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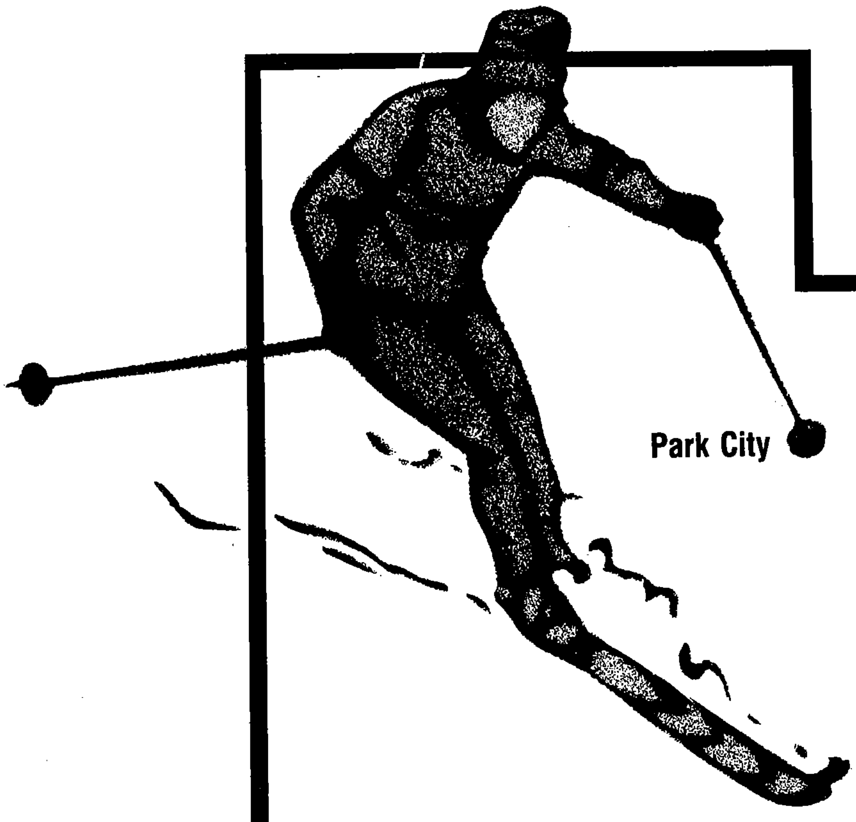

Construction to Minimize Environmental Impact

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BRIGHAM YOUNG UNIVERSITY

Park City, Utah
August 17-19, 1988

PROCEEDINGS OF THE 39TH ANNUAL

HIGHWAY GEOLOGY SYMPOSIUM

CONSTRUCTION TO MINIMIZE ENVIRONMENTAL IMPACT

SYMPOSIUM VENUE

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UTAH DEPARTMENT OF TRANSPORTATION

UTAH GEOLOGICAL AND MINERAL SURVEY

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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, 37 consecutive annual meetings have been held in 24 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 held in San Jose, California and the 36th HGS was in Clarksville, Indiana. This year's meeting, the 37th, was held in Helena, Montana, the capital of the Big Sky Country.

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee composed of approximately 20 engineering geologists and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. 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 protem 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 principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee trip provided Stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and Glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area which illustrated the 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 was through the Rockies of recent interstate construction in the Boulder Batholith. The trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

At the technical sessions, case histories and state-of-the-art papers are most common with highly theoretical papers the exception. The papers presented at the technical sessions are published in the annual proceedings. Some of 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 Carolina Department of Transportation in Raleigh 27611. Costs generally range from $5.00 to $15.00, plus postage.

Highway Geology Symposium

MEDALLION AWARD WINNERS

Hugh Chase	1970
Tom Parrott	1970
Paul Price	1970
K. B. Woods	1971
R. J. Edmonson	1972
C. S. Mullin	1974
A. C. Dodson	1975
Burrell Whitlow	1978
Bill Sherman	1980
Virgil Burgat	1981
Henry Mathis	1982
David Royster	1982
Terry West	1983
Dave Bingham	1984
Vernon Bump	1986

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.

Highway Geology Symposium

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Highway Geology Symposium

39TH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

AUGUST 17-19, 1988, PARK CITY, UTAH

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A BRIEF OVERVIEW OF THE GEOLOGY OF UTAH

Hellmut H. Doelling
Senior Geologist, Mapping Section
Utah Geological and Mