bidding. The engineer provides the conceptual type, size, and location of the desired wall. In addition, prequalification requirements are checked and the required information described above under Pre-Bid designs are provided for the specialty contractor's use.

During the bidding process, the specialty contractor prepares a preliminary design and a firm cost estimate. General Contractors receive bids from prequalified specialty contractors for the various wall alternates and subsequently select the lowest price to include in their bid. Once the contract has been awarded, the specialty contractor prepares his final design and construction drawings and submits the documents to the engineer for review. After acceptance of the design, construction begins.

Other types of retaining walls that the reader may be familiar with for fill situations such as mechanically stabilized embankment walls, gabions, or cribwalls may also be included in the contract documents through Pre- or Post-Bid designs.

ADVANTAGES TO OWNERS OF MODIFIED PERFORMANCE SPECIFICATIONS VERSUS CLOSED SPECIFICATIONS

- Allows for innovative designs, resulting in lower construction costs.

- The specialty contractor shares in the responsibility for the design, construction, and performance of the completed wall.

- The engineer has the opportunity to review each contractor's design and to critique the various design elements, thus enabling him to incorporate necessary revisions into each design prior to bidding. This procedure eliminates confusion or misinterpretation that often results in design changes after the bid that could adversely affect construction costs.

- Allows the Owner to take full advantage of the experience of qualified specialty contractors.

- Design costs are minimized.

- Allows value engineering to be utilized more effectively since the value engineering is essentially performed prior to contract award and the start of construction. The Owner retains one hundred percent of the value engineering savings rather than sharing the savings.

- Minimizes change orders and contract disputes.

CASE HISTORIES

PERMANENTLY ANCHORED RETAINING WALL FOR A STATE HIGHWAY DEPARTMENT

The following case history represents a Closed Specification project where the anchored retaining walls were designed by the State Highway Department (Owner) and his consultant. The price paid by the Owner for this competitively bid project was very high. Due to the nature of the following discussion, the project location, Owner, and consultant will not be identified.

PROJECT BACKGROUND

In 1985, a State Highway Department (Owner) recognized the need to widen and re-deck a bridge over an existing Interstate highway. In addition, the Owner had the foresight to realize that the Interstate would require widening in the near future. An innovative technique for the proposed widening under the existing bridge was conceptualized. The proposed walls would be constructed just inside the abutments for the existing three span bridge (see Figure 6.) Thus, the existing slope paving could be removed, retaining walls could be constructed adjacent to the abutments, and the new lanes could be built without demolishing and reconstructing the existing bridge, which carried a busy State Route.

RETAINING WALL DESIGN

Because the existing bridge was to be widened and re-decked, a special opportunity existed to construct a portion of the proposed retaining walls under the bridge without a low headroom restriction. Some of the work for the proposed retaining walls could be performed from the bridge during renovation, after the decking had been removed.

An anchored retaining wall was envisioned as the most cost effective means for the construction of the retaining walls. It was assumed that typical steel soldier beam anchored walls would be the best type of retaining walls to meet the project requirements. Once the decking was removed from the bridge, the soldier beams could easily be installed through the longitudinal bridge beams from temporary mats located on the bridge. This procedure eliminated the complicated, if not impossible process of setting or driving beams under the bridge. Also, because the abutments were founded on spread footing foundations, stressing of the tiebacks during construction would limit any undesirable outward deflection of the retention system.

Unfortunately, the elegant nature of this concept ends at this point in the case history. It was decided that the piles would be set in drilled holes, just inside the abutment footings. Very large W21 X 132 soldier piles were specified. The design called for the piles to be extended 21 feet below final grade, using 3000 psi concrete in the sockets. Since the designers were unfamiliar with the use of "lean-mix" concrete as a filler around the drilled-in soldier piles above final grade, sand was specified as the filler material. Chemical stabilization of the sand before excavation for the retaining walls was specified by drilling and grouting the loose sand with sodium silicate.

The Owner let a contract to a local drilling company to drill the holes and place the beams along the lines of the walls. The drilling contractor was a caisson installation specialist, and had no previous experience with anchored wall construction. Figure 7 is a photograph of the finished beams after restoration of the slope paving and bridge deck.

A local engineering consulting firm was then retained to design the anchored retaining walls and write a special provision to the specifications to cover this type of construction. Considerable difficulty was encountered during the design process. Since the bridge was founded on shallow foundations, sizable surcharge loadings were imposed on the anchored retaining walls. The size of steel wale required to transfer anchor loads to the soldier piles would not fit into the flanges of the soldier piles, even when 50 ksi steel was used. Because the problem seemed insurmountable to the designers, the specifications called for the contractor to leave pockets in the back of the precast concrete fascia wall panels to accommodate the heads of the anchors. Since the bottoms of the precast panels were designed to parallel the down grade of the road grade, each precast panel was required to be cast specially to mate with the anchor locations. Special insulators were also specified between the anchors and the wale for corrosion protection.

Reinforced concrete wall segments were designed to carry the permanent loads imposed by the earth pressure and surcharge acting between the soldier piles. These walls were 8 inches thick, and were located between the soldier piles, within the webs of the beams. However, temporary excavation support was also required to hold the soil during the tieing of the reinforcing steel, forming, and placement of the concrete walls. The designers specified bridge deck form, used vertically, as the temporary support. Due to the corrugations in the deck form, it was also specified that styrofoam be glued into the corrugations placed against the soil such that no void space remained at the contact. The deck form was to be held against the rear flanges of the soldier piles using angle clips welded to the webs of the beams.

Reinforced concrete pilasters were also specified inside the flanges of the soldier piles for corrosion protection and as the anchorage for the precast fascia walls. The pilasters were to be cast around protruding reinforcing bars cast into the fascia.

Permanent drainage for the retaining walls was specified as a single layer of woven geotextile placed against the soil behind the bridge deck form and continuous between the soldier piles.

Wing walls for the retaining walls were also designed. Since access outside to the bridge was unrestricted, a design scheme similar to that employed under the bridge was developed. The wing walls were designed as cantilevered walls with no anchors. Drilled-in soldier piles with 20 foot sockets below final grade were specified. Socket concrete was again specified as 3000 psi, but instead of sand in the remainder of the drillhole, 2000 psi concrete was called for.

A reinforced concrete cap beam was designed at the tops of the retaining walls. These beams were cast-in-place along the top of the precast fascia panels. The beams were massive and heavily reinforced.

BIDDING

It was apparent to National Foundation Company during the bidding process that several economies could be made if design changes would be allowed. However, the bid documents did not allow for alternates, and the project was bid as designed. National Foundation Company was the successful anchored wall subcontractor. It was later learned that several other retaining wall subcontractors did not bid the project due the complex and difficult nature of the design. The overall bid price for the retaining wall, including the precast facing was very high.

REDESIGN AFTER THE BID

After the bid, it was discovered that the existing soldier piles beneath the bridge were misaligned. The beams were as much as 6 inches out of line. This error made it virtually impossible to install the steel wales between the soldier piles.

National Foundation Company approached the Owner along with the concrete subcontractor to suggest design changes that would allow for a more constructible retaining wall. National Foundation Company suggested that a cast-in-place concrete wale would accommodate the misalignment of the soldier piles while easily carrying the required design loads.

The use of more standard and easily constructed precast connection details would also be facilitated by the use of the reinforced concrete wale. The use of concrete would also eliminate the need for the special insulator pads.

The fascia contractor also suggested using flat-bottomed precast facing elements, eliminating the need for individual casting of the panels. The concrete contractor also pointed out several forming problems with the pilasters and the 8 inch thick cast-in-place concrete walls. Several of these elements were unconstructible. These forming problems would virtually be eliminated with the use of the concrete wale.

Shotcrete was proposed by National Foundation Company as an alternate to the bridge deck form as temporary support for the concrete walls. Where the walls were not planned to extend above grade, shotcrete was suggested as an alternate to the tedious installation of the bridge deck form.

Lean-mix concrete was suggested as a filler above the sockets for the wing walls. Removal of the 2000 psi concrete from the soldier piles was considered to be very difficult and unnecessary. Although it was too late to change the use of sand as the filler in the drillholes under the bridge, National Foundation pointed out that the chemical stabilization of this material cost the owner more than 75 thousand dollars. The use of lean-mix for this application could have saved on the order of 50 thousand dollars.

National Foundation also pointed out that the use of a woven geotextile as a drainage mat was inappropriate. A filtered waffle type material was recommended to relieve any hydrostatic forces that might develop behind the walls. These drainage materials are typically extended vertically in strips between soldier piles and connected to weep holes at the bottom of the wall.

The size and purpose of the cast-in-place cap beams were also discussed. Since the beams were assumed to be architectural, inquiries were made why the beams were so large and heavily designed. The consultant stated that the beams were placed and structurally attached to the tops of the soldier piles to act as pseudo beam seats in case the bridge ever settled.

The Owner was amenable to many of the suggestions made by the contractors. Since National Foundation Company is also a design/build contractor, employing engineers registered in the state in question, design services for these changes were offered at no cost since the bid was based on the design shown in the contract bid documents. However, the Owner's consultant had a significant effect on the redesign through his review comments.

The reinforced concrete wale idea was accepted, but review of the original design submitted by National Foundation by the Owner's consultant added a significant amount of additional reinforcement to the wale, even though all codes had been satisfied. Figure 8 illustrates the approved heavily reinforced wale design, and shows the details of the other components of the wall system adjacent to the wale. The flat bottomed, standard width precast fascia panels were approved. The use of shotcrete below the top level of the wale was approved. The use of lean-mix, and waffle type drainage boards were also accepted. The large cap beam was retained as a settlement guard as described earlier. Figure 9 illustrates the tight confines for the drill rig and the low overhead construction conditions.

CLAIM

Due to design changes required by the Owner's consultant, a claim was filed for additional material, labor, forming, and design costs on the project. The total claim amount was for more than 50 thousand dollars. The claim is currently being reviewed by a claims consultant for the Owner, and has not yet been resolved.

PERMANENTLY ANCHORED RETAINING WALL FOR VIRGINIA DEPARTMENT OF TRANSPORTATION, RICHMOND, VA

The following case history represents a Modified Performance Specification project. The anchored retaining walls for this project were designed by National Foundation Company as a Pre-bid Design for the Commonwealth of Virginia Department of Transportation (VA DOT). The price paid by the Owner for this competitively bid project was relatively low.

PROJECT BACKGROUND

The new Parham-Chippenham Connector (VA Route 150) is currently under construction in Richmond Virginia. Adjacent to where the Connector crosses the James River, the new roadway is being constructed to underpass existing River Road (VA Route 650). As part of the grade separation for the roadways, permanently anchored retaining walls are being constructed at the new bridge abutments for River Road. The retaining walls are adjacent to the new bridge abutments, which are founded on drilled pier foundations.

Several anchored retaining wall contractors were contacted by the Commonwealth of Virginia Department of Transportation to prepare Pre-Bid designs for the proposed permanently anchored walls for this project. The VA DOT prequalified the contractors and gave them notice to proceed with design about 120 days before the project was bid. No compensation was given to the contractors for their design expenses.

The VA DOT provided only conceptual details of the proposed retaining wall such as roadway geometry, and the elevation view wall envelope. Geotechnical information was also provided. The design drawings and calculations were reviewed by the VA DOT and their consultant, Hayes, Seay, Mattern and Mattern for conformance with the specifications and applicable codes. Revisions were checked and approved by the consultant and the VA DOT.

RETAINING WALL DESIGN

National Foundation Company submitted a Pre-Bid design for the project after prequalifying as an experienced anchored retaining wall contractor. The permanently anchored retaining wall design included drilled-in soldier piles consisting of two wide-flange steel beams. The soldier piles were cast into predrilled holes at 8 foot center to center spacings. Ten foot long sockets were provided below final grade. The soldier piles were cast into 3000 psi concrete in the sockets, and into lean mix concrete above final grade.

Permanent ground anchors consisted of three strands of stress relieved seven wire strand. The anchors were drilled between the double beams, through the lean mix and into the soil. The anchors were attached to the soldier piles via steel wedge plates, bearing plates, and anchor heads with wedges.

As the excavation proceeded from original grade, wood lagging was placed to support the exposed soils between the soldier piles. Filtered waffle type drainage boards were placed on the outside of the lagging, mid-way between the soldier piles. Nelson studs were welded in pairs along the outside flanges of the soldier beams. A permanent 12 inch thick reinforced concrete facing was cast against the soldier beams and lagging using single sided formwork. Weep holes were attached to the bottom of the drainage boards and extended through the facing (see Figure 10.)

BIDDING

The approved Pre-Bid designs were included in the contract documents that were distributed to General Contractors for bids. Therefore, the General Contractors were able to see the various designs that were proposed by the prequalified anchored retaining wall subcontractors prior to the bid. The designs were competitively bid by the anchored wall contractors to the General Contractors. National Foundation Company's low bid was accepted by the successful General Contractor, and the subcontract for the construction of the permanently anchored retaining walls was awarded.

REDESIGN AFTER THE BID

No redesign after the bid was required since National Foundation Company had performed the original design. Thus, significant time was saved in the project schedule, and delays were averted. No change orders have been necessary during the contract.

Figures 11 and 12 are photographs of the project during construction.

CLAIM

No claims have been submitted on this project concerning the construction of the permanently anchored retaining walls. Although the project is not yet completed, it has proceeded very smoothly, and no contract disputes have occurred. No claims are anticipated.

COMPARISON OF THE CASE HISTORIES

The two case histories presented were both projects performed for State Departments of Transportation. The projects were both constructed within the Piedmont Physiographic Province and essentially built during the same seasons of the same year. Costs for the permanently anchored retaining walls for the Closed Specification project were more than 60 percent greater than for the Pre-Bid design project. Some differences in the projects could account for a slightly higher cost for the Closed Specification project. The Closed Specification case was constructed in low headroom conditions. However, other features of the Closed Specification project should have reduced the overall cost of the retaining walls. For example, the soldier beams under the bridge were installed under a separate contract. Therefore, the cost of the materials and drilling for the soldier piles was not included in National Foundation Company's bid price for the project.

SUMMARY

Contract specification options for anchored (tied-back) retaining walls are discussed in this paper. Closed Specifications, Open Specifications and Performance Specifications are discussed and compared. In addition, two recent case histories of retaining wall construction are presented and discussed. One of the cases was bid using Closed Specifications and the other was bid with a Modified Performance Specification (Pre-Bid design.) The costs for the Pre-Bid design case were considerably less than for the Closed Specification project.

Owners who allow qualified anchored retaining wall contractors to bid Open or Performance Specifications can experience significant cost savings for the construction of anchored retaining walls.

REFERENCES

1. Clough, Richard H., Construction Contracting, John Wiley and Sons, New York, 1975.

2. Nicholson, Peter J. and Boley, Dennis L., "Construction Alternates: A Cost Effective Method For the Design and Construction of Retaining Walls," ASCE Symposium on Permanent and Temporary Ground Anchors, Seattle, 1985.

TABLE 1

TYPICAL LIST OF SPECIALTY CONTRACTOR PREQUALIFICATION REQUIREMENTS

The specialty contractor shall complete or revise, depending on the contractor's current qualification status, a prequalification application. This application should include all relevant equipment and experience information concerning permanently anchored walls. This application must be submitted to the Owner/engineer for review. The following additional information shall be provided.

1. The contractor shall be experienced in the design and construction of permanently anchored retaining walls.

2. The contractor's staff shall include at least one registered professional engineer with at least three (3) years of supervisory experience in the design and construction of permanently anchored retaining walls.

3. The contractor's staff shall include drill operators and foremen with a minimum of two (2) years of experience constructing permanently anchored retaining walls.

4. The contractor shall list at least five (5) active or completed projects built in the last three (3) years that are similar in concept and scope to the proposed anchored retaining wall.

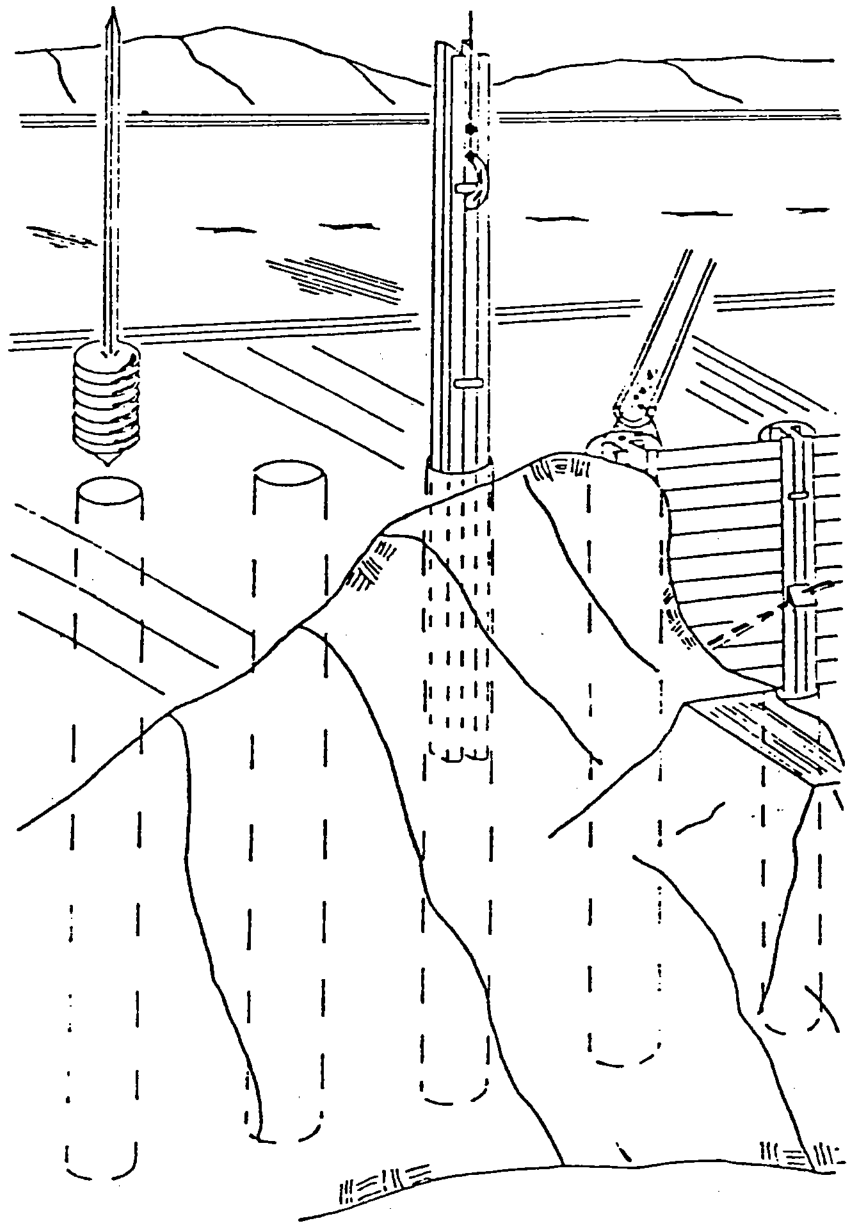


FIGURE 1

CONSTRUCTION SEQUENCE I
DRILLING AND PLACING OF SOLDIER BEAMS

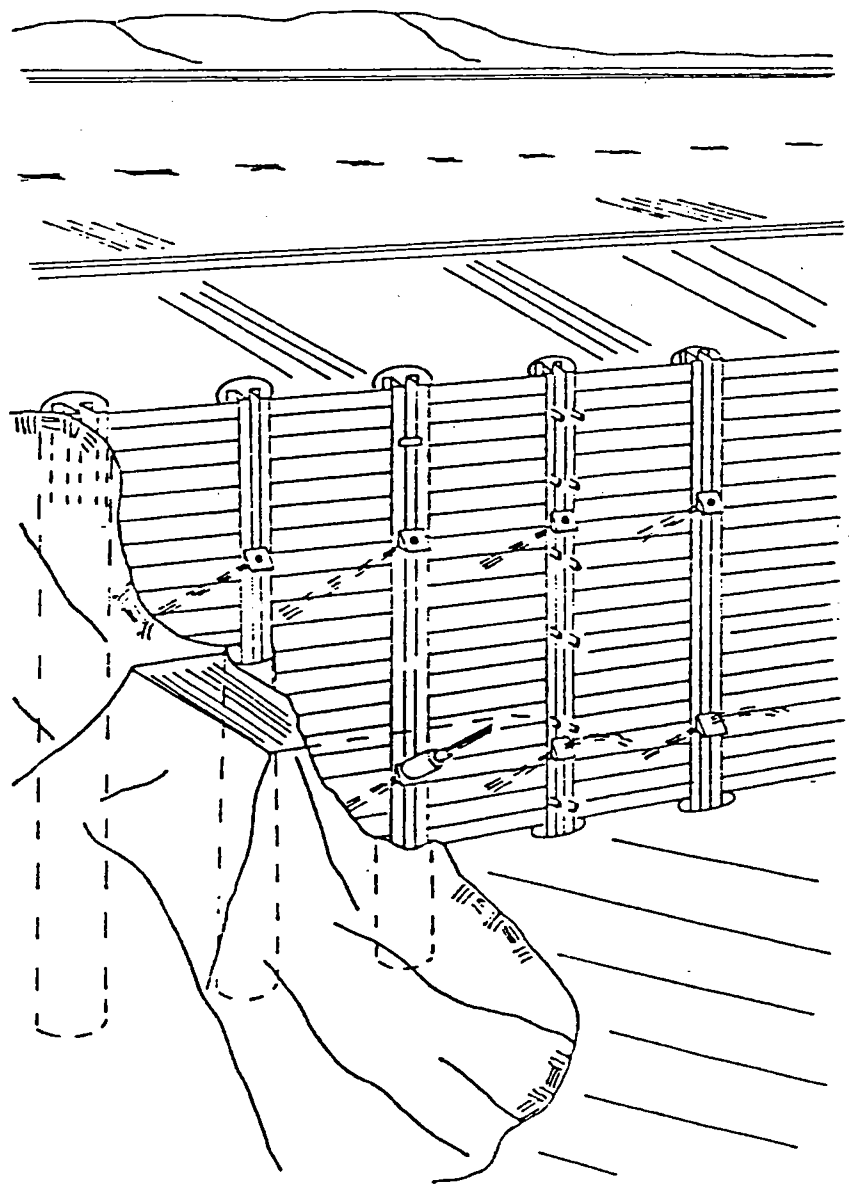


FIGURE 2

CONSTRUCTION SEQUENCE 2
EXCAVATION AND PLACING OF LAGGING AND ANCHORS

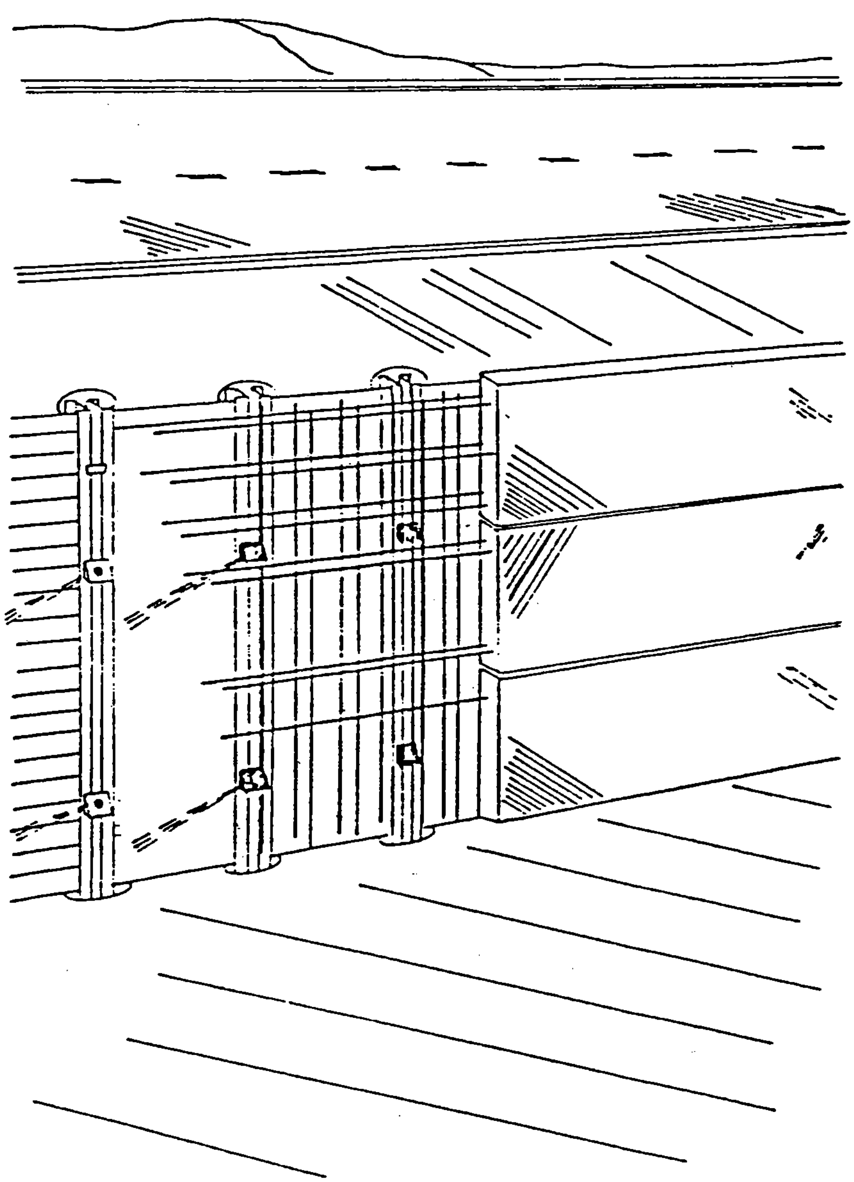
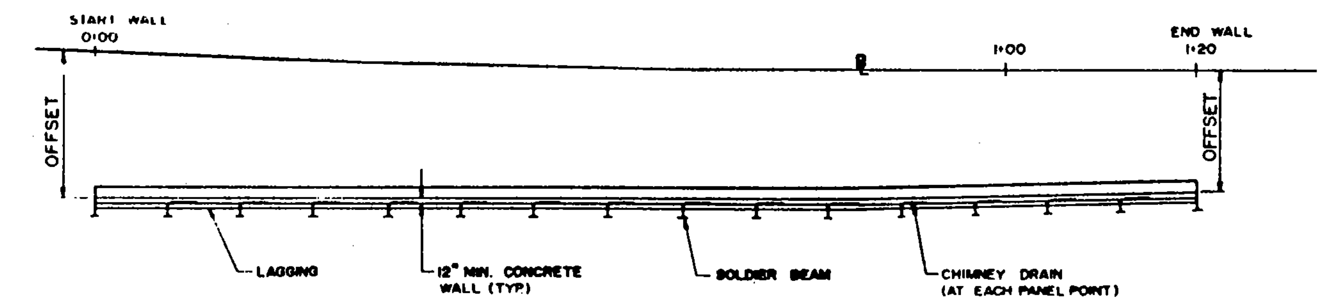
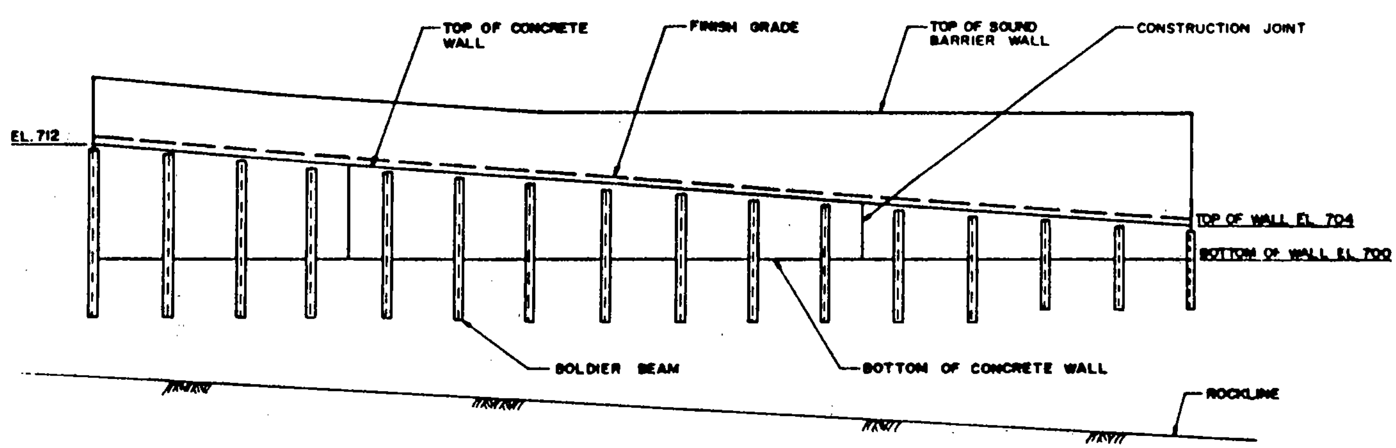


FIGURE 3

<u>CONSTRUCTION SEQUENCE 3</u>
PLACEMENT OF REINFORCING STEEL AND CASTING OF CONCRETE FACING



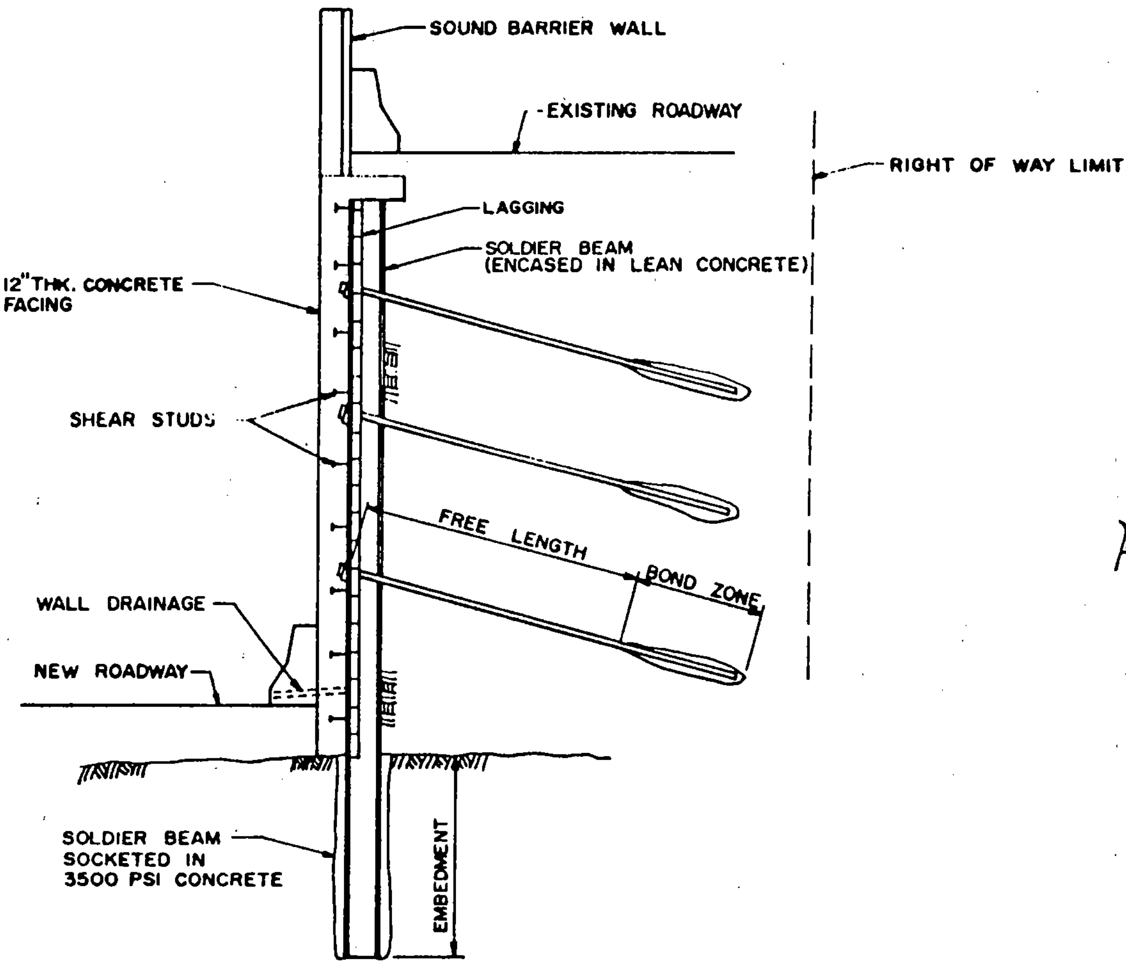
PLAN VIEW



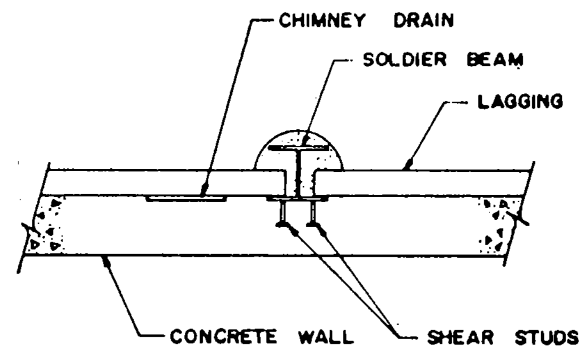
ELEVATION VIEW

FIGURE 4

DEPARTMENT of TRANSPORTATION	
ANCHORED RETAINING WALL PLAN & ELEVATION	
SCALE: 1"=10'-0"	DWN DWM



ANCHOR WALL SECTION



SOLDIER BEAM DETAIL

FIGURE 5

DEPARTMENT OF TRANSPORTATION	
ANCHORED RETAINING WALL DETAILS	
SCALE: N.T.S.	DWN.: DWM

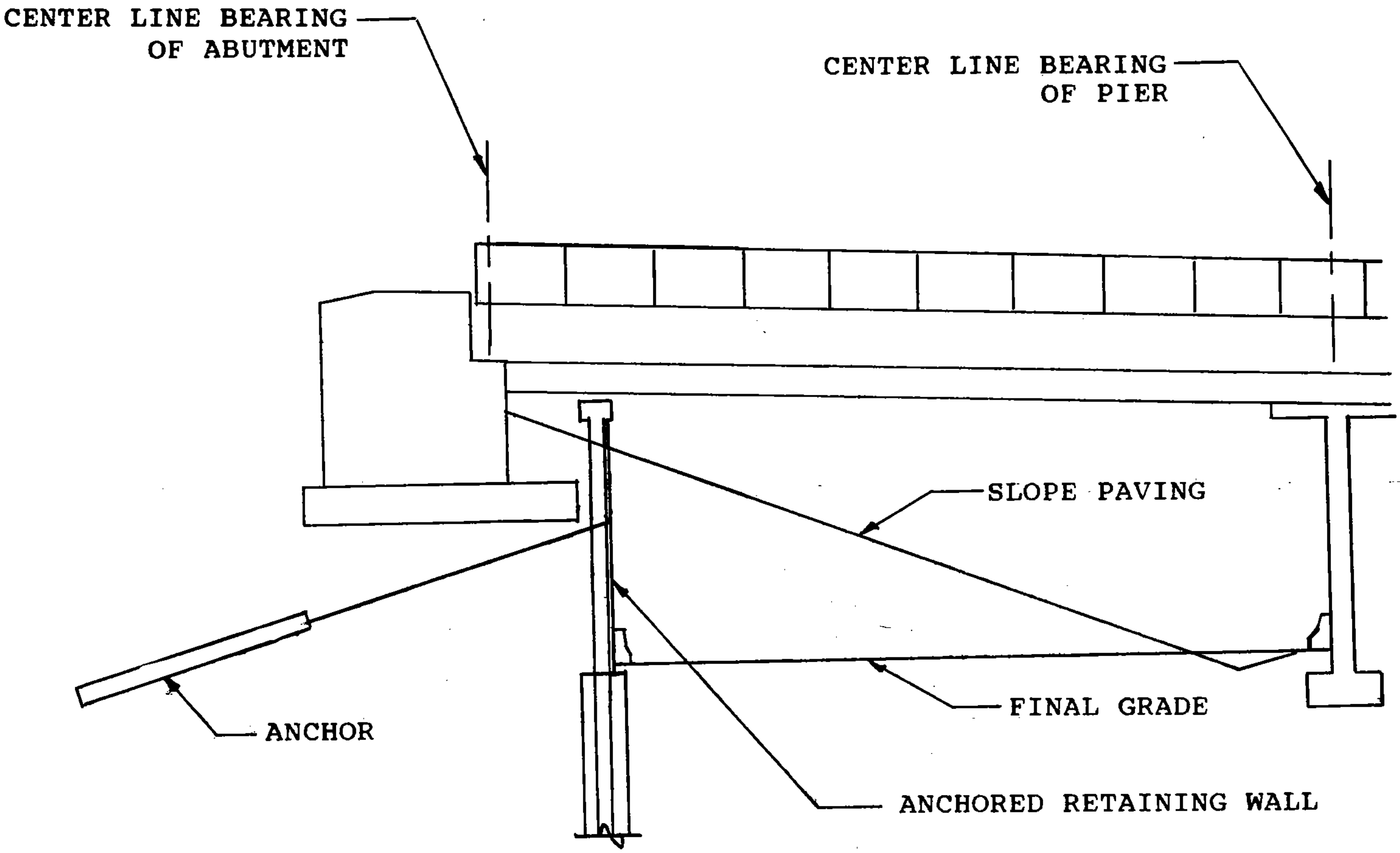


FIGURE 6 - CONCEPT FOR CONSTRUCTING RETAINING WALLS WITHOUT BRIDGE DEMOLITION TYPICAL BOTH ABUTMENTS

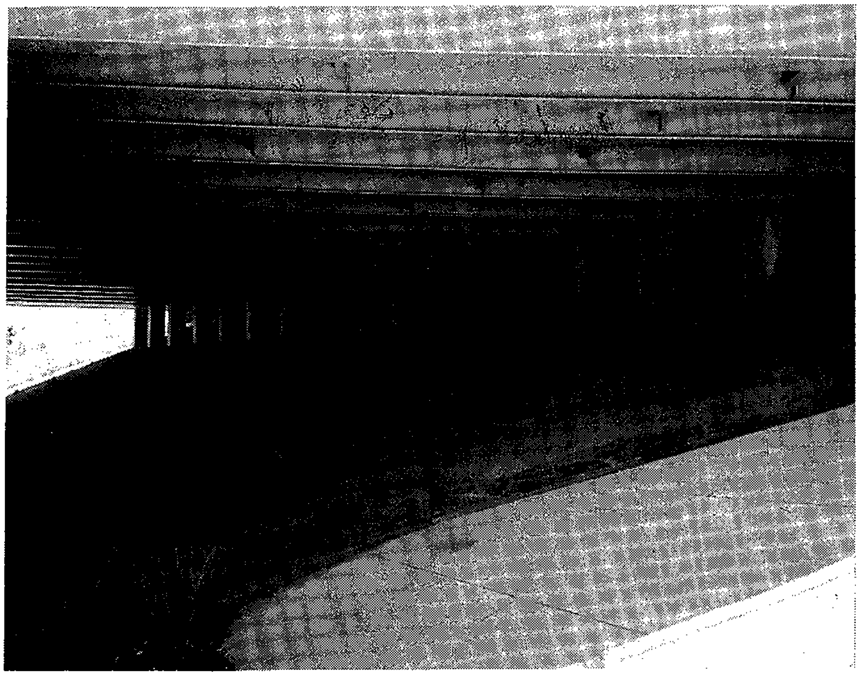


FIGURE 7 - DRILLED-IN SOLDIER PILES

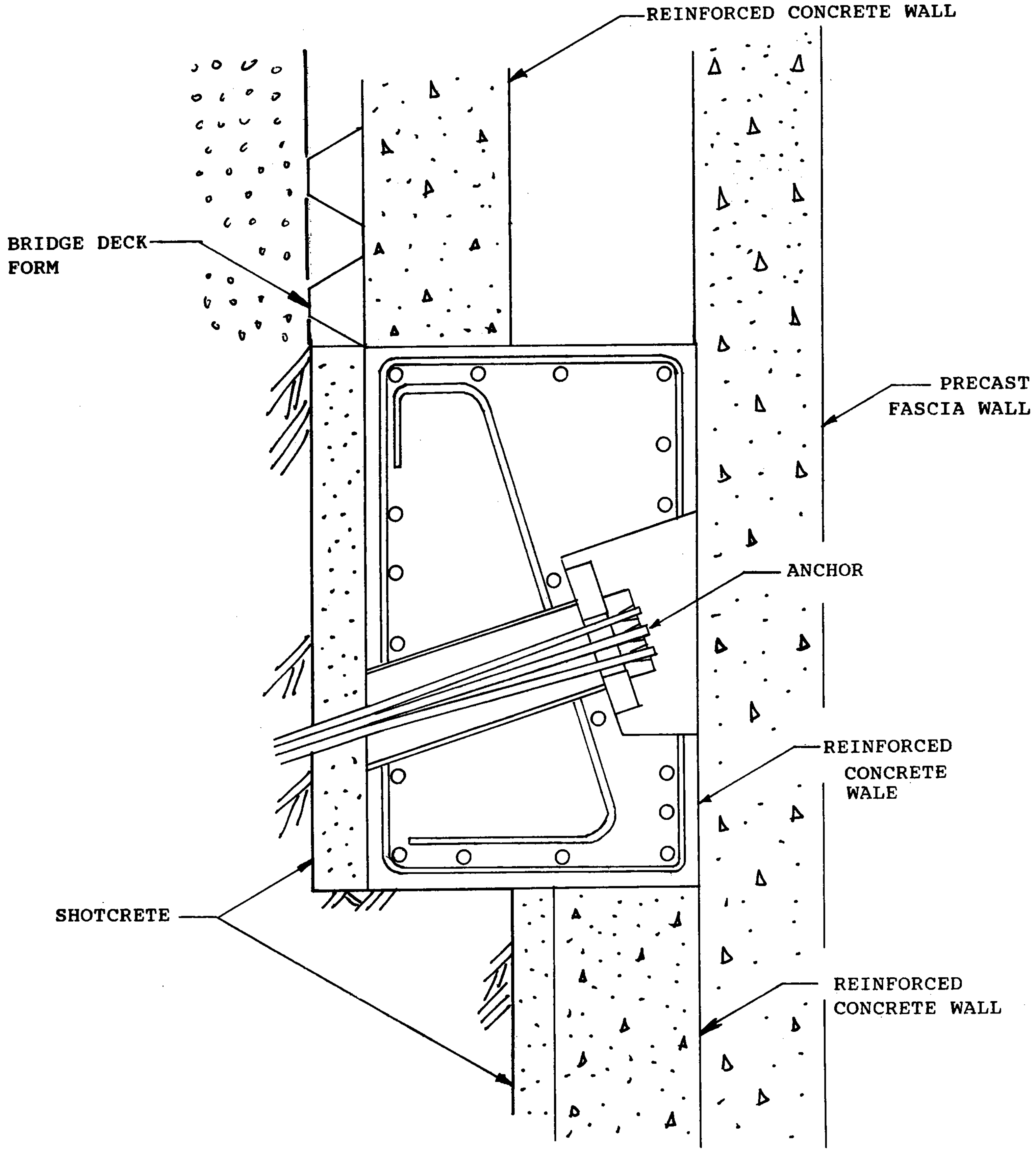


FIGURE 8 - REINFORCED CONCRETE WALE AND WALL ELEMENTS

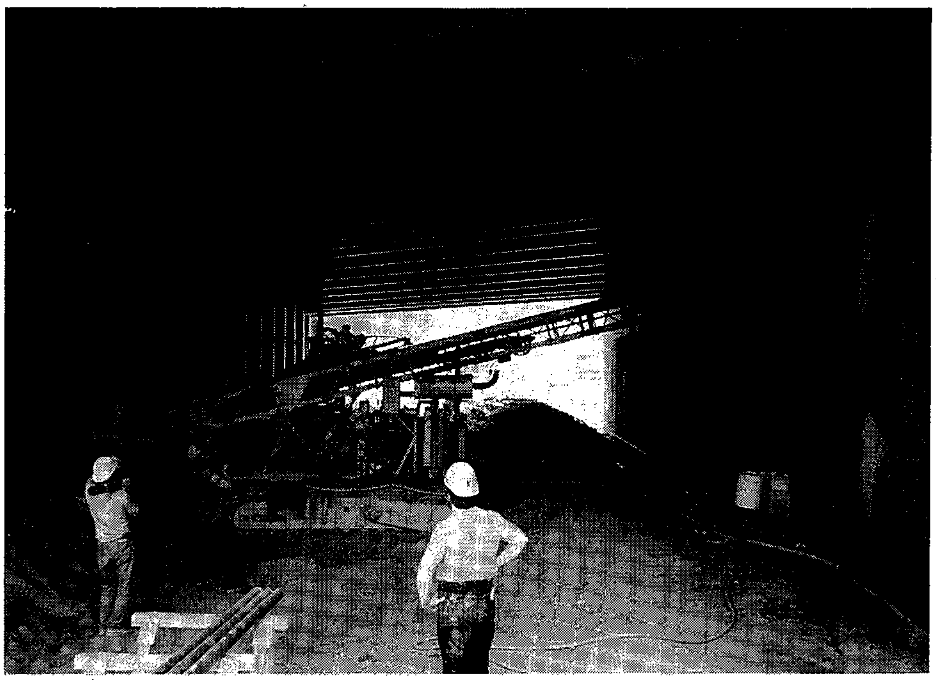


FIGURE 9 - DRILLING FOR ANCHORS

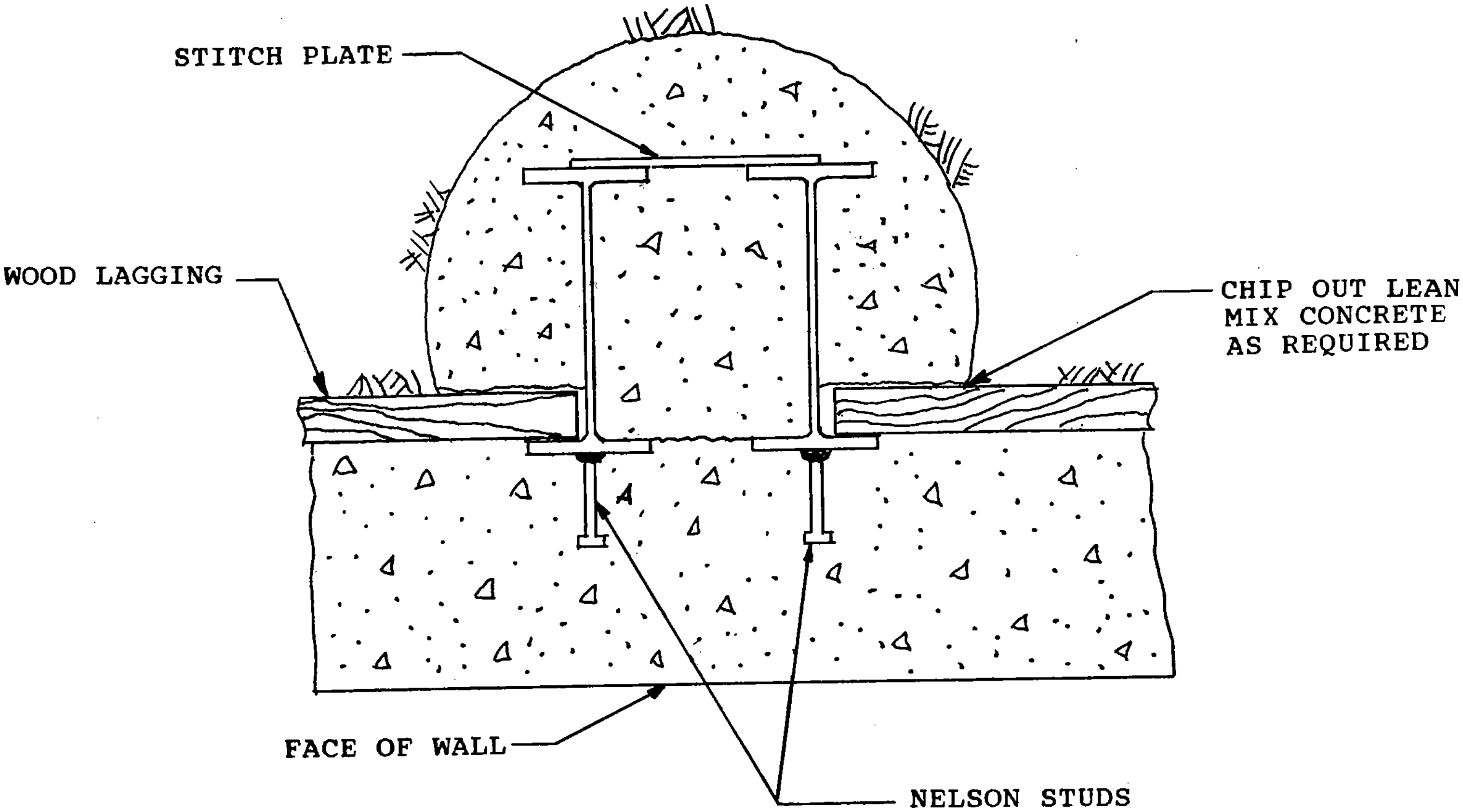


FIGURE 10 - PLAN SECTION THROUGH SOLDIER PILE

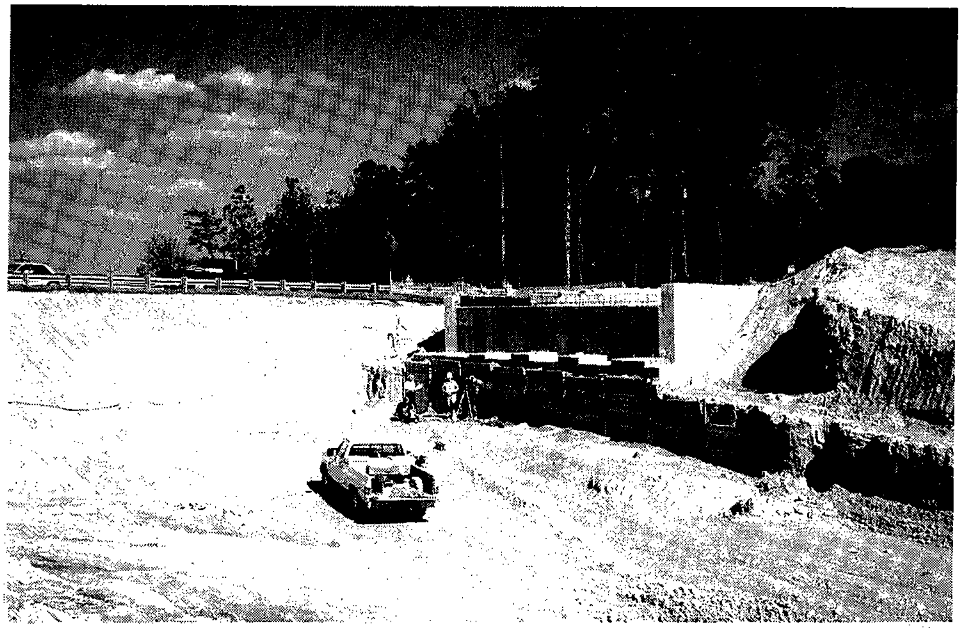


FIGURE 11 - ANCHORED RETAINING WALL AND BRIDGE ABUTMENT DURING CONSTRUCTION

FIGURE 12 - ANCHORED RETAINING WALL DURING CONSTRUCTION

FINITE ELEMENT ANALYSIS OF A RIB-REINFORCED STEEL CULVERT

by

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INTRODUCTION

A finite element analysis was performed to assess the stability of a rib-reinforced, low profile, long span steel arch culvert located at Hayden Creek, Hayden Lake, Idaho. The culvert, 11.2 feet high and span of 34.5 feet, was designed on the basis of empirical methods and suffered some unexpected deformations (sag) during the first few months after installation. Because of limitations of conventional empirical methods of analysis, the finite element method was used to model the complex soil-structure interaction conditions at the culvert and to provide an assessment of the future ability of the culvert to accommodate the design loads safely.

Several finite element computer programs are available for this type of analysis, including FINLIN (Leonards and Roy, 1976), SSTIP, NLSSIP (Duncan, undated). However, the CANDE program (Katona, et al., 1976a,b; 1978; 1981), developed for the Federal Highway Administration, was selected for this study as it has been adjudged as providing the most flexible and realistic treatment of the culvert soil interaction (Leonards, et al., 1982). This paper describes the soil-structure model used in the analysis of the Hayden Creek culvert. Factors of safety for compression and buckling stability are computed from the finite element analysis results and the limitations of the approach are presented below.

FINITE ELEMENT MESH

Figure 1 shows the mesh that was adopted for analyzing the full-section for cases where differential settlement of the foundation could be appropriately simulated. Figure 2 illustrates the mesh used for analyzing a symmetric, half-section of the culvert and surrounding soils. The plane strain meshes used in this study were not investigated for solution convergence and accuracy, since

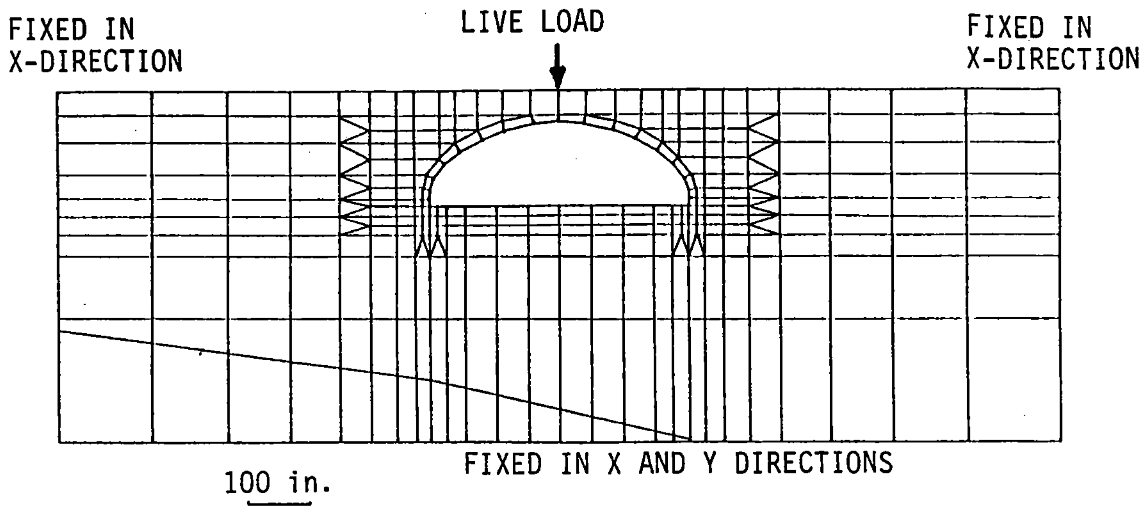

Figure 1, Full Section finite element mesh

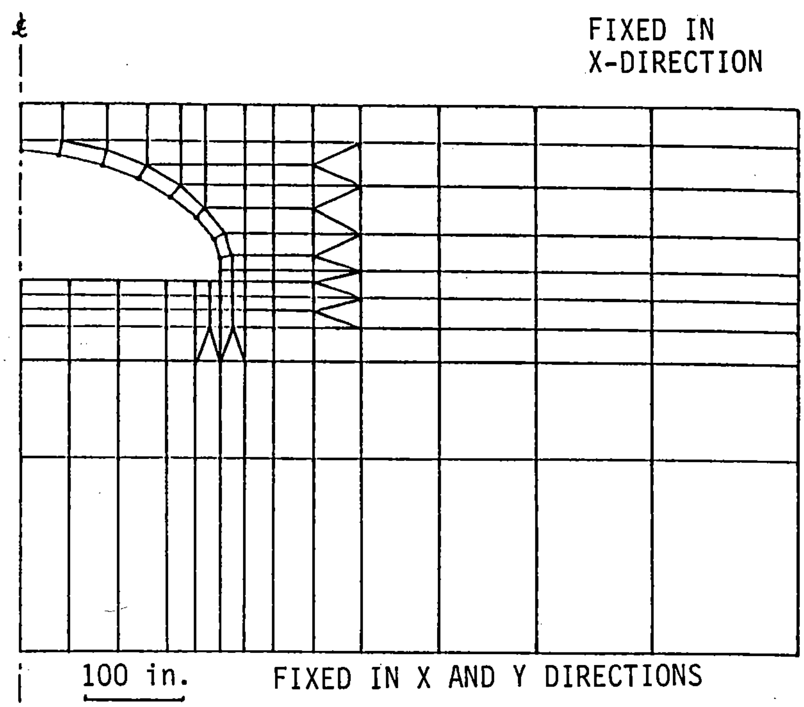

Figure 2, Half-section finite element mesh they have a form that is similar to meshes used for previous analyses by Katona, et al. (1978, 1982). Both meshes are fixed at the lower boundary -- location of bedrock, and are restrained in the horizontal directions at the two side-boundaries. The meshes for the full and half-sections consisted of 167 and 352 elements and 176 and 360 nodes, respectively.

CONSTRUCTION SEQUENCE

It has been shown by Katona (1978), Leonards, et al. (1985) and McVay and Selig (1982) that the construction sequence must be modeled during the analysis phase for a realistic assessment of deformations and stresses. This is simulated by "adding" the soil above the springline in layers, thus effectively accounting for the modulus - overburden stress dependency. The sequential construction effects generally dominate the behavior of large span culverts with a shallow cover similar to the Hayden Creek culvert (Duncan, 1979).

In this study the construction sequence was simulated by the six steps listed below and illustrated in Figure 3 :

1. Soil is "built" to foundation level (Level A in Fig. 3),
2. The culvert is erected,
3. Compacted soil placed to level of springline (Level B),
4. Compacted soil placed to Level C,
5. Compacted soil placed to just above crown (Level D),
6. Compacted soil placed to Final Grade (Level E).

Additional steps were included in the analysis to apply the live-loads simulated by a line load applied above the crown at finished grade elevation.

ANALYSIS OF HAYDEN CREEK CULVERT

Model of Subsurface Profile

The subsoil profile was developed from the data collected from three soil borings performed at the site. Figures 4 and 5 show the distribution of soils used for the analysis of the full and half sections, respectively. Estimated model parameters for the soil zones are presented in Table 1. The model parameters were selected for three different soil conditions from the values suggested by Duncan, et al. (1980) for the appropriate soil types as follows :

Table 1, Soil Paramters for Duncan-Chang Model

Parameter	Soil A			Soil B			SOIL C			SOIL D
	"Good"	"Avge"	"Poor"	"Good"	"Avge"	"Poor"	"Good"	"Avge"	"Poor"	"Avge"
K	800.0	700.0	450.0	600.0	500.0	300.0	600.0	500.0	300.0	200.0
n	0.5	0.4	0.45	0.5	0.35	0.4	0.5	0.35	0.4	0.35
c	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ϕ	50.0	50.0	50.0	45.0	40.0	40.0	45.0	38.0	38.0	30.0
R_f	0.55	0.7	0.6	0.6	0.7	0.65	0.6	0.7	0.65	0.85
G	0.25	0.3	0.3	0.3	0.33	0.35	0.3	0.33	0.35	0.35
F	0.05	0.05	0.1	0.07	0.05	0.1	0.07	0.07	0.1	0.1
d	10.0	5.0	5.0	5.0	5.0	4.0	5.0	5.0	4.0	4.0
γ	125.0	125.0	125.0	117.0	105.0	105.0	60.0	60.0	60.0	50.0

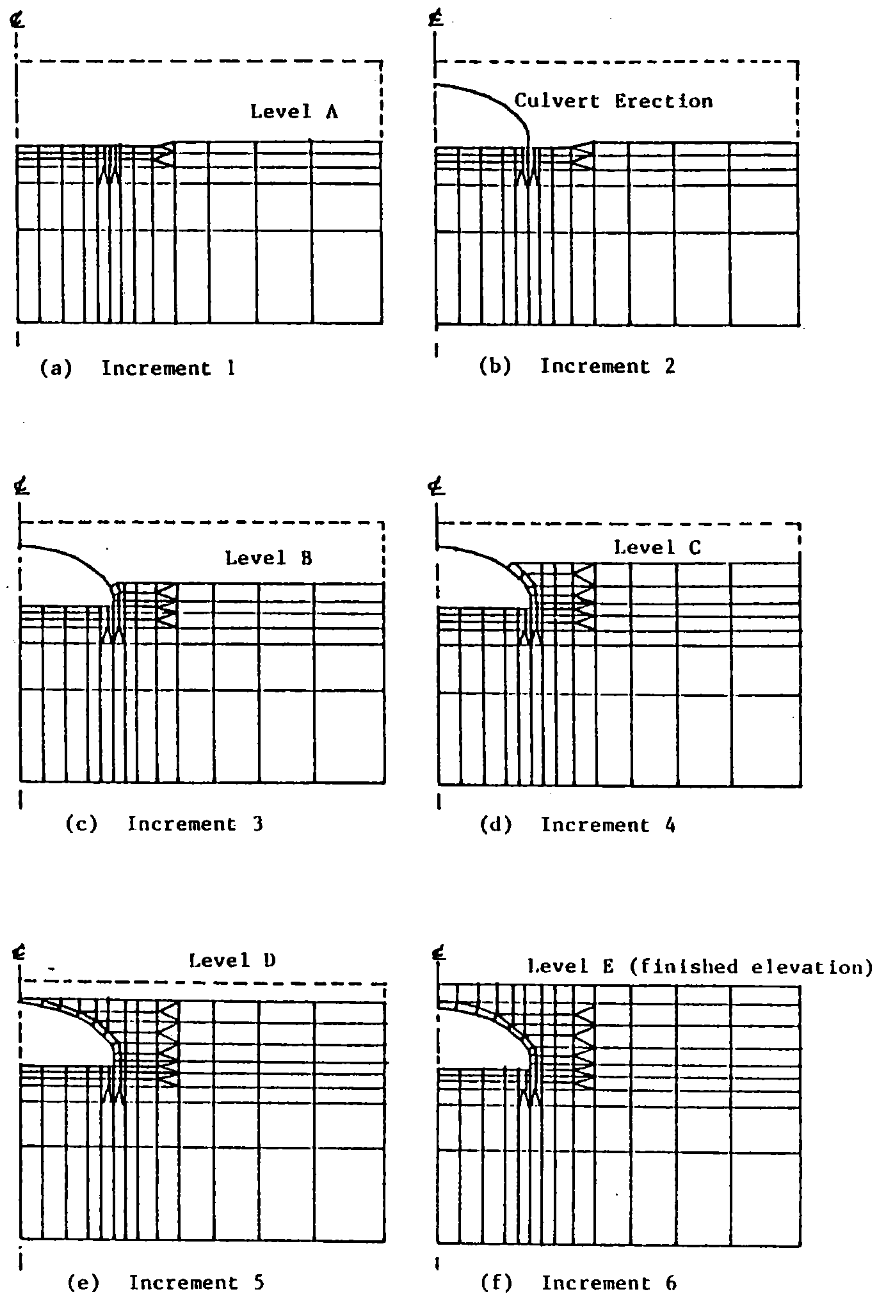

Figure 3, Incremental construction sequence used in the finite element analysis.

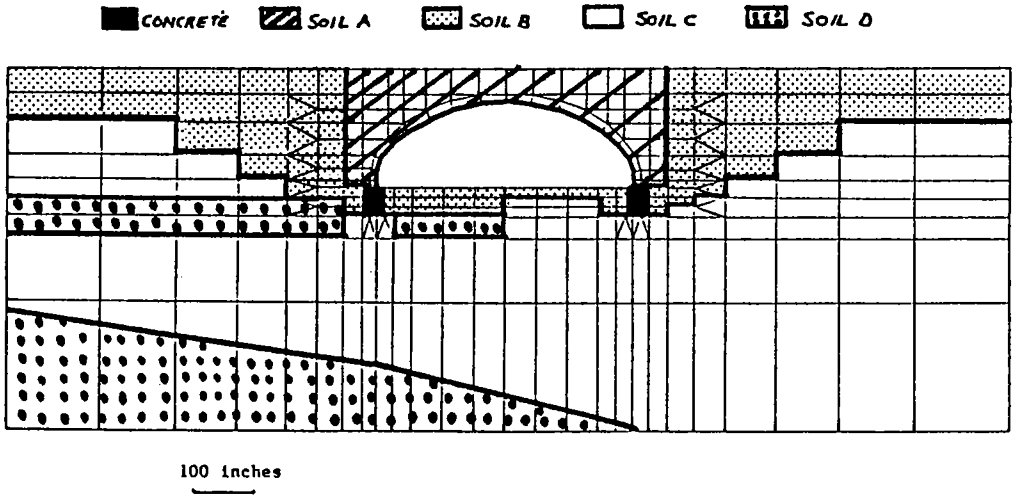

Figure 4, Subsoil model showing soil types (full-section)

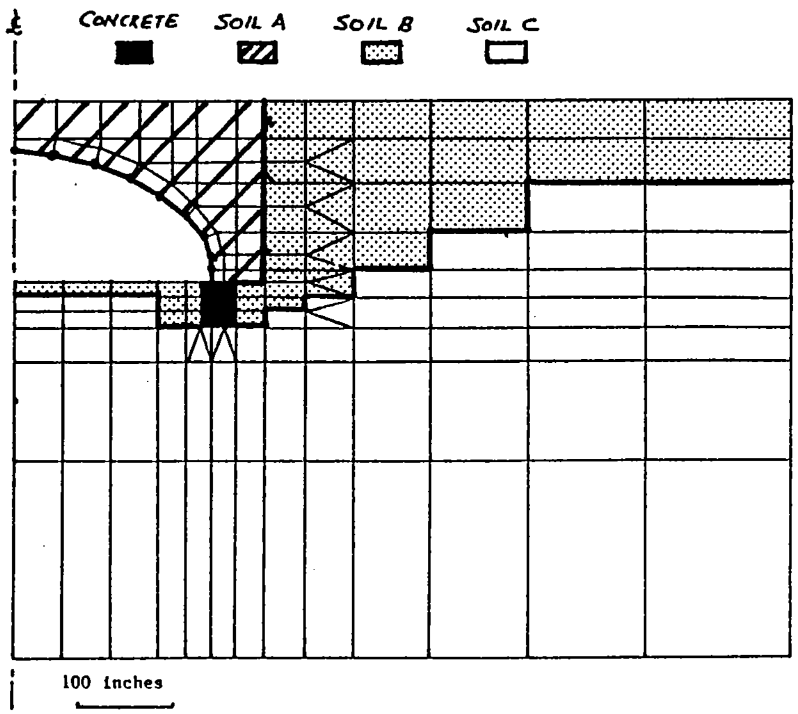

Figure 5, Subsoil profile showing soil types (half-section)

"Good" conditions were selected on the basis of optimum conditions during the compaction and backfilling operation. This category is an "upper-bound" for the strength and compressibility of the soils. With these values, lower stresses and deformations can be expected in the culvert.

"Average" conditions are based on an estimate of the actual conditions that may exist on the site. These values are based on studies of soils of a similar nature from the area and results of the laboratory tests performed on disturbed or remolded samples obtained from the soil borings. The deformation results from the analysis are expected to be similar to values that may have been predicted during the design of the culvert.

"Poor" conditions are based on the assumption that the subsoils and the compacted backfill have "low" strength and high compressibility. Such parameters can be expected to generate large stresses and deformations in the culvert due to a lack of lateral support that is essential for such large span structures.

In using these ranges of soil parameters, the analysis is expected to provide a range of possible results according to the adopted soil conditions.

RESULTS

The finite element analysis provides the following results at the pipe element nodes : (1) Displacements, (2) Thrust forces, (3) Bending Moments, (4) Shear forces. Thrusts and bending moments control the ability of the culvert in supporting the gravitational and design live loads. The shear forces are not expected to affect the stability of the culvert and were neglected in the the computation of the factors of safety. The factor of safety values were calculated according to the following :

Factor of Safety against "pure" compression failure,

$$FOS_t = \frac{P}{A\,\sigma_y}$$

Factor of Safety against buckling (Duncan, 1979),

$$FOS_b = 0.5F_1\left[(F_1^2\ F_2^2 - 4)^{0.5} - F_1F_2\right]$$

where :

A = Cross-sectional area of culvert
S = Section modulus
σ_y = Yield stress of culvert
F_1 = P_p/P
F_2 = M/M_p
P = Thrust force in culvert
P_p = $A\sigma_y$, the axial force at yield
M = Bending moment
M_p = Plastic moment resistance, $1.5\sigma_y S$ (estimated)

Results of the analyses are presented according to the pipe node numbering system shown in Figure 6 for the half and full sections, respectively. The numbering of the nodes increases in a clockwise direction or from the left to the right.

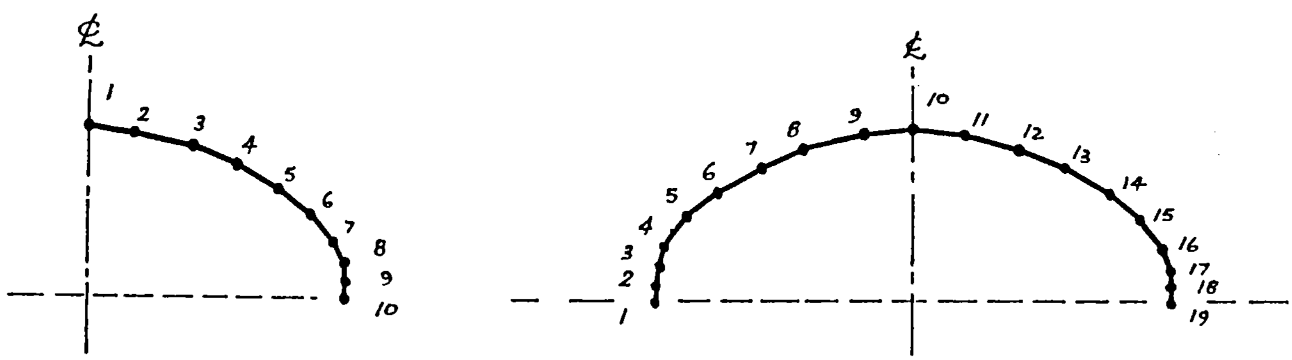

Figure 6, Location of pipe nodes

Linear Elastic Analyses

In order to obtain a general understanding of the effects of reasonable variation in soil parameters and culvert geometry (deformed versus ideal) within a reasonable expenditure of computational effort, a series of preliminary "runs" were performed using constant elastic moduli. The results of four analyses are summarized in Table 2.

Table 2, Linear Elastic Results

	Load Case Description	FACTOR OF SAFETY Thrust	FACTOR OF SAFETY Buckling	Live Load	FACTOR OF SAFETY Thrust	FACTOR OF SAFETY Buckling
1.	Design culvert, One construction step Half-section.	6.8	4.3	250	6.2	4.3
				750	5.3	3.6
				1500	4.1	3.0
2.	Deformed culvert, One construction step Half-section.	6.7	4.9	250	6.1	5.2
				750	5.2	3.8
				1500	4.1	3.0
3.	Design culvert, Six construction steps Half-section.	7.2	5.0	500	6.0	4.1
4.	Design culvert, Six construction steps Full-section.	7.3	5.1	500	6.1	4.2

Non-linear Analyses

The non-linear analyses are expected to provide a better simulation of the anticipated behavior of the culvert and surrounding soils. These analyses were performed using the subsoil models presented in Figures 4 and 5, and the parameters given in Table 1. A summary of the computed FOS values is presented in Table 3, and the results are discussed later.

Table 3, Results from Non-linear Analysis

	Load Case Description	FACTOR OF SAFETY Thrust	FACTOR OF SAFETY Buckling	Live Load	FACTOR OF SAFETY Thrust	FACTOR OF SAFETY Buckling
	<u>Half-section</u> (Fig. 2 mesh)					
1.	"Good" Soils,	6.7	5.4	500	5.5	4.5
2.	"Average" Soils,	6.6	5.1	500	5.7	4.3
3.	"Poor" Soils,	6.2	4.6	250	5.8	4.3
				500	5.3	3.8
	<u>Full-section</u> (Fig 1 mesh)					
4.	"Good" Soils,	6.5	3.8	500	5.3	3.2
5.	"Poor" Soils,	6.3	4.7	500	5.4	3.9

Discussion of Non-linear Cases

Half-section analyses

The non-linear cases represent a realistic simulation of the culvert-soil interaction and are expected to provide the most reliable results. These results, summarized in Table 3, indicate FOS values ranging between 6.2 and 6.7 for thrust failure and 3.8 and 5.4 for a buckling failure. In the case of the half-section, the "poor" soil conditions generated the lowest FOS values. Additional results from this case are presented in Figures 7-10.

Figures 7 and 8 present the predicted displacements of the culvert nodes at three stages of construction and loading. These stages are illustrated in Fig. 3 and the final three may be summarized again as :

1. Inc - 5 --- fill placed to just above crown level,
2. Inc - 6 --- fill at finished elevation,
3. Inc - 7 --- Application of a live load of 500 lb/in directly above the crown.

The y-deflections reveal the anticipated "peaking" at the crown level where crown deflections are smaller at node 1 in comparison to node 4. This effect was noted during construction of the culvert, but was not measured. With the application of the 500 lb/in live load at the crown, the "peaking" effect is nullified and there is an increase in crown deflections with respect to the nearby nodes. The x-deflections indicate the general tendency of the culvert to increase its span at the springline. This lateral movement is required to mobilize the lateral resistance of the compacted backfill.

The predicted thrust force is presented in Figure 9 for the final three loading sequences. The thrust force increases around the culvert, as expected, and was found to be a maximum of 1640 lb/in at the footing level for the dead load condition. In applying the live load of 500 lb/in, the maximum thrust increased by 275 lb/in to 1915 lb/in. The largest increase in thrust was experienced at node 5, amounting to 490 lb/in.

The bending moments for the same case are presented in Figure 10. Positive bending moments are defined as those generating tension on the outside face of the culvert. It can be seen that due to the residual effects of the "peaking", the bending moments at the crown are positive under dead-load conditions, but the applied live load tends to generate negative bending moments. This results

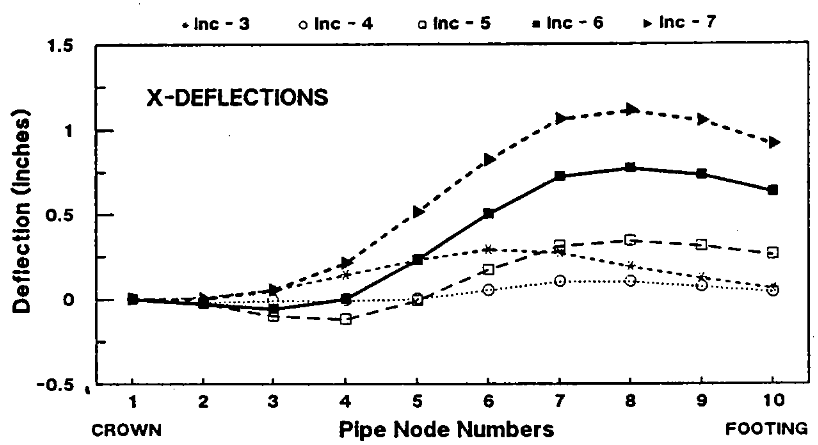

Fig. 7, Horizontal Deflections (poor soil)

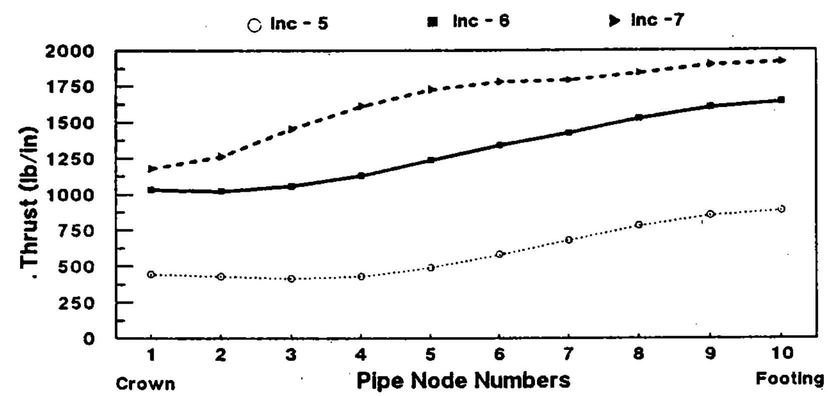

Fig. 9, Computed thrust forces (poor soil)

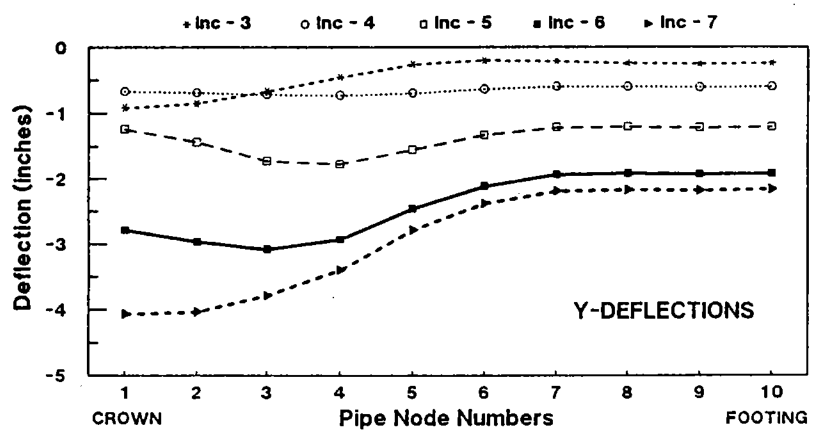

Fig. 8, Vertical Deflections (poor soil)

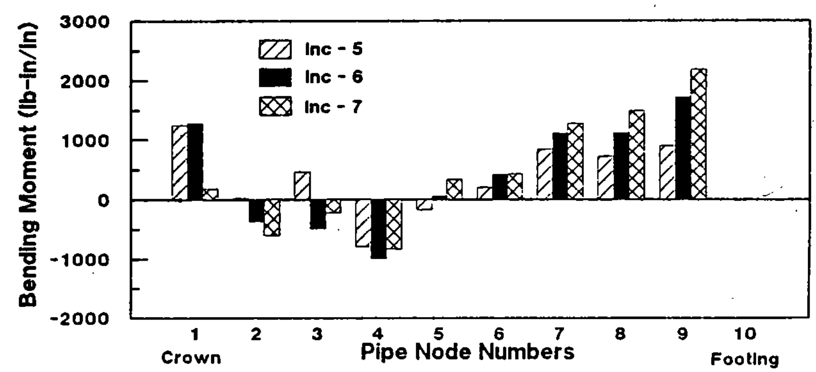

Fig. 10, Computed Bending Moments (poor soil) in a substantial reduction (from 1285 to 170 lb-in/in) in the bending moments at the crown. The maximum bending moment increased from 1705 lb-in/in to 2175 lb-in/in at the springline. Overall, the introduction of a live load of 500 lb/in does not generate any large, additional bending moments.

Full-section analyses

On the basis of results from the half-section analysis, two full-section cases were also analyzed to investigate the effects of a soft layer under the west footing. The FOS values are summarized in Table 3 and the predicted deflections, thrust forces and bending moments are discussed below.

"Good" soil conditions

The deflections of the culvert for the "good" soil conditions are presented in Figure 11 and 12. For the full-section case, the effect of the soft layer leads to non-symmetric deflections and further complicates their interpretation. However, the "peaking" is again evident from the y-deflection values. Also, the inclusion of the soft layer results in larger vertical settlements at the west footing, as expected, in comparison with the east footing. This differential settlement amounts to about 1.25 inches. The horizontal deflections are complex and difficult to interpret due to the inherent interaction between the culvert and surrounding soils.

The thrust forces are presented in Figure 13 for the last two construction sequences and for a live load of 500 lb/in. For the dead load case (Inc - 6), a minimum thrust force of 1000 lb/in is predicted at the crown, and a maximum force of 1575 lb/in occurs at node 17 which is close to the west springline. Generally, the dead load thrust increases from the crown (node 10) to nodes 5 and 15, and then reduces slightly down to the footing level. This variation may be caused by the differential settlement which results in the development of positive arching and an apparent reduction in the dead loads. The application of the 500 lb/in live load increased the thrust at all nodes. A maximum increase of 530 lb/in was predicted at node 14 and the overall maximum thrust of 1931 lb/in was computed at node 17, close to the east springline.

The bending moments for this case are shown in Figure 14 and indicate that the west portion of the culvert near the springline has the higher bending stresses. These moments increased from a maximum dead load value of 3190 lb-

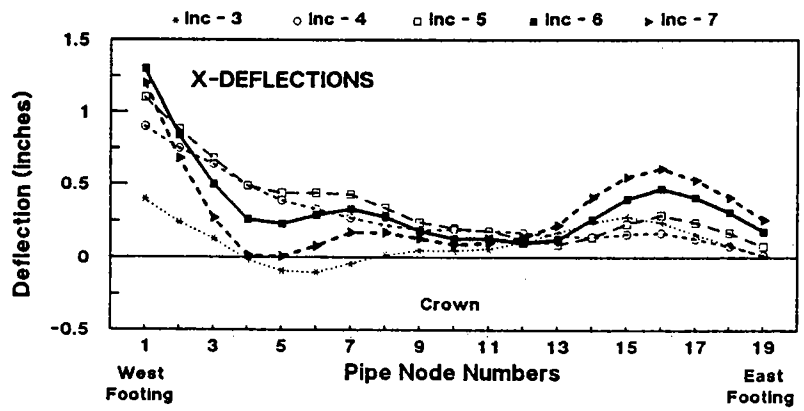

Fig. 11, Horizontal Deflections (good soil)

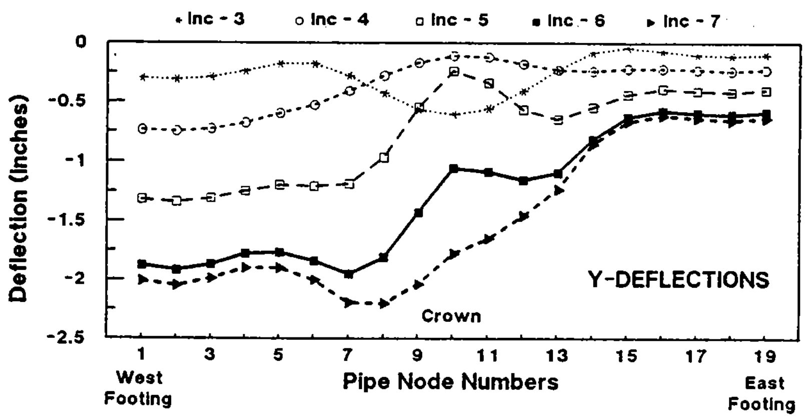

Fig. 12, Vertical Deflections (good soil)

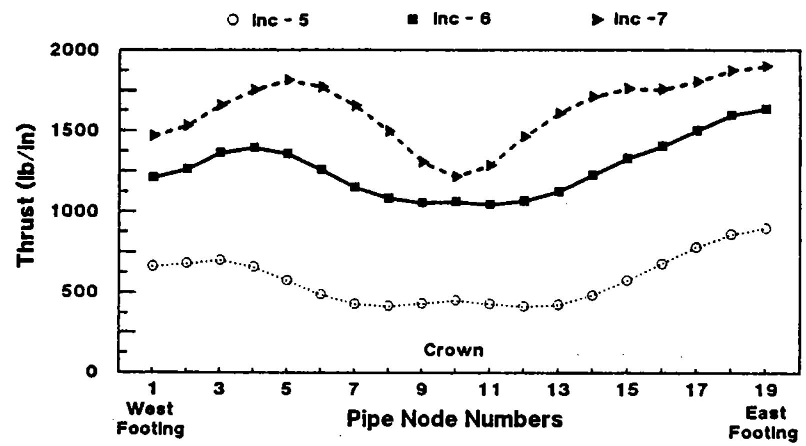

Fig. 13, Computed Thrust Forces (good soil)

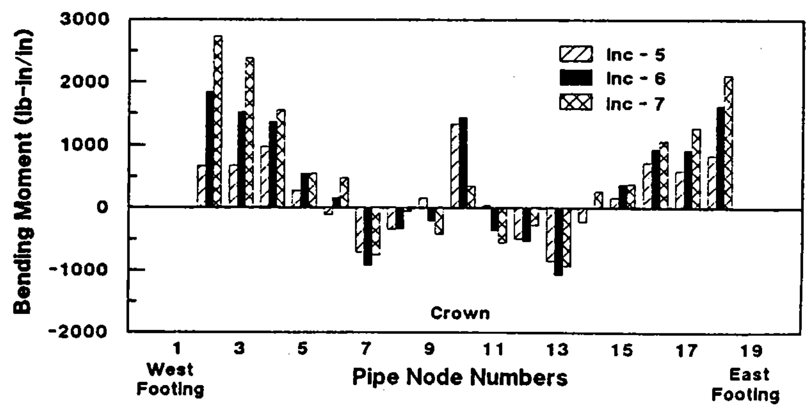

Fig. 14, Computed Bending Moments (good soil) in/in (node 3) to 4040 lb-in/in (node 2) with the application of the live load. The bending moments induced by the live load are significant for this case, with a predicted 57 percent increase at the west springline (node 2). However, this bending moment is not expected to cause any instabilities since there is a FOS = 3.2 against buckling failure. The live load effects on the east side are less pronounced and only increase bending moments by a small amount.

"Poor" soil conditions

Results of the final analysis using the full-section mesh and "poor" soil condition parameters are presented in Figures 15 - 18. For this case, the contrast between the "poor" soils and the soft layer was marginal and only a small amount of differential settlement was predicted by the finite element analysis. The "peaking" effects were successfully simulated during the construction sequence. The x-deflections are again difficult to interpret, but there exists a reasonable symmetry between the predicted deformations at the east and west nodes of the culvert.

The computed thrust forces are similar to the previous case and ranged from 1045 lb/in, close to the crown, to a maximum value of 1630 lb/in at the east footing level for the dead load case (Inc - 6). The predicted bending moments are smaller than the "good" soils case with maximum values of 1840 lb-in/in and 2730 lb-in/lb for the dead load and live load cases. Again the application of live loads was significant, leading to a 48 percent increase in the bending moment at the west springline.

SUMMARY OF RESULTS

On the basis of the finite element analysis of the Hayden Creek culvert, the following minimum factors of safety were determined from several cases :

		FACTOR OF SAFETY	
		Thrust	Buckling
1.	Dead Load Only	6.2	3.8
2.	LL = 500 lb/in	5.3	3.2

The values given for a live load (LL) of 500 lb/in represent twice the HS-20-44 wheel loading. Minimum values for the design live load represented by a

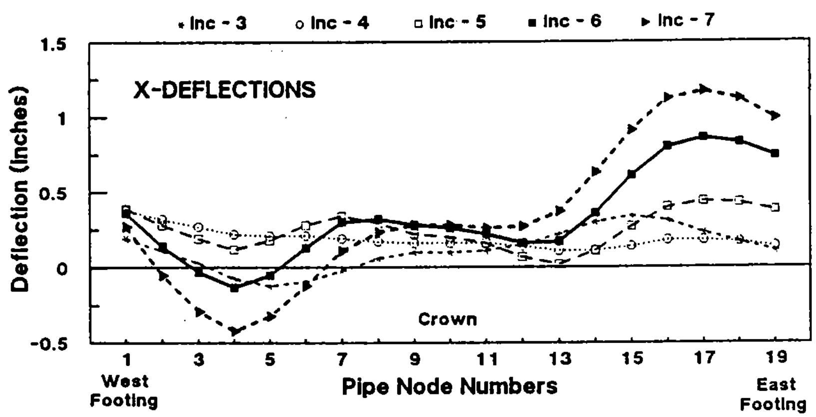

Fig. 15, Horizontal Deflections (poor soil)

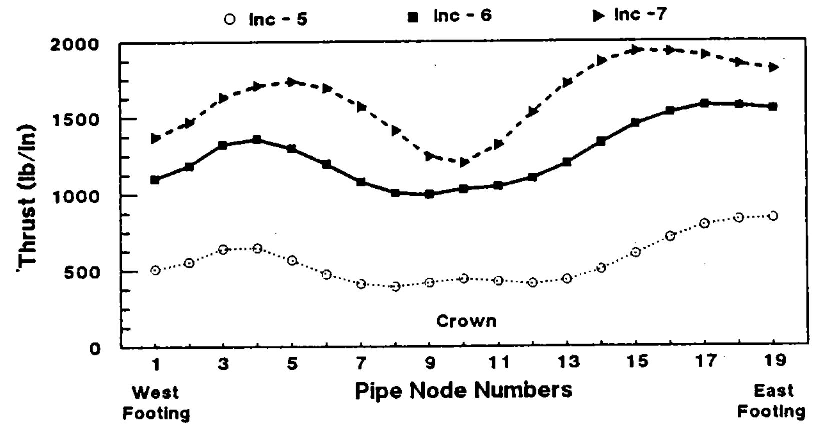

Fig. 17, Computed Thrust Forces (poor soil)

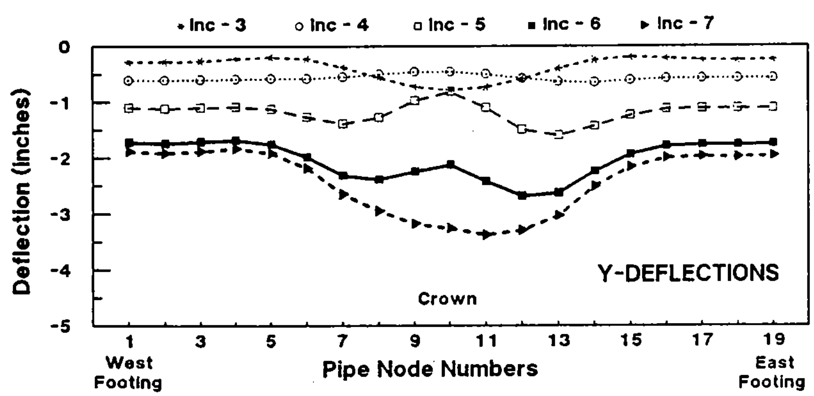

Fig. 16, Vertical Deflections (poor soil)

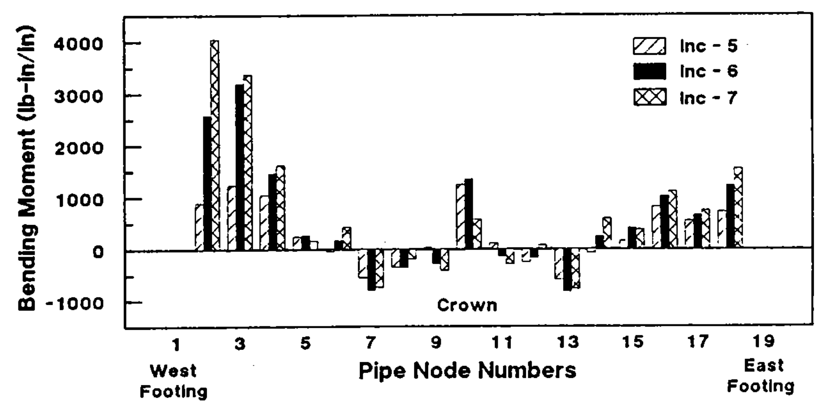

Fig. 18, Computed Bending Moments (poor soil)

HS-20-44 wheel were not computed directly, but may be estimated from other analyses, to be 5.7 and 3.5 for a thrust and buckling failure, respectively. These values are greater than the factors of safety recommended by AASHTO.

SUMMARY OF CPU TIME

The CANDE program was executed on an IBM 4381 operating under the VM/CMS operating system at the University of Idaho. It should be emphasised that although nine load cases have been presented in this paper, over 40 load cases were examined to gain an understanding of the deformation behavior of the Hayden Creek culvert. For any finite element analysis, this phase of familiarization is essential as two identical problems are rarely encountered in practice. The actual CPU times required to perform the reported nine load cases, presented in Tables 2 and 3, are summarized in Table 4, below.

Table 4, Summary of CPU Time for Analysis

	Load Case		CPU Time (minutes)
Elastic Analysis:	1	-----	2
	2	-----	2
	3	-----	3
	4	-----	5
Non-Linear Analysis:	1	-----	10
	2	-----	11
	3	-----	10
	4	-----	46
	5	-----	43

From Table 4, one can readily see that the CPU time for non-linear analyses may be up to nine times greater than that required for the linear elastic analyses. Since the cost of computer time on an IBM 4381 is usually in the \$ 1,000 - 1,500 per CPU hour range, the engineer proposing to use a finite element analysis should budget at least ten hours of CPU time. Thus a typical analysis may cost about \$ 12,500 just for computer time.

CONCLUSIONS

The following points summarize the finite element analyses :

1. Slight deformations in the shape of the culvert do not significantly affect the stability of the culvert as the factors of safety can be expected to be greater than 2.0.

2. Although direct comparisons are not really possible, the factors of safety determined using the linear soil properties were greater than the non-linear soil analyses.

3. For the half-section, non-linear analysis, the "poor" soil condition parameters generated the lowest factor of safety values.

4. For the full-section, non-linear analysis, the contrast between the "good" soil conditions and the soft layer under the west footing gave the lower factors of safety.

Finally, it should be recognized that a non-linear, finite element analysis is likely to require large amounts of CPU time. In view of the computational expenses, the finite element approach should only be used for analyses that must investigate the deformation behavior of the culvert. However, the authors believe that the finite element method may still be used regularly, with appropriately selected elastic parameters (i.e. constant soil moduli), to gain a general insight into the deformation behavior of the culvert.

ACKNOWLEDGEMENTS

This study was supported by the Idaho Transportation Department and the University of Idaho. The finite element analysis runs were performed by Li Jing, a doctoral student in the Dept. of Civil Engineering. This support and help is gratefully acknowledged.

REFERENCES

Duncan, J.M. (1979) Behavior and design of long-span metal culverts, ASCE, Vol. 105, No. GT3, March, pp. 399-418.

Duncan, J.M., Byrne, P., Wong, K. S. and Mabry, P. (1980) Strength, stress-strain and bulk modulus parameters for finite element analyses of stresses and movements in soil masses, Report No. UCB/GT/80-01, University of California, Berkeley, August, 70 pp.

Duncan, J.M. and Chang, C.Y. (1970) Non-linear analysis of stress and strain in soils, ASCE, Vol. 96, No. SM5, September, pp. 1629-1653.

Duncan, J.M. et al. (undated) Soil structure interaction program, SSTIP. University of California, Berkeley.

Duncan, J.M. et al. (undated) Nonlinear soil structure interaction program, NLSSIP. University of California, Berkeley.

Katona, M.G. (1978) Analysis of long-span culverts by the finite element method, Transportation Research Record 678, National Academy of Sciences, Washington, D.C., pp. 59-66.

Katona, M.G. (1982) Effects of frictional slippage on soil-structure interfaces of buried culverts, Transportation Research Record 878, National Academy of Sciences, Washington, D.C.

Katona, M.G. and Smith, J.M. (1976) CANDE user manual, Report No. FHWA-RD-77-6, Washington, D.C., October, 151 pp.

Katona, M.G., Smith, J.M., Odello, R.S. and Allgood, J.R. (1976) CANDE-a modern approach for the structural design and analysis of buried culverts, Report No. FHWA-RD-77-5, Washington, D.C., October, 466 pp.

Katona, M.G., Vittes, P.D., Lee, C.H. and Ho, H.T. (1981) CANDE-1980: Box culverts and soil models, Report No. FHWA/RD-80/172, Washington, D.C., May, 210 pp.

Leonards, G.A., Juang, C.H., Wu, T.H. and Stetkar, R.E. (1985) Predicting performance of buried metal culverts, Transportation Research Record 1008, National Academy of Sciences, Washington, D.C., pp. 42-52.

Leonards, G.A. and Roy, M.B. (1976) Predicting performance of pipe culverts buried in soil, Report No. JHRP-76-15, Purdue University, West Lafayette, May.

Leonards, G.A., Wu, T.H. and Juang, C.H. (1982) Predicting performance of buried culverts, Report No. FHWA/IN/JHRP-81-3, Purdue University, West Lafayette, June, 213 pp.

McVay, M.C. and Selig, E.T. (1982) Performance and analysis of a long-span culvert, Transportation Research Record 878, National Academy of Sciences, Washington, D.C., pp. 23-29.

FIRST WIRE ROPE NET ROCKFALL PROTECTIVE BARRIER INSTALLED AT THE GRAND CANYON NATIONAL PARK

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Vice President & General Manager
Brugg Cable Products, Inc., Houston, TX

INTRODUCTION

The wire rope net barrier at the Grand Canyon National Park was installed in order to provide protection against rockfall to the only pumphouse in the Grand Canyon and the working personnel in the immediate vicinity. Emphasis is placed on the design criteria and the construction of the defensive barrier. Additionally, the paper focuses on the installation of the system and the coordination of helicopter transport for labor and materials to the construction site. A brief discussion concerning the specification and the importance given by the National Park Service with regard to aesthetics, i.e., unobtrusiveness of the system in order to minimize the visual impact on the area.

LOCATION AND ACCESS TO THE CONSTRUCTION SITE

All material was shipped by common carrier via Arizona State Highway 89 to Jacob Lake and on from there using Highway 67 to the North Rim Project site. Due to bad weather conditions, the stretch of highway from

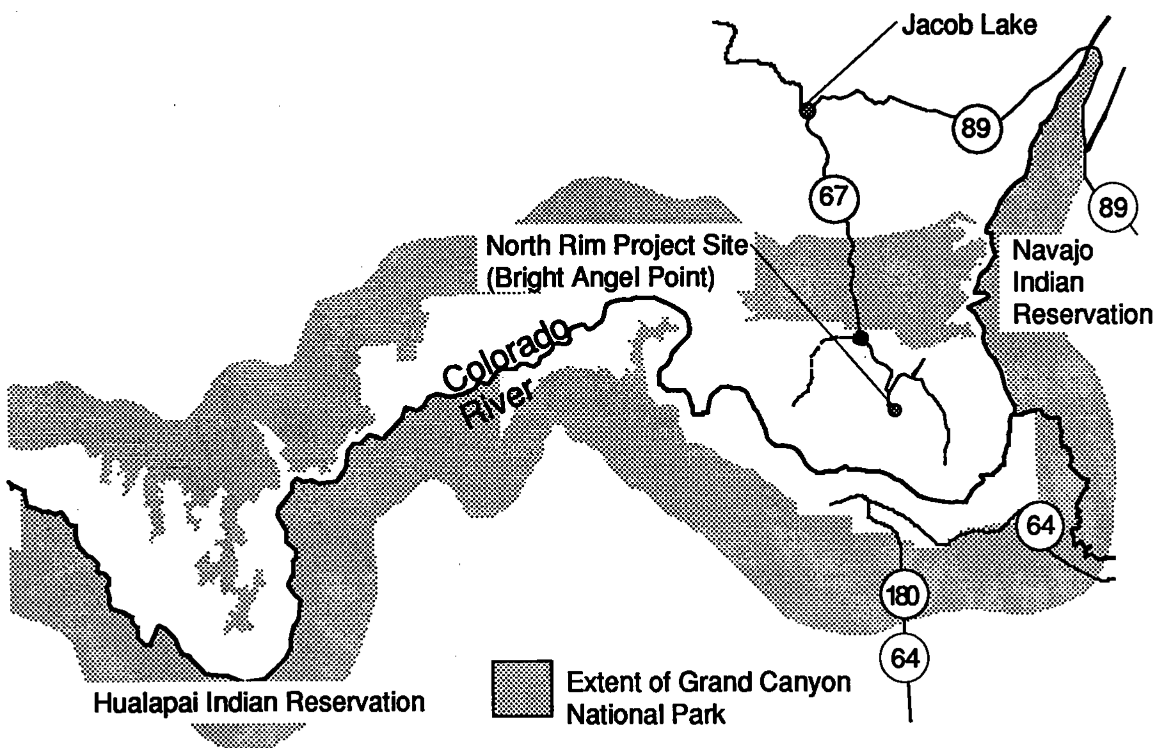


Figure 1 - Grand Canyon Map

Jacob Lake to the North Rim is closed around October or sooner and typically, does not open until May of the following year. Also, it should be mentioned that load restrictions are commonly imposed in the spring for this part of the road due to the unstable road foundation material.

The construction site near the Roaring Spring Pumphouse on the North Rim Unit of the Grand Canyon National Park is accessible only via helicopter or by a narrow trail. (Figure 1) Inasmuchas we only had two weeks to complete this installation prior to the closing, a decision was made to use a certified helicopter service for transporting the construction personnel and goods to the job site. The least expensive trail route using mules for transporting the installation material, etc. was out of the question because the installation could not have been completed in the time allotted.

ROCKFALL SYSTEM SPECIFICATIONS AND DESIGN

A fence having a length of 80 feet was selected by the National Park Service along the 4900 elevation contour about 46 feet away from the pumphouse. (Figure 2)

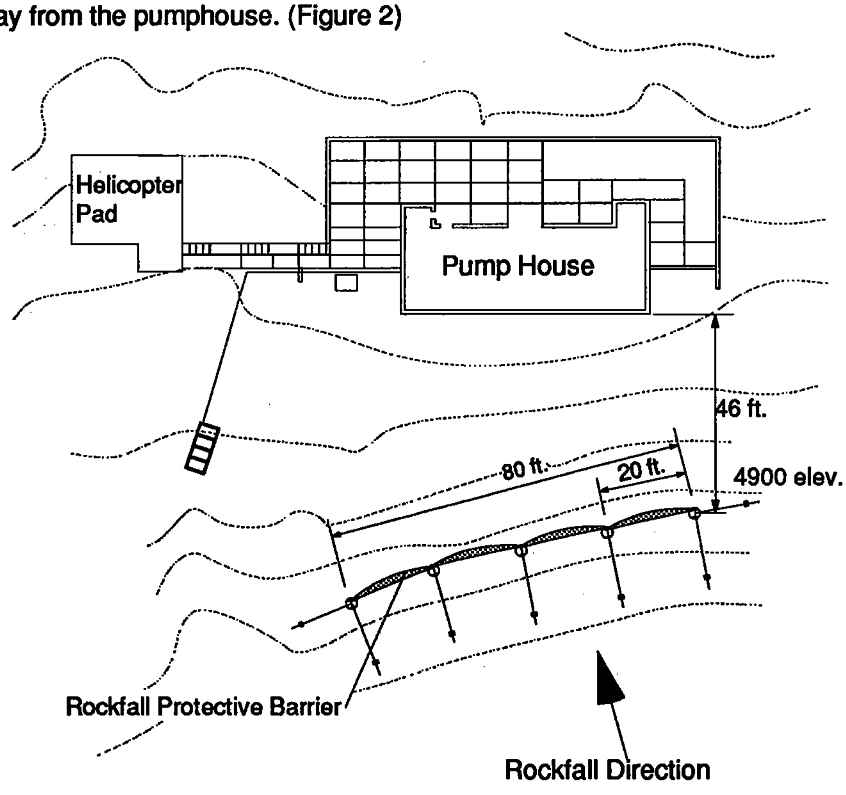


Figure 2 - Site Plan

The specification called for a lightweight system to minimize transportation cost. The barrier must be constructed from standard components for rapid and easy installation onsite. Maintenance must be low and the system shall be resistant to corrosion and terminal deterioration. Once the system has been installed it shall be painted with a corrosion-resistant paint, selected by the National Park Service Project Supervisor so that it would blend into the surrounding scenery.

The basic rockfall protection barrier design data is as follows:

Angle of Slope	:	37° to 42° from horizontal
Max. Weight of Rock	:	2,000 lbs.
Max. Impact Energy System Can Absorb	:	30 ft. Tons
Calculated Dropping Height of Rock	:	30 ft.
Assumed Max. Jumping Height of the Falling Rock	:	7 ft.
Max. Velocity in Relation to Max. Height and Acceleration Due to Gravity	:	43.93 ft/s
Ground Surface	:	Sand/Sandy Loam w/Frequent Cobbles
Vegetation in Surrounding Area	:	Juniper, Pinon Pine, Shrublike Oak

In accordance to the specifications, not only did the National Park Service request the submittal of structural and impact force calculations for the design load pertaining to the conditions indicated on the construction drawings, but in addition they also asked for test results conducted on rockfall nets of identical or similar construction. The tests must prove that the wire rope nets are capable of arresting the specified falling rocks without permanently damaging the net system.

Calculations from our engineering team pertaining to network dynamics and anchorage of the rockfall system and comparing these findings with those of the actual impact load data available from earlier test results, lent themselves to the system design as follows. (Figure 3)

The system chosen at the Grand Canyon Pumphouse, in accordance with our test results, should be capable of absorbing an energy of 94 ft. Tons, allowing for a design which provides more than adequate strength, easily absorbing the specified impact forces of 30 ft. Tons. It should be mentioned that tests which were conducted in our factory pertaining to nets of similar design, can absorb kinetic energies of as much as 252 ft. Tons.

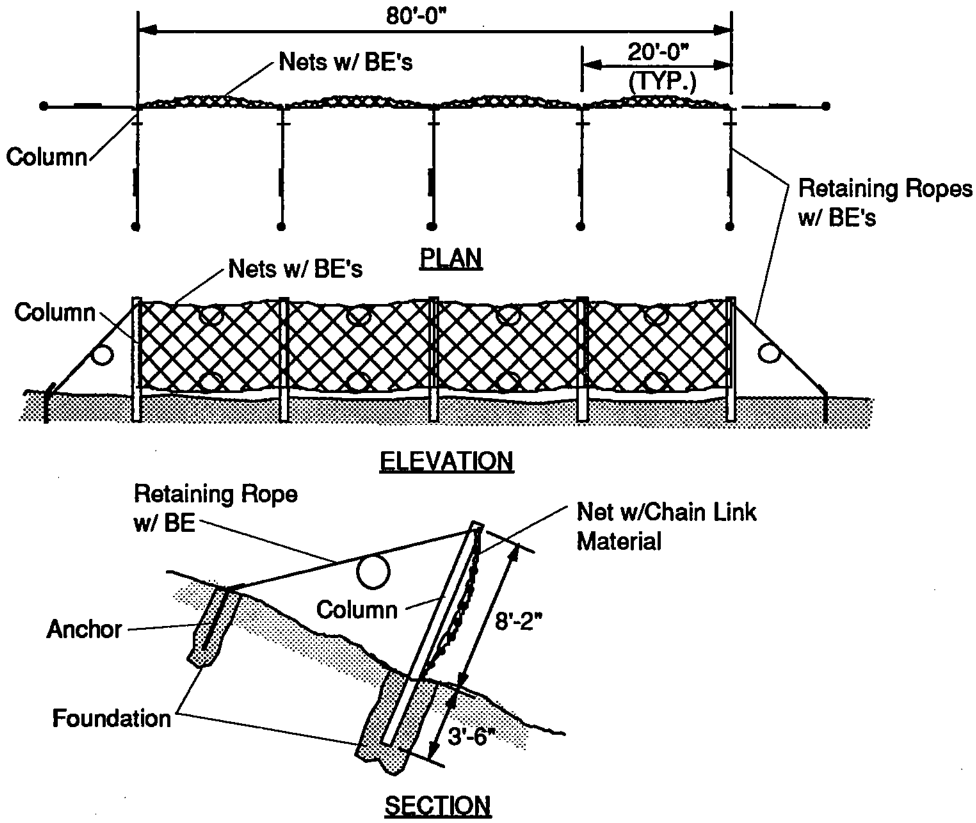


Figure 3 - Net Installation Details

TRANSPORT AND INSTALLATION OF THE ROCKFALL SYSTEM

As mentioned previously, all material and personnel had to be transported to the project site via helicopter. The loading helipad was located at the National Park Service North Rim. It was requested that all material be brought from the staging area to the helipad in a timely manner so as to not disrupt the daily flying schedule.

Luckily, the existing helipad at the pumphouse could be used for unloading of the personnel and goods. All material had to be immediately moved from the helipad to a designated storage area.

Once all the material was received at the jobsite, stacking-out of the system began immediately. The National Park Service provided the water, sewer and electrical hook-up for the duration of the installation. Digging of

Figure 4

Figure 5

the foundation for the columns and anchors had to be done mostly by hand which was quite time-consuming. Once this task was completed, the setting of the columns and concreting of same followed. (Figures 4 & 5)

Considering the climate and other difficult conditions, the installation progressed smoothly and on time. After the columns and the system retaining anchors were securely set in the ground, (Figure 6) the net supporting ropes (top and bottom), each incorporating braking elements, could then be strung between the columns. The system retaining ropes, each equipped with a braking element, were then secured to the wire rope anchors.

Once the wire grid was completed, the individual nets were then placed and seamed together with the wire ropes. (Figure 7)

Finally, commercial type cyclone fencing material was spread and fastened to the wire rope netting in order to hold back smaller rubble. The wire rope net having a larger mesh size, is designated for stopping heavier sized rocks.

The construction crew consisting of six and a supervisor, completed the installation in less than six days and just before the first snowfall appeared. In case the protective barrier could not have been completed according to the schedule, a work stoppage would have occured until late spring of the following year which would have been costly.

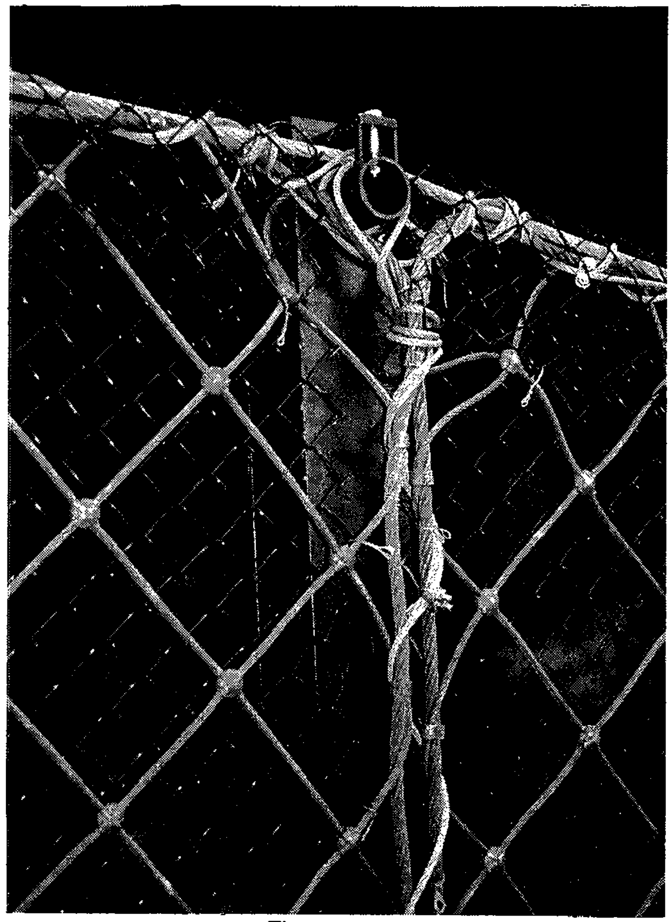


Figure 6

CONCLUSION

It should be noted that there is no system that can be installed that assures complete protection in all situations, however, with proper design and the selection of the best materials and construction as well as an awareness of the surrounding conditions such as those that existed with this project, a large portion of the danger can be avoided or substantially reduced. Of course, periodic inspections of the system should occur in order to ascertain what, if any, repairs are required. Also, the nets must be cleared of all rocks which might have accumulated from time to time.

Figure 7

In conclusion, we would like to give special consideration to the National Park Service for their effort in having specified a system which is not only most effective for the given situation, (Figure 8) but equally important, a system which is aesthetically and environmentally pleasing and blends into one of the Eight Wonders of the World !

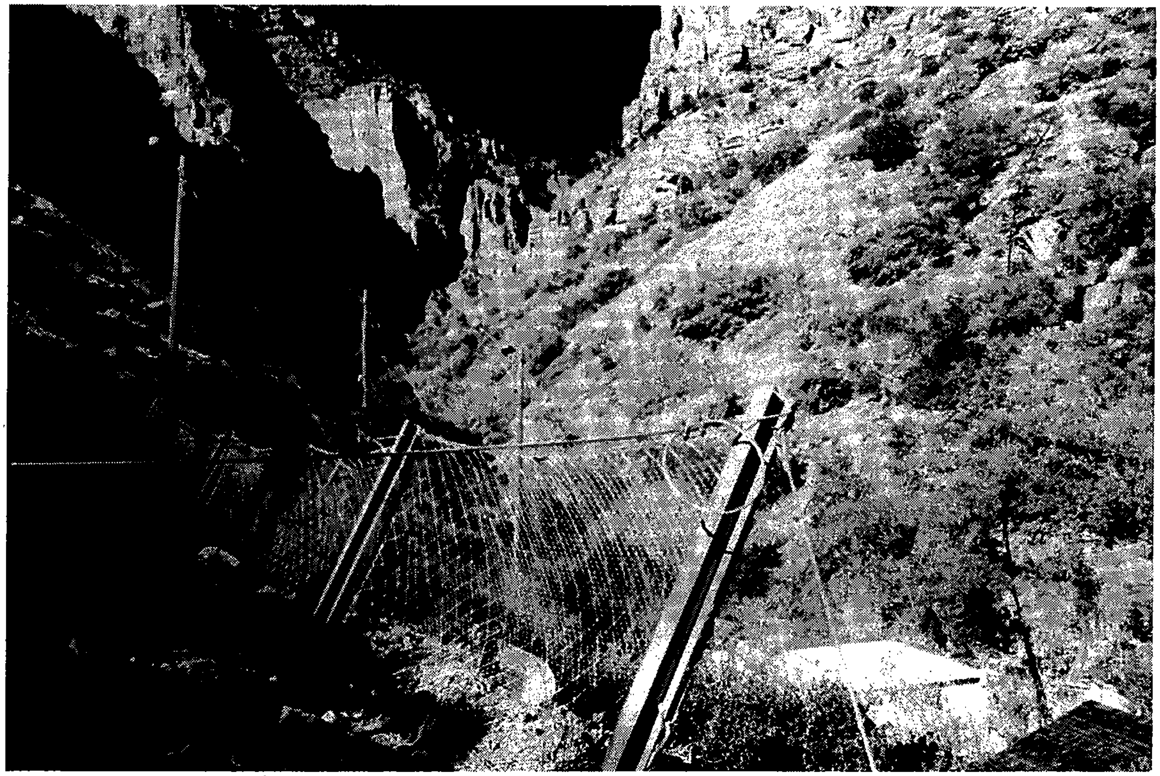


Figure 8

SPECIAL TREATMENT OF

MINE OPENINGS IN

ROCK CUT SLOPES

ABSTRACT

"SPECIAL TREATMENT OF MINE OPENINGS IN ROCK CUT SLOPES"

BY: E. M. Wright

KYDOH, Geotechnical Branch

Coal mining operations are very common in mountainous areas of Eastern Kentucky. New highway alignments are quite often projected thru areas where room and pillar mining methods have been utilized since the early thirties. Mining maps are not always available and when they do exist it is very difficult to correlate with surface features. The openings when left unsupported progressively weather and deteriorate, resulting in unstable rock cut slopes. Pneumatic backstowing with granular material is being recommended in the design phase as a method for providing support and long term stabilization of rock cut slopes.

A case history in Knott County and Kentucky's "Special Note for Mine Openings" are included in this paper.

INTRODUCTION

The primary industry in Eastern Kentucky is coal mining with Pike, Floyd, Perry, Knott, Martin and Johnson Counties being leading producers. Mining by contour strip, mountain top removal, augering, and room and pillar are the primary methods of coal extraction. Secondary recovery by pulling or robbing pillars is almost a standard practice in underground mine works. For economy and efficiency, longwall mining is becoming more common in a few areas. The thickness of mineable coal seams varies from 30 to 72 inches and in some areas multiple seam mining does occur. New highway alignments are quite often projected thru areas where room and pillar methods have been utilized since the early thirties. Mining maps are not always available and when they do exist it is very difficult to correlate with surface features.

All mine openings are not considered a problem. Small openings for air vents or entries, near grade, are usually backfilled to keep out trespassers and animals. As a general rule, rooms are not encountered within 30' to 50' from the edge of a hillside and structural support is not required. The size of the openings and presence of fractures are the primary factors influencing stability. The following case histories exemplify correction techniques of reconstruction, chemical stabilization, pneumatic backstowing, stabilization benches and excessive blasting damages. Refer to figure 1.

Reconstructed Cut Slopes

North of Prestonsburg on US 23, in front of Hardees restaurant, a vertical cut slope was reconstructed (in 1974) to include a 25' intermediate bench at the elevation of a mine opening. The rebuilt slopes, above the mine, were on 1/4:1 with 40' lifts. The opening was

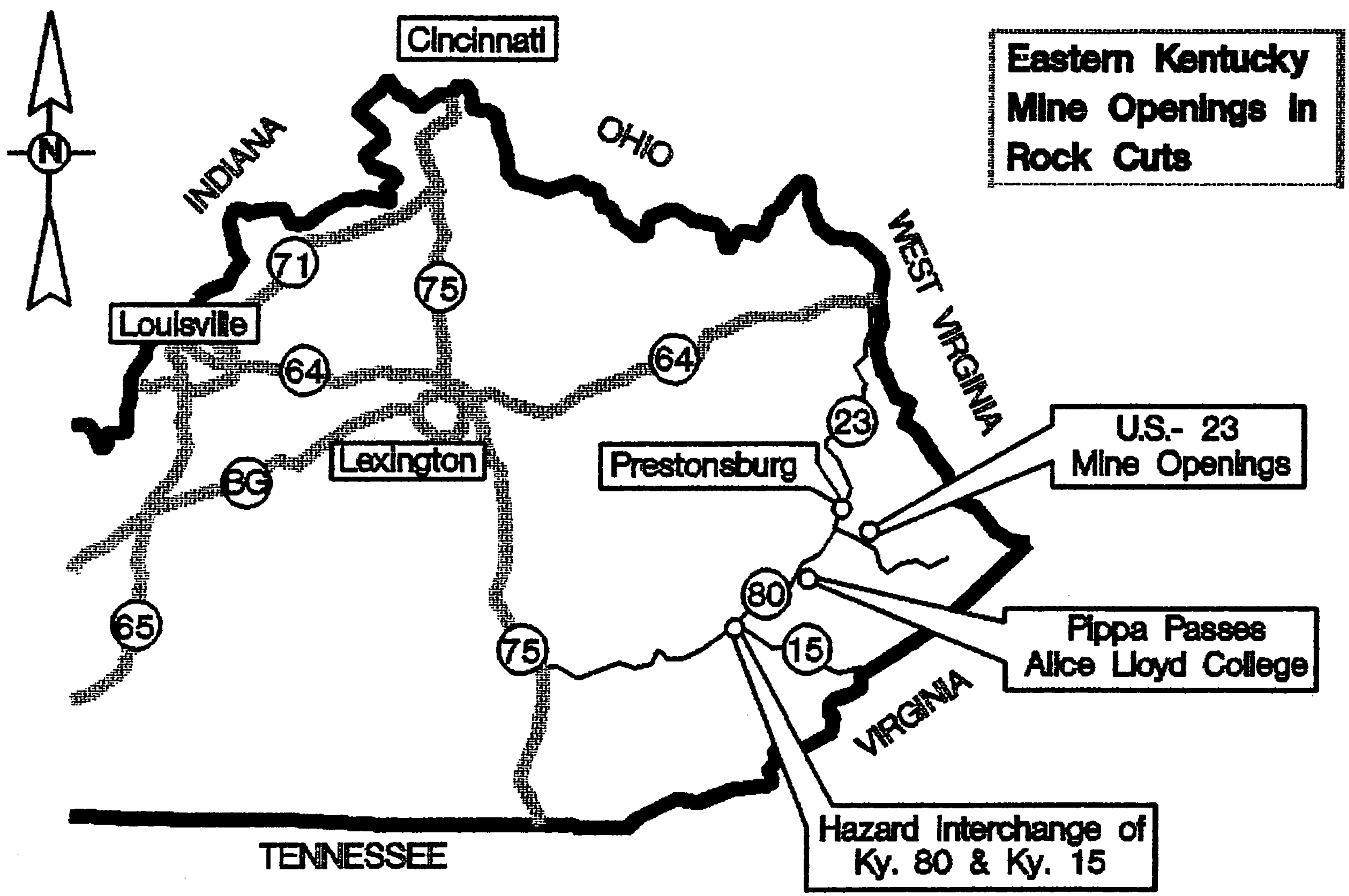

Eastern Kentucky
Mine Openings In
Rock Cuts
N
Cincinnati
INDIANA
OHIO
WEST VIRGINIA
Louisville
Lexington
Prestonsburg
U.S.- 23
Mine Openings
Pippa Passes
Alice Lloyd College
Hazard Interchange of
Ky. 80 & Ky. 15
VIRGINIA
TENNESSEE
71
75
64
64
23
BG
65
75
80
15

Figure 1.

not properly refilled and intersecting joint planes created a potential wedge failure. The cut has been weathering for approximately 14 years and the slope is beginning to show new fractures and bedding separation.

CHEMICAL STABILIZATION OF CUT SLOPES

South of Prestonsburg, on US 23, a shopping center being developed in Pin Hollow happened to be surrounded by abandoned mines in the Number 3 Elkhorn Coal Seam. The developer had a $10 million investment and wanted to utilize all available space by excavating steep slopes thru the mine areas. In 1985 construction was suspended because of unstable highwalls adjacent to the store locations. Due to legal and economic problems, the center was sold at a public auction for $2.8 million. Nesbitt Engineering was retained to redesign and stabilize the highwalls. Each section presented different problems because the north highwall intersected at a skew and the south wall was almost perpendicular to the openings. The highwalls were reconstructed to reduce the overall slope angle to 45 degrees by using 1:20 slopes in short lifts with variable width benches. Polyurethane foam was injected into the fractures to stabilize the cut. Specifications for the grout are given in a paper by J.S. Martin(1). This is the first attempt to use chemical stabilization methods in Kentucky and is particularly interesting. The total cost of the correction is approximately $1.2 million at this time, and the slopes are currently being monitored for stability.

STABILIZATION BENCHES

Southeast of Prestonsburg on US 23 towards Pikeville one of the long highway cut sections was back into the mountain far enough to encounter a mine. The orientation of the openings, joint systems and direction of the roadway was at a skew. The cut slope directly above the mine is continually failing; however this is a 30' lift with a 25' intermediate bench. At this time the overall stability of the cut is acceptable and most of the rockfall is retained on the designed stabilization bench. The concept of wider intermediate benches below mine openings appears to have merit in this situation.

EXCESSIVE BLASTING DAMAGE

The rock cuts in the intersection of KY 80 and KY 15, near Hazard, are an example of strip, auger and underground mines encountered in highway construction. The section exposed is approximately 350' deep and is a popular place for studying the eastern Kentucky coal producing formations. The cut is situated in a favorable direction (perpendicular) for orientation of mine entries, joint systems and trend of the highway. One of the cut sections bisects the mountain and exposes rooms and pillars of a mining operation. This is an excellent location to observe subsidence problems and the long term effects of blasting damage due to construction of the highway. In addition to normal blasting damage, an attempt to collapse the openings by blasting the pillars during construction has made the cut very unstable and is accelerating the progressive slope failures. At this time, maintenance forces are using a "Do Nothing" alternate until the rocks fall in the roadway. Geological descriptions of the formations are described in the Annual Geological Society of American Coal Division Field Trip, November 1981.(2)

PNEUMATIC BACKSTOWING

A project located in Knott County, referred to as the Alice Lloyd College Bypass, is the first attempt the Department has made in utilizing pneumatic backstowing methods for stabilizing rock cut slopes above collapsing mine entries. A one-room school, started in 1917, expanded on Caney Creek and eventually developed into a four-year college. The campus was bisected by KY 899, a narrow roadway that carried coal trucks and a significant amount of traffic. The roadway bypass was designated as an economic development project, and the schedule was accelerated after one of the coeds was abducted from the campus and murdered.

The proposed route included two deep cut sections and very limited waste areas. In an attempt to reduce the costs, the grade was increased to 10 percent with average lift heights of 40 feet. The design was for 1:20 slopes in sandstone and 1/4:1 in siltstones. Near the end of the project coal mine adits in the Upper Elkhorn #3 coal zone were observed, at elevation 970, and the alignment was adjusted to avoid the openings. The Amburgy coal zone (elevation 1090) was encountered in five core holes and voids were not present. There was no indication of mining activity on the ground surface.

Melco-Greer Incorporated was low bidder for the project and a contract was awarded for $6,968,828.23. One location in the middle of the project was approved for a waste site which would accommodate 1 million cubic yards of material. Cost overruns occurred when two large overburden slides had to be stabilized. In the process the Department was required to buy one gas well.

After the 180' cut was opened, six large openings in the Amburgy coal zone were encountered about 20' above grade. The contractor followed standard procedures for blocking off the adits by refilling with loose material.

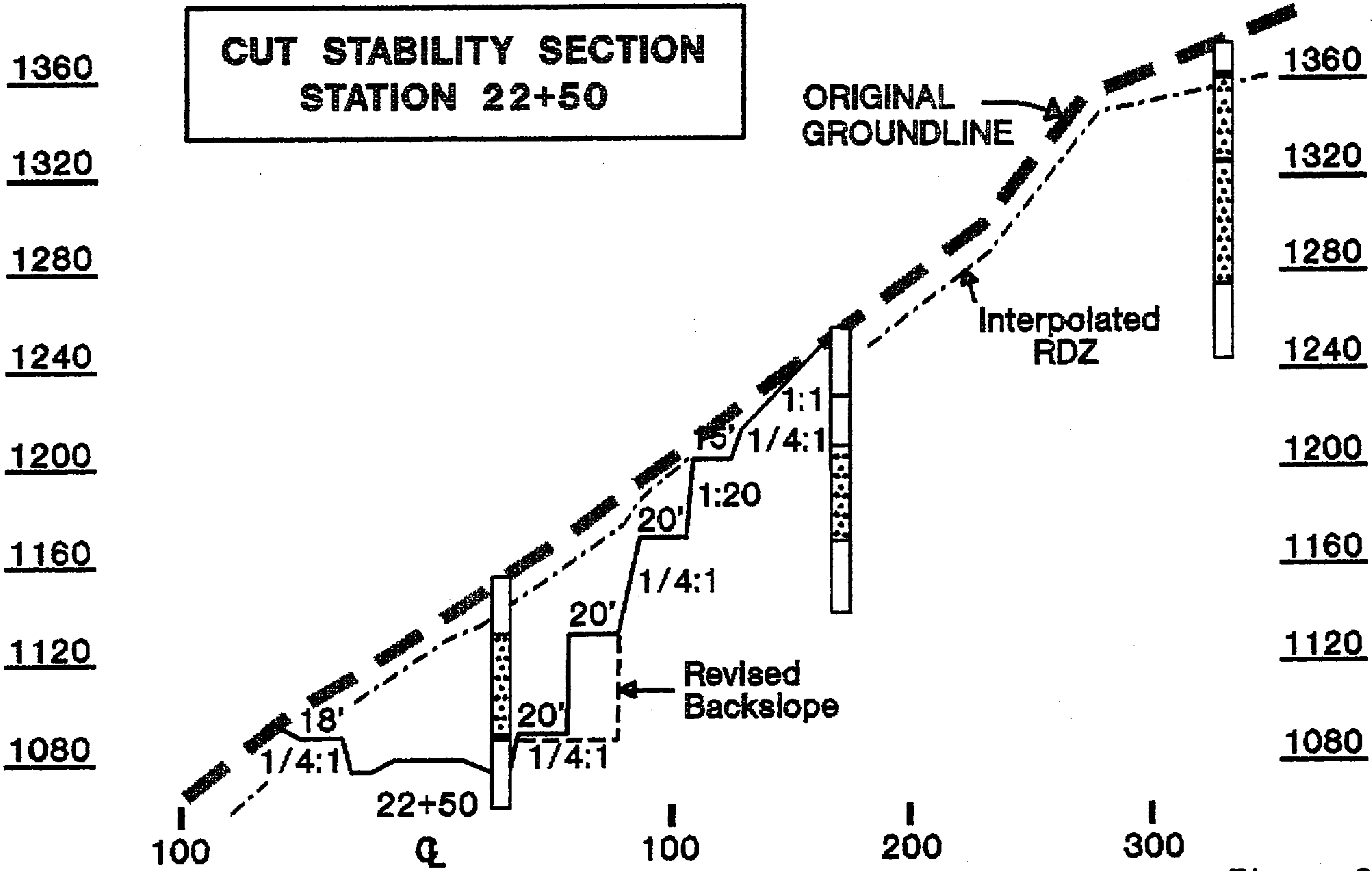
This did not provide vertical support for the overlying massive bedded sandstone. When the lateral support was removed the slope began progressively breaking up. The resident engineer called for assistance on October 31, 1988 and Geotechnical personnel suggested three alternates.

1/. Redesign the cut from elevation 1360 down to elevation 1090 by flattening slopes.

2/. Leave the cut slope and assume that the entire slope would not fail. This is a "do nothing" alternate.

3/. Remove the unstable material and backstow all openings with #57 limestone. (Refer to figure 2). Alternate 1 was unacceptable since the project had already encountered a large cost overrun, unstable overburden slopes could be encountered, and the next available waste site was 1 1/2 miles down the road. The additional cost could amount to another 2 million dollars. Alternate 2 was not favored because the cut was progressively failing daily and a massive failure threatened the campus area below. Alternate 3 was agreed to since the situation was critical and an immediate solution was required to meet the emergency, and at the same time protect the seven million dollar investment.

Personnel from Micon Services Incorporated, Pittsburg, Pennsylvania, reviewed the project and advised the contractor and the Department that their services of pneumatic stowing of the openings would provide controlled,



CUT STABILITY SECTION
STATION 22+50
1360
1320
1280
1240
1200
1160
1120
1080
ORIGINAL
GROUNDLINE
Interpolated
RDZ
1:1
15'
1/4:1
1:20
20'
1/4:1
20'
Revised
Backslope
20'
1/4:1
18'
1/4:1
22+50
100
℄
100
200
300

Figure 2.

full-contact roof support to prevent the backslope from collapsing and could be performed safely outside the mine.

The equipment utilized by Micon consisted of a 350 KVA diesel generator, a Holywell pneumatic stowing machine, eight-inch steel and plastic stowing pipe with roustabout couplers, a 3.5 cubic yard hopper with vibrating syntron, a grout piston pump and an air compressor. The pneumatic stowing machine is an electric-powered blower which produces a high volume of air (3600 cfm) at low pressures (6-8 psi). Aggregate is fed into the hopper with a backhoe or grout bucket and a combination of plastic and steel pipe is used to transport the material into the mine entries. A grout pump and air compressor are used to inject the cement (wet or dry) into the stowing pipe.

The Department negotiated with the contractor to stabilize the cut by removing approximately 9,000 cubic yards of fractured sandstone at $18.23/cubic yard and a lump sum of $25,350.00 for backstowing the mine openings. Removal of the highly fractured sandstone was a safety requirement before Micon could work on the mine openings under that part of the cut. During this operation a large, open, horizontal subsidence crack about 75' long x 2' wide x 20' back under the remaining backslope was exposed. The excavation was discontinued at that time and Micon was requested to backstow the opening immediately with dense grade material and 7% cement to prevent the progressive collapse of the backslope. The decision to use dense grade material was based on the assumption that the additional fine material could fill any smaller joints or fractures. This proved to be a problem. The material was wet and stuck to the hopper and stow pipe walls. The stowing rate was reduced because capacity was being used to move water, and the lift height was increased to approximately 60' above grade.

Complete backfilling of the dimensions of this opening required the pipe to be moved systematically from side to side on a very narrow bench. After the opening was about half full, the material was changed to number 57 limestone and the production rate doubled.

The remaining mine openings were stowed with number 57 limestone with 7% cement added to the last 5' to provide a good facing. Perforated 8" drainage pipe was added to each opening to prevent build up of water pressure.

The final cost for stabilizing this cut was:

8" perforated pipe	180'LF @ $10.00 =	$ 1,800.00
Special Bench Excavation	9843 CY @ $18.23 =	$179,437.89
Backstowing Mine Openings	LS @ 25,350.00 =	$ 25,350.00
Special Mobilization (crane)	LS @ 6,000.00 =	$ 6,000.00
Special Demobilization (crane)	LS @ 6,000.00 =	$ 6,000.00
	Total	$218,587.89

Conclusion

At this time the Department has concluded that pneumatic stowing of abandoned mine entries is a reliable method of stabilizing cut slopes. The costs can be reduced considerably by:

1/. Backfilling the entries as soon as they are exposed.

2/. Using excavated sandstone (less than 1 1/2 inch diameter) from the construction project.

SPECIAL PROVISION FOR MINE OPENINGS

As a result of this project and others the Department developed a Special Provision for construction procedures when abandoned mines are encountered on highway projects. The notes are as follows:

Any vertical mine or air shaft under a proposed embankment, whether shown on the plans or not, shall be filled with broken stone (sandstone, limestone or rock-like shale SDI is 95) from roadway or borrow excavation and capped with an 8-inch reinforced concrete slab. The slab shall be in accordance with Section 711 of the current Standard Specifications for Road and Bridge Construction.

Any mine tunnels or horizontal auger openings in mined out areas below grade which show signs of subsidence, whether shown on the plans or not, shall be thoroughly investigated at the direction of the engineer by rock coring, probing or other means. The openings shall be collapsed, or undercut and backfilled with broken stone (sandstone, limestone, or rock-like shale SDI 95 or greater) from roadway or borrow excavation.

The material shall be backfilled in accordance with Section 207 Embankment. At the direction of the engineer, pneumatic backstowing of broken stone (sandstone or limestone maximum size 1 1/2") may be utilized to backfill openings which are inaccessible or difficult to backfill by other means. If feasible, positive drainage of the tunnels or openings shall be provided through use of pipe underdrains or other suitable drainage facilities. Payment for this work shall be in accordance with Section 104.03 or 109.04 "Extra Work" in the current Standard Specifications for Road and Bridge Construction.

Any mine tunnels or horizontal auger openings which are exposed in cut slopes, whether shown on the plans or not, shall be backfilled a minimum distance of 20' from the face of the cut. To insure that the void is completely backfilled, pneumatic backstowing with broken stone (sandstone or limestone maximum size 1 1/2 inch) is required. If feasible, positive drainage of the tunnels or openings shall be provided through the use of pipe underdrains, surface ditches or other suitable drainage facilities. Pipes and other material used for drainage shall be paid for at the unit bid price for those items. Pneumatic backstowing or other special equipment shall be paid for in accordance with Section 104.03 or 109.04 "Extra

Work" current edition of Standard Specifications for Road and Bridge Construction.

CUT SLOPE DESIGN PROCEDURES INVOLVING MINE OPENINGS

Prior to designing new highways the area is to be completely reviewed. This review will always begin with a United States Geological Map (1"=2000' scale) which provides an excellent source of information for geological formations and coal zones. If coal mining has occurred on the alignment, special consideration is applied to the design of a rock cut section. The concept for a cut slope is derived from the geological review which contains information in a format consisting of the following:

1/. General Geology

Lithology of Rock Overburden

Percent of Sandstone in the Overburden

Dip of the Beds

Gradational Changes in Lithology (vertical & horizontal)

Type of Roof and Floor of Mine

2/. Engineering Geology & Properties of Overburden

Depth of Overburden

Rock Quality Designation of Sandstone and Siltstone Beds

Joint Spacing (vertical & horizontal)

Direction and Angle of Joints

Continuous or Discontinuous Fractures or Joints

Durability of the Shales

Presence of Clastic Dikes

Structural Domain (limits of influence)

3/. Mining Data

Type of Mining Method (room & pillar or longwall)

Elevation of Mine

Size of Mine Openings

Pillar Dimensions

Percent of Extraction

Age of Mine Works

Roof Fall Records

4/. Site Conditions

Orientation of Mine Openings to Highway Direction

Presence or Accumulation of Water

Position of Mine Openings in the Mountain

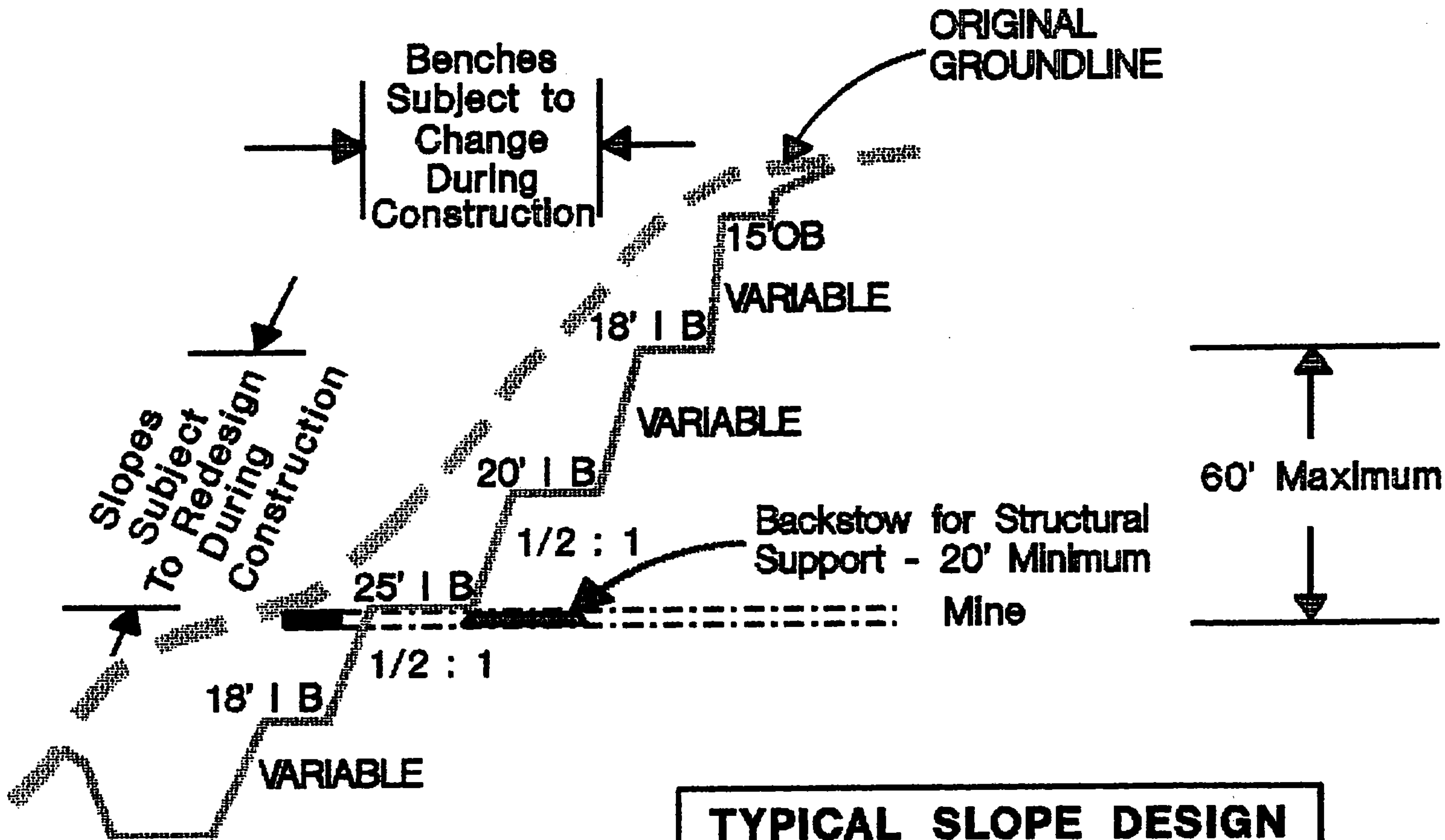
History of Mine Subsidence in the General Area

Multiple Seam Mining

Anticipated Blasting Damage During Construction

Fires

The factors are generally the same ones used in designing a rock cut slope except that interpretation of mine data combined with site conditions are usually the basis for including safety features which permit adjustments during construction. Figure 3 illustrates the typical design which is normally utilized. The 25' intermediate bench at the mine opening and the next two lifts above (maximum 60') are subject to be redesigned during construction if necessary. The slope directly above the opening is recommended to be 1/2:1 because



TYPICAL SLOPE DESIGN FOR MINE OPENINGS IN CUT SECTIONS

the 63^{o} slope angle is usually flatter than the subsidence fractures. Pneumatic backstowing with number 57 (sandstone or limestone) a minimum distance of 20' into the opening provides structural support and prevents disintegration of the cut slope. When the openings are sealed, positive drainage is provided if feasible.

Acknowledgements

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References

1. J. S. Martin and D. A. Newman, "The Characterization and Redesign of A Failing Highwall in Eastern Kentucky" Society of Mining Engineers Preprint Number 88-80.
2. J. C. Cobb, D. R. Chesnut, N. C. Hester, J. C. Hower "Coal and Coal Bearing Rocks of Eastern Kentucky" Annual Geological Society of American Coal Division Field Trip, November 1981.
3. S. A. Zaburunov "Controlling Subsidence with Stowing A Case History" Reprint from Coal Mining, September 1986.
4. D. Mason "Auger Hole Stowing Allows Deep Mine Entries" Reprint from Coal Mining, December 1986.

RECENT HIGHWAY TUNNEL PROJECTS IN THE APPALACHIAN MOUNTAINS

Richard Humphries* and Randy Sullivan*

ABSTRACT

The upgrading of the infrastructure in the Appalachian Mountains requires the construction of new tunnels and improvement of some of the existing tunnels. Most of the tunnels are at shallow depth in good or moderate quality rock, so the ground behavior is structurally controlled, though there is also the potential for slaking and overstressing on some projects. Advances in rock engineering have resulted in a better understanding of ground behavior and more cost effective investigations, excavation methods and rock support. This paper describes some of the geotechnical aspects of five tunneling projects in the area where the use of modern methods has resulted, or will result, in significant cost savings. The five projects are:

- The twin highway tunnels at Cumberland Gap between Kentucky and Tennessee;
- The stabilization of the 100-year old "Little Tunnel" as part of the Cumberland Gap Project in Tennessee;
- The proposed Crooked Arm Ridge Tunnel on the Foothills Parkway in Tennessee;
- The upgrading of two of the 50-year old tunnels on the Blue Ridge Parkway, North Carolina; and
- The proposed River Diversion Tunnels at Harlan, Kentucky.

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INTRODUCTION

Many new roads and road improvement projects are under construction or planned in the Appalachian Mountain Region. Because of the mountainous topography, many projects involve the construction of new tunnels. In addition, much of the infrastructure is old and requires upgrading and several of the existing tunnels require additional support due to deterioration of the rock over time.

This paper describes some of the geotechnical aspects of five of the recent tunneling projects in the Appalachian Mountains. It describes the ground behavior at each of the projects and how cost savings have been possible from the use of modern rock engineering.

GROUND BEHAVIOR AND SUPPORT DESIGN

The types of ground behavior in rock tunnels are usually divided into the following five categories (Reference 1):

1. Loosening or structural control: rock displacing under its own weight along pre-existing discontinuities;

2. Stress-slabbing: the formation of new fractures as a result of stress concentrations around the tunnel opening;

3. Squeezing: creep without perceptible volume change;

4. Swelling: volume increase; and

5. Slaking: volume change and deterioration due to changes in the physical environment.

These types of ground behavior are not unique properties of the rock, but depend on the size and shape of the opening, the stress level, humidity, groundwater conditions, and time. Actual ground behavior often involves a combination of several of these categories.

In general, in the Appalachian Mountains, the highway tunnels are at shallow depth and the rock is good or moderate quality so the ground behavior (or potential mode of failure) is structurally controlled, i.e., blocks of rock bounded by pre-existing discontinuities (joints, faults, shear zones, bedding partings, etc.) which are exposed by the excavation, fall or slide into the tunnel. The most effective ground support for this behavior is to install active support (rockbolts and shotcrete) to limit the deformation or loosening of the rock mass and, thus, retain the interlocking and enable the rock to support itself (Reference 2). The traditional types of tunnel support, which include steel arches and timber sets, are not suited to this type of ground behavior because they provide passive support and allow significant deflection to occur before they apply a support load. This allows

Richard Humphries
Randy Sullivan

the rock mass to loosen, reduces its interlocked strength and increases the load on the supports.

Squeezing is usually associated with soft argillaceous rocks. Swelling is typically associates with montmorillonitic rocks salt deposits. None of these are common in the Appalachian Mountains and are not present at any of the five sites. However, there is the potential for stress-slabbing and slaking on some of the projects. Stress-slabbing occurs when the stresses in the rock surrounding the tunnels, induced by the excavation process, exceed the strength of the rock mass. This induces new fractures in the intact rock. To prevent failure and subsequent collapse, a flexible support is required to "knit" the rock mass together and to prevent failed zones of rock from falling into the tunnel. Again, the most appropriate form of support is a combination of rockbolts, dowels and shotcrete. The rockbolts or dowels are usually anchored in the zone beyond the zone of overstressing.

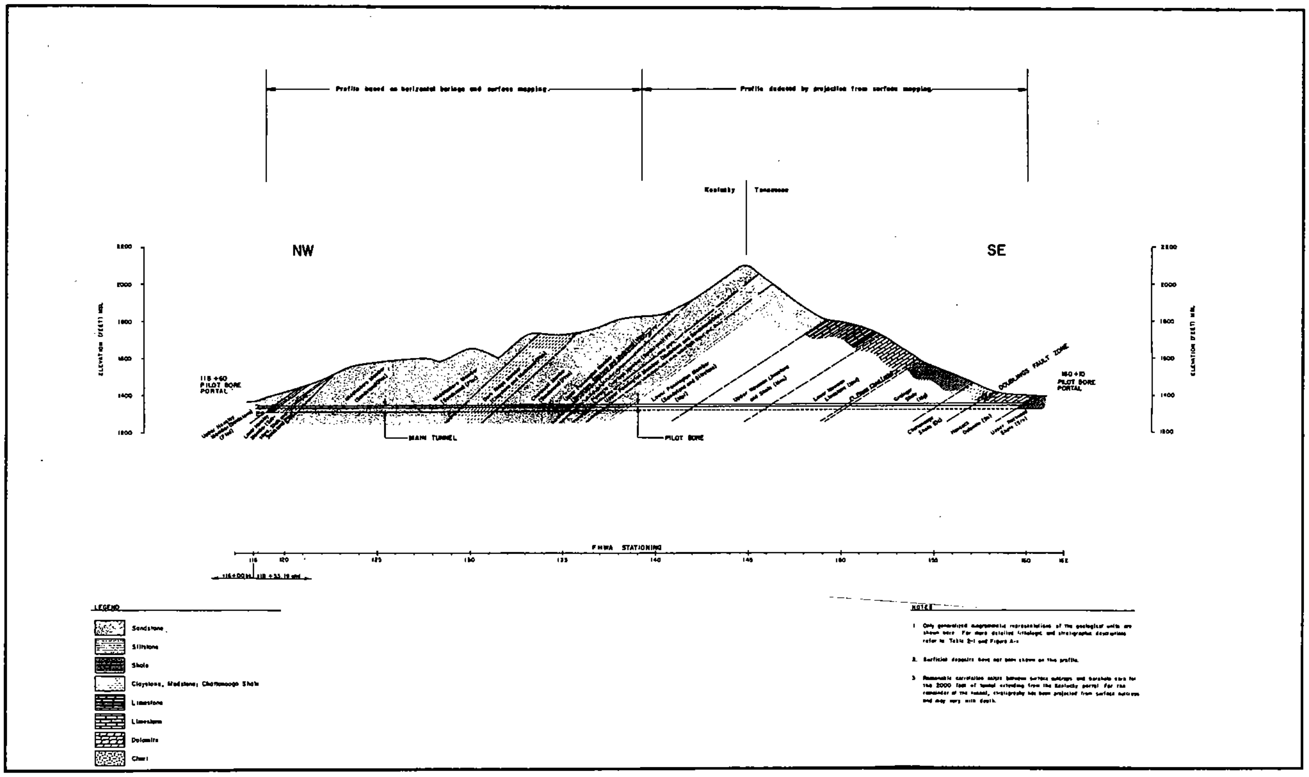
Slaking is common in many of the shaley rocks in Appalachia. It occurs when argillaceous rocks are exposed to changes in moisture content, freeze-thaw cycles or changes in confining pressure and results in volume change and deterioration of the surface. An effective way of preventing slaking from occurring is to apply a layer of shotcrete, often in combination with rock bolts or dowels, soon after the rock is exposed, to limit the changes in the moisture content.

CUMBERLAND GAP TUNNELS

Twin highway tunnels, 4,100 feet long, through Cumberland Mountain form the main part of the Cumberland Gap Project on U.S. Highway 23E between Cumberland Gap, Tennessee and Middlesboro, Kentucky. The Federal Highway Administration (FHWA) is serving as the design and construction manager for the project, on behalf of the owner, the National Parks Service. The twin tunnels will be about 4,100 feet long and will have a circular arch shape with an excavated width of about 44 feet and a maximum height of about 36 feet. Excavation of the portals is currently (1989) in progress and excavation of the tunnels is planned to start towards the end of 1989.

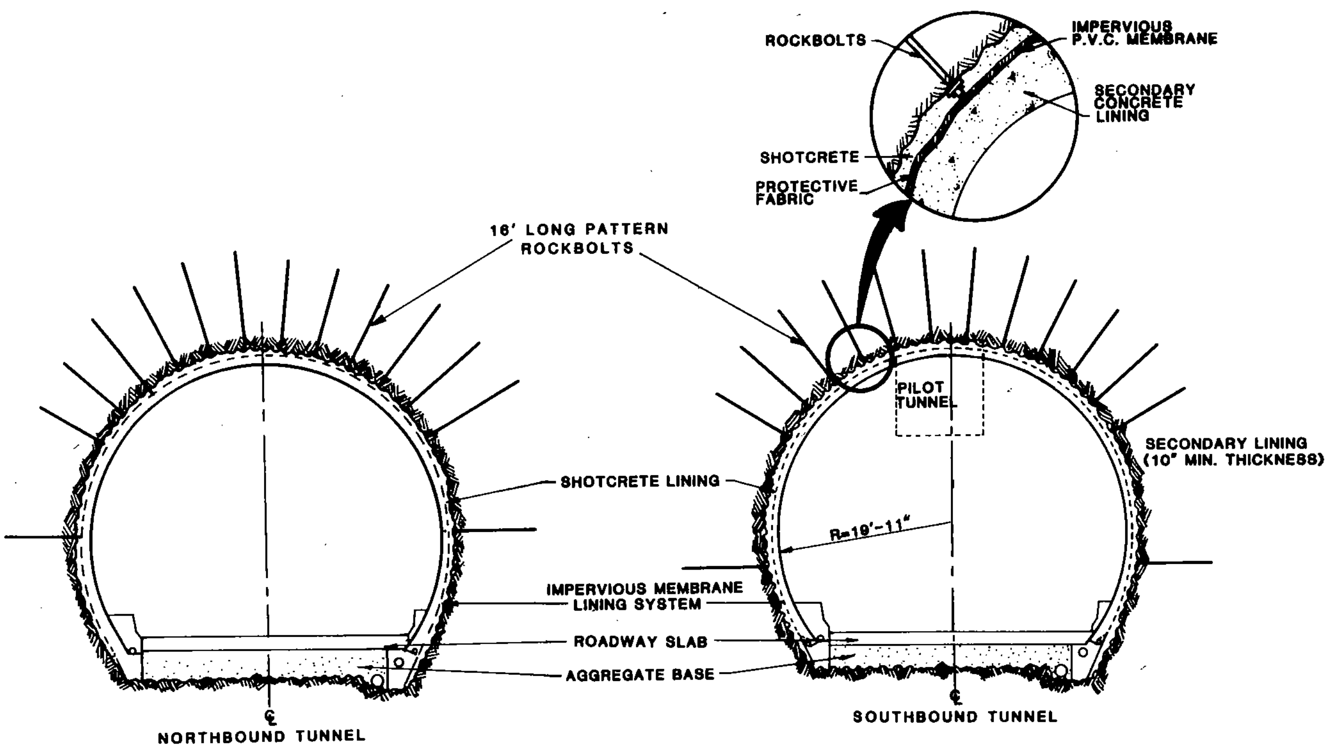
The tunnels will be excavated in sandstones, limestones, and shales, which dips at 35 to 50 degrees and strike nearly perpendicular to the tunnel alignment, as shown in Figure 1 (Reference 3). In 1985-86, a pilot tunnel was excavated as a crown drift along the alignment of the southbound tunnel. The objective of the pilot tunnel was to investigate the rock conditions along the tunnel alignment and to reduce the uncertainties for the construction of the main tunnels. The pilot tunnel has made it possible to directly observe and measure the following:

- stratigraphy;
- major structural features, such as faults and shears;



CUMBERLAND GAP TUNNELS - GEOLOGIC PROFILE

FIG. 1



CROSS-SECTION THROUGH COMPLETED TUNNELS.

CUMBERLAND GAP

FIG. 2

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Randy Sullivan

- weathering and alteration;
- orientation, roughness, persistence, spacing, and infilling characteristics of typical discontinuities sets;
- intact compressive strength;
- groundwater inflows; and
- the response of the rock mass to excavation.

The pilot tunnel has shown that more than 90 percent of the main tunnels will be driven through medium to very strong rock (uniaxial compressive strength of greater than 5,000 psi) that will provide excellent anchorage for rockbolts. The ground behavior in the main tunnels will involve loosening along pre-existing discontinuities, plus slaking in the shale, claystone, mudstone, and siltstone units. Stress-slabbing, squeezing and swelling were not observed in the pilot tunnel and are not expected in the main tunnels.

The pilot tunnel has also shown that there are short sections where the ground is too soft to provide anchorage for rockbolts and where the need for immediate positive support is required to insure the safety of workers at the tunnel heading. These soft zones range up to a maximum of 30 feet in length along the tunnels and will be supported by a combination of lattice girders and shotcrete.

The main tunnels are currently (1989) in the design phase. They are planned to be excavated by the drill-and-blast method, though a short section on the Tennessee side may be excavated by roadheader. They will be excavated by the top-heading-and-bench method, with the top heading probably excavated as two or three blasts. The water proofing of the tunnels will be provided by a PVC membrane which will be supported by a secondary concrete lining, as shown in Figure 2.

LITTLE TUNNEL, CUMBERLAND GAP, TENNESSEE

The approach roads to the Cumberland Gap Tunnel in Tennessee are being constructed over a 100-year old railroad tunnel within the town of Cumberland Gap. This tunnel, which has not been in service for some years, is supported by a brick arch lining for the western third of the tunnel which was excavated in Rockwood shale, and timber sets and lagging for the eastern two-thirds of the tunnel which was excavated in limestone. As part of the work on the Cumberland Gap project, a study was made of the condition of this railroad tunnel, which is called "Little Tunnel," to see if it could safely house utilities. The brick lining was generally found to be in excellent condition as were most of the timber sets and lagging. However, there were a few areas where the timber sets had rotted significantly or were showing signs of distress from overloading.

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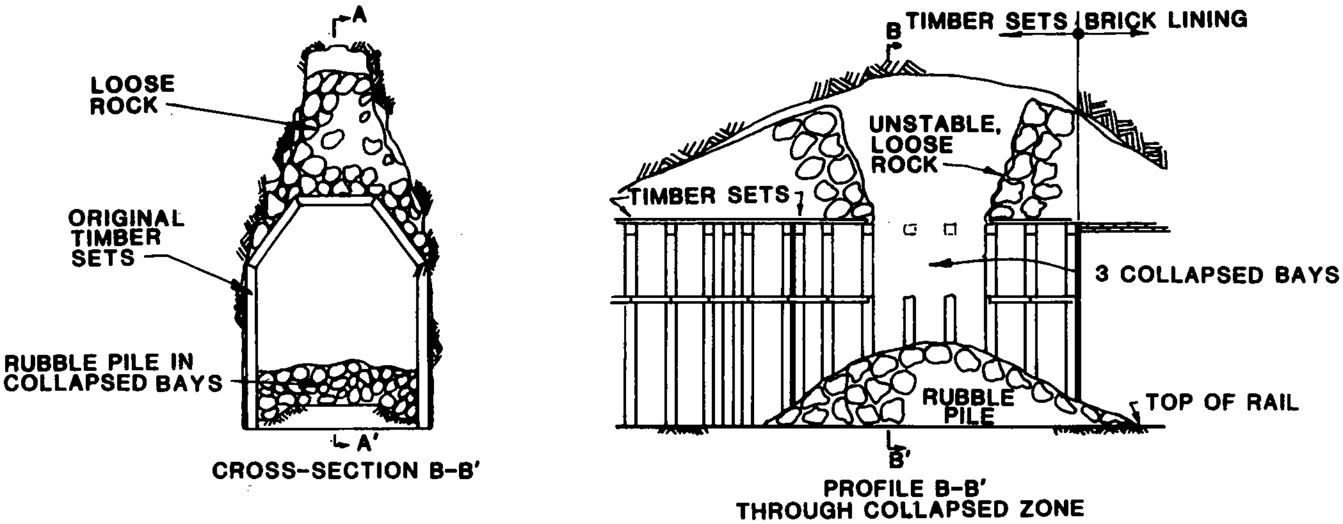
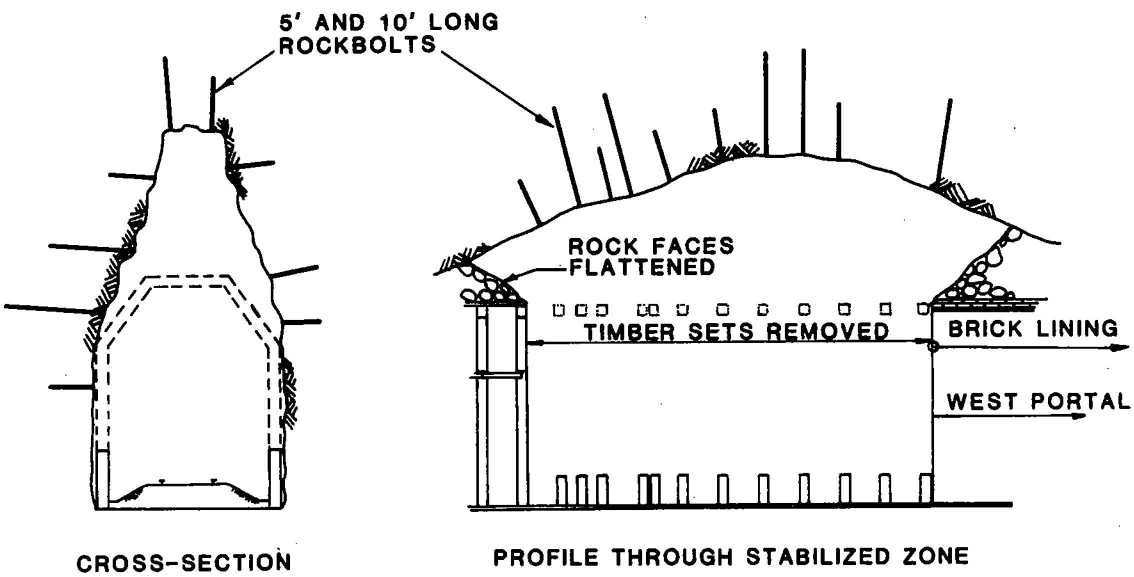
Randy Sullivan



INITIAL SUPPORT & COLLAPSE IN LITTLE TUNNEL

FIG. 3



STABILAZITION OF COLLAPSED ZONE IN LITTLE TUNNEL

FIG. 4

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Shortly after the condition survey was made, a collapse occurred in one of the areas that had been identified as a problem area. A section through the collapsed zone of the tunnel in shown in Figure 3. This section of the tunnel is in a limestone which has a high unconfined compressive strength and weak bedding partings which dip near horizontally. With a maximum rock cover over the tunnels of approximately 90 feet, the stability of the tunnel is controlled by the structure of the rock. The timber sets, which provided a passive resistance to failure, allowed the rock above the crown of the tunnel to loosen, lose its interlocking, and stope up to form a stable arch as shown in the cross-section in Figure 3. The loosened rock collapsed onto the timber sets, as shown, and the gradual deterioration of the wood resulted in the collapse.

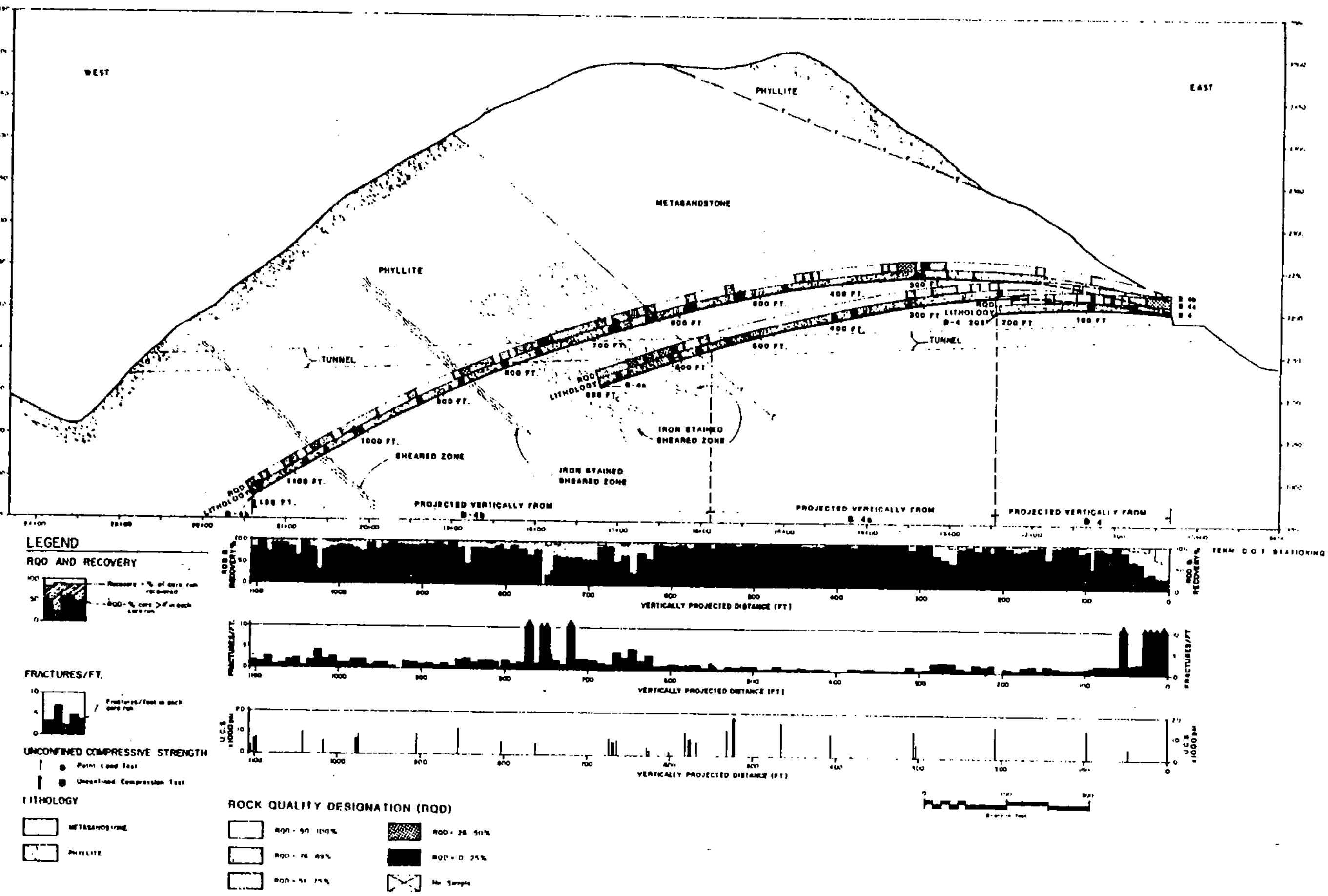
The tunnel was stabilized by removing a total of 10 bays of timber sets, flattening the piles of rock above the remaining timber sets, and installing rock bolts in the remaining rock arch to maintain its intact strengths and prevent further loosening of the rock mass. The stabilized zone of the tunnel is shown in Figure 4.

Additional support at the east portal, where loosening had caused sinkholes at the surface, was provided by constructing reinforced concrete arches in the bays between the timber sets using the existing timber sets as support for concrete forms. After the internal support was strengthened, pea gravel was tremmied in from the surface to fill voids in the rock mass in an effort to make the ground load on the tunnel support as uniform as possible.

CROOKED ARM RIDGE TUNNEL, TENNESSEE

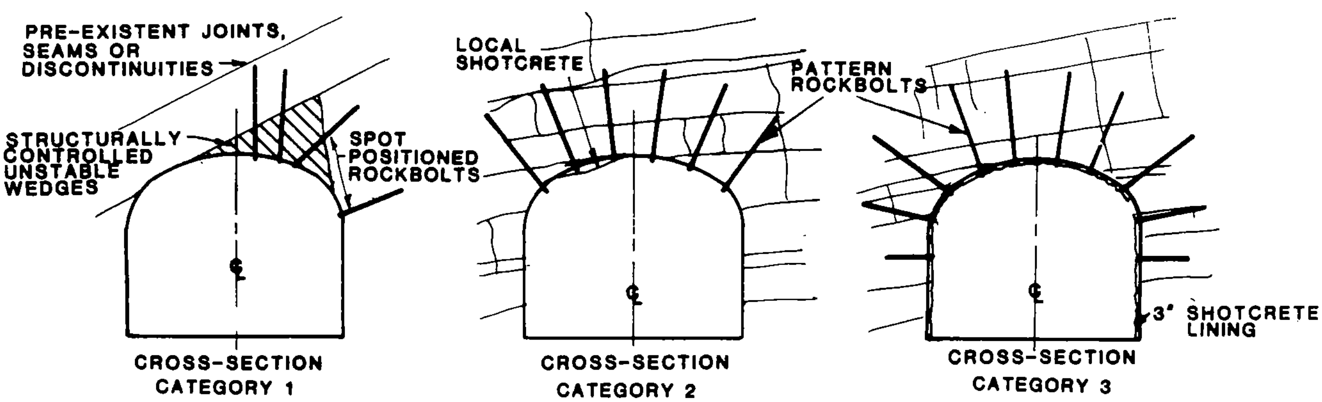
The Foothills Parkway is under construction on the western side of the Smokey Mountains National Park. As part of the section between Wears Cove and Pigeon Forge, Tennessee Department of Transportation is planning to construct a tunnel through Crooked Arm Ridge. The tunnel will be 1200 feet long and will carry two lanes of traffic, one lane in either direction.

Geologic conditions were initially investigated by mapping the rock exposures above the centerline of the tunnel and projecting rock conditions to tunnel level. Subsequently, a series of boreholes were drilled at the east portal and the location of the east portal moved some 100 feet to avoid portaling in weak rock. Because access was only available to the east portal, and environmental constraints limited access to the ridge above the tunnel, a horizontal core hole was drilled to investigate conditions along the centerline of the tunnel. The horizontal drilling program was successfully completed in 1984 with three separate boreholes being drilled to achieve the maximum borehole length of 1,158 feet, as shown in Figure 5 (Reference 4).



TUNNEL ALIGNMENT PROFILE SUMMARIZING GEOTECHNICAL DATA AT CROOKED ARM RIDGE TUNNEL

FIG. 5



TUNNEL SUPPORT FOR EACH ROCK CATEGORY AT CROOKED ARM RIDGE TUNNEL

FIG. 6

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The horizontal drilling program confirmed the anticipated rock types along the alignment. The extent of weathering and of fault gouge along the contact between the phyllite and meta-sandstone were found in the boreholes to be substantially less than expected. Instead, an area of shattered, partially re-healed rock was encountered at the contract. Rock quality along the tunnel alignment was generally better than anticipated from surface mapping.

The trajectory of the boreholes is shown on Figure 5. From the borehole alignment, it was possible to project the geology to the level of the tunnel crown and to estimate the rock support requirements and tunnel excavation conditions. From the information obtained from the boreholes, it was possible to divide the tunnel into a number of discrete geotechnical units of similar rock quality, based on percentage of core recovery, RQD, fractures per foot, and unconfined compressive strength. Using the rock mass classification systems of Bieniawski (Reference 5), and Barton (Reference 6), three rock support categories are expected in the tunnel. These are shown in Figure 6 and consist of:

- Spot position rockbolts in zones of good rock with widely spaced joints (Category I);

- Pattern rockbolts with shotcrete in local areas in zones with more closely spaced joints and minor shears (Category II); and

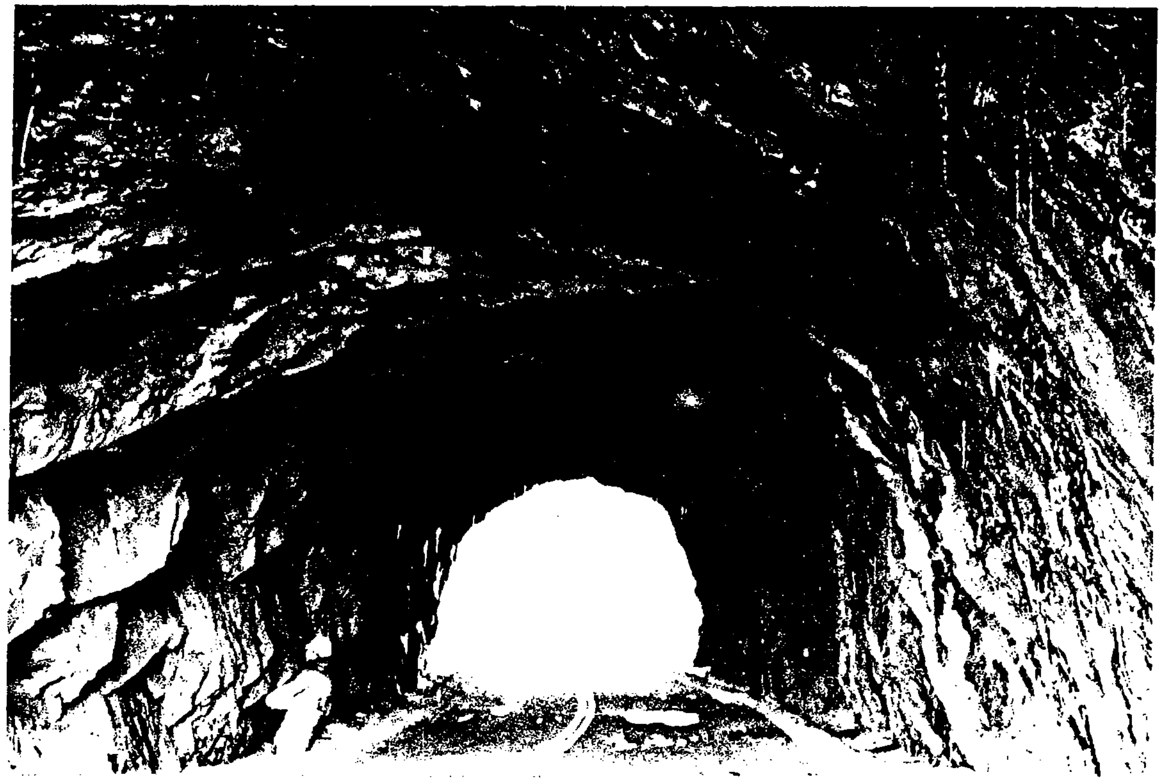
- Pattern rockbolts with a full shotcrete lining in zones of weaker rocks with closely spaced joints or with numerous shear zones (Category III).

The horizontal core holes proved very useful in obtaining a substantial amount of detailed information along the full length of the tunnel at reasonable costs. As a results, neither a pilot tunnel nor further geotechnical investigations are considered to be necessary before the start of construction.

STABILIZATION OF TUNNELS ON THE BLUE RIDGE PARKWAY

The section the Blue Ridge Parkway, north of Asheville, North Carolina, was constructed some 50 years ago. The Parkway is in extremely steep terrain and has a total of 8 tunnels between Asheville and Little Switzerland. A few of these tunnels have masonry structures at the portals, but most of the tunnels rely on the natural strengths of the rock for support.

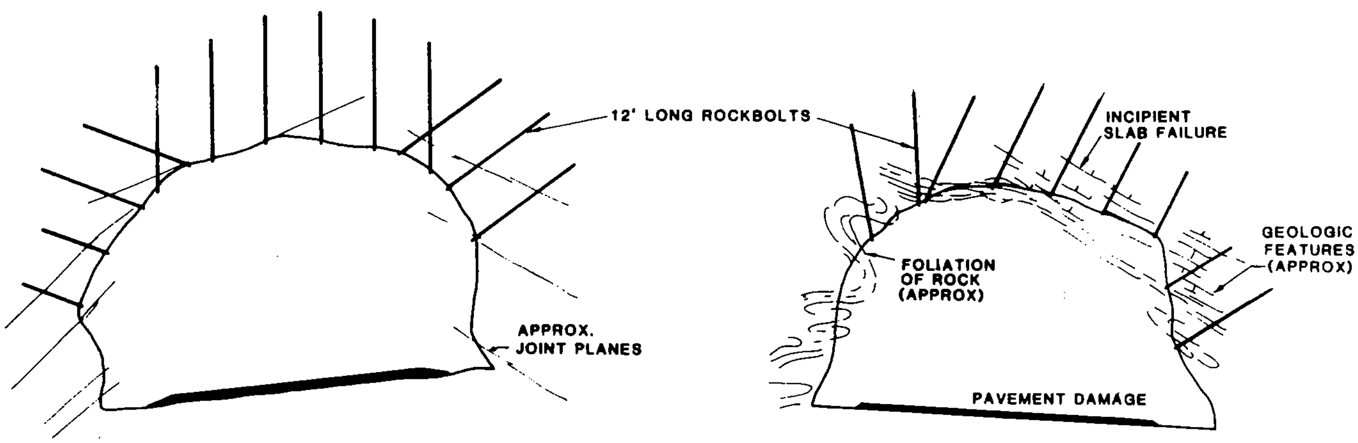
Over the life of the tunnels, there has been significant deterioration of the rock by frost action and loosening of the rock joints. This has resulted in numerous rock falls within the tunnels and at the portals.



ROUGH RIDGE TUNNEL PRIOR TO STABILIZATION

BLUE RIDGE PARKWAY

FIG. 7



ROCK SUPPORT IN ROUGH RIDGE TUNNEL

ROCK SUPPORT IN WILD ACRES TUNNEL

STABILIZATION OF

BLUE RIDGE PARKWAY TUNNELS

FIG. 8

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Following a condition survey of all eight tunnels, Wild Acres Tunnel and Rough Ridge Tunnel were selected as having the highest priority needs for repairs. The condition of Rough Ridge Tunnel prior to the repairs is shown in Figure 7.

The unconfined compressive strength of the metamorphic rock in all of the Blue Ridge Parkway Tunnels is very high. The tunnels are short and the cover over the tunnels is low. Consequently, all stability of the tunnels is governed by the rock structure. Geotechnical mapping of the tunnels indicated that pattern of rockbolts placed at appropriate orientations would provide adequate support for each of the tunnels. The final design for each tunnel used 12 foot long rockbolts, inclined to cross the major discontinuities, as shown in the two cross-sections in Figure 8. The pattern of rockbolts was varied along the length of the tunnels to suit the rock conditions. In places, shotcrete was required to support minor blocks between the rockbolts.

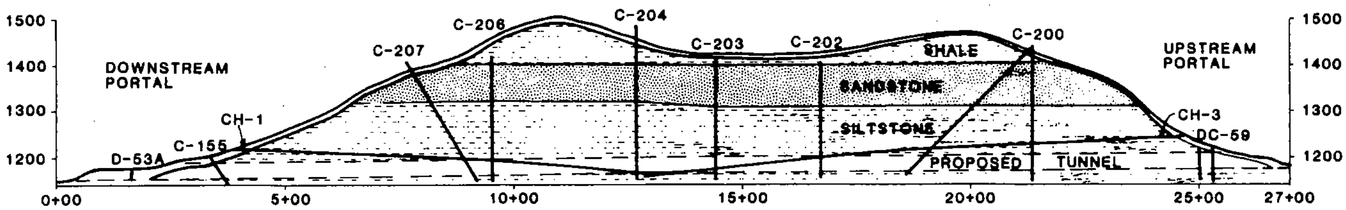
In addition to the rockbolts and shotcrete within the tunnels, masonry structures were constructed at the portals to improve aesthetic conditions and provide additional support. Some rockbolting has also been installed at the portals, but considerable effort has been made to limit the visibility of the rockbolts by cutting off the protrusions and removing the face plates.

The stabilization of Wild Acres Tunnel and Rough Ridge Tunnel was performed in early 1988, while the Parkway was closed during the winter months.

HARLAN DIVERSION TUNNELS, KENTUCKY

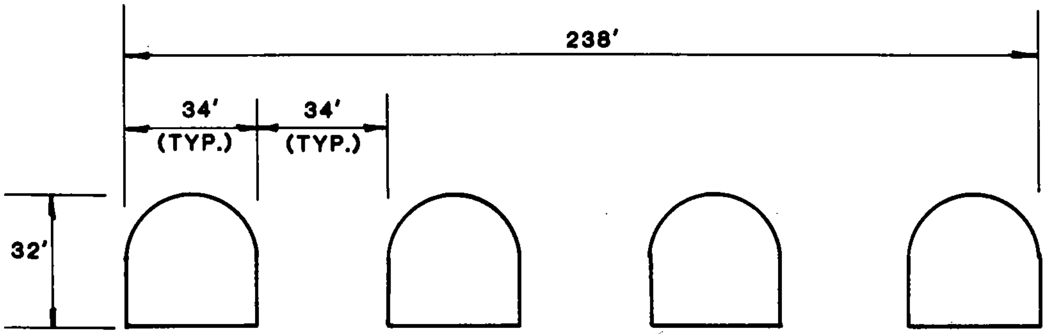
Four parallel tunnels, 35 feet in diameter and each 2,000 feet long, are planned by the Corps of Engineers to divert the upper Cumberland River around the Town of Harlan, in eastern Kentucky, to prevent flooding in the area town. The project will entail relocation of Highway 38 over the upstream portal and of Highway 72 over the downstream portal of the project.

The tunnels will be excavated through the horizontally bedded Hame Formation Siltstone as shown in Figure 9 and 10. Extensive vertical, inclined and horizontal core hole drilling has indicated that there are virtually no joints within the full length of the tunnels. The strength of the siltstone, which is highly anisotropic because of the horizontal bedding, is expected to determine the stability conditions during tunneling. Because of the absence of jointing, structural control is not expected to be a factor, except in minor areas, but extensive stress analyses, by both the Boundary Element Method (Reference 2) and the Flac program, have indicated that zones of overstressed rock are possible above the crown and at the invert of the tunnels, as shown in Figure 11. The zones of overstressing shown in this figure are zones where the applied stresses from the excavation of the tunnels



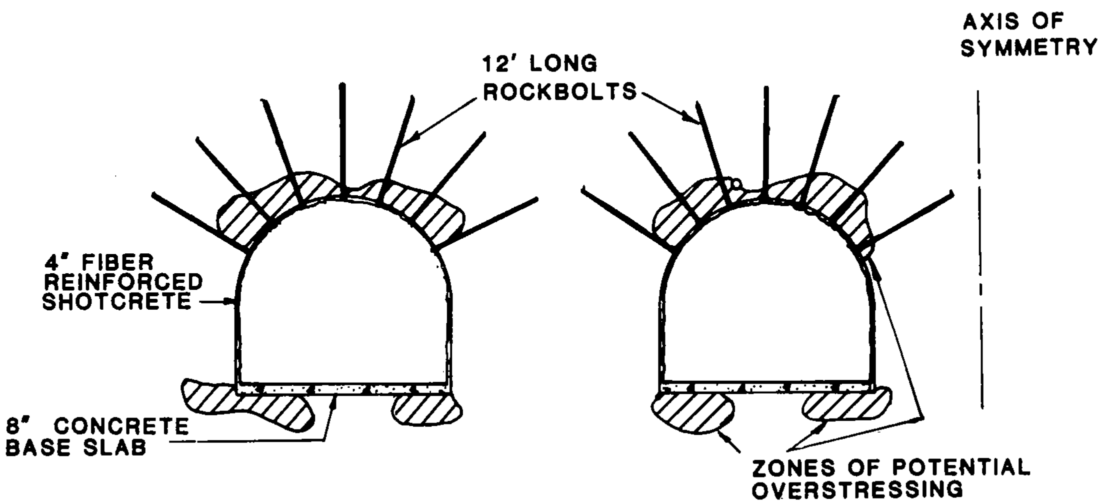
GEOLOGIC PROFILE THROUGH HARLAN DIVERSION TUNNELS

FIG. 9



CROSS-SECTION THROUGH HARLAN DIVERSION TUNNELS

FIG. 10



TUNNEL SUPPORT & ZONES OF POTENTIAL OVERSTRESSING
HARLAN DIVERSION TUNNELS

FIG. 11

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may exceed the strength of the rock mass or the shear strength of the bedding planes surrounding the tunnels. In order to ensure the stability of the tunnels, a pattern of 12 foot long rock bolts anchored beyond the zone of overstressing are planned to "knit" the rock mass together and hold the rock in place if overstressing, or slippage along the bedding planes, does occur.

The siltstone unit through which the tunnels will be excavated, has a tendency to slake so a four-inch fiber-reinforced shotcrete layer is planned to prevent moisture changes within the rock mass. In addition, an eight-inch concrete base slab is planned.

Extensive hydraulic model testing has been performed to examine the hydraulic characteristics of the tunnels. The tunnels are in the final design phase and construction is due to start in September 1989.

CONCLUDING REMARKS

At the five tunnel projects in the Appalachian Mountains described in this paper, the following types of ground behavior have been encountered or are expected:

- Cumberland Gap - structural control and slaking;
- Little Tunnel - structural control;
- Crooked Arm Ridge Tunnel - structural control;
- Blue Ridge Parkway - structural control; and
- Harlan Diversion Tunnels - overstressing and slaking.

In all five tunnels, the use of rockbolts and shotcrete has resulted, or will result, in substantial savings in rock support costs over the conventional use of steel arches or timber sets and lagging.

REFERENCES

1. Cording, E.J. 1984 "State-of-the-Art: Rock Tunneling," Tunneling in Soil and Rock, Procedings of Session at GEOTECH '84 ASCE, Atlanta, Georgia.

2. Hoek, E. and Brown, E.T. 1980 "Underground Excavations in Rock" Institution of Mining and Metallurgy, London, U.K.

3. Sullivan, W.R. and Leary, R.M. 1987 "Benefits of the Pilot Tunnel on the Cumberland Gap Highway Tunnel Project" RETC, New Orleans, Louisiana.

Richard Humphries
Randy Sullivan

4. Humphries, R.W. and Ingram, P.M. 1985 "Investigations and Preliminary Design of Crooked Arm Ridge Tunnel" Southeastern Transportation Geotechnical Engineering Conference, Gatlinburg, Tennessee.

5. Bieniawski, Z.T. 1974 "Geomechanics Classification of Rock Masses and its Application in Tunneling" Proc. 3rd International Conference on Rock Mechanics, ISRM, Denver, Colorado.

6. Barton, N., Lien, R. and Lunde, J. 1974 "Engineering Classification of Rock Masses for Design of Tunnel Support" Rock Mechanics, Vol. 6, No. 4.

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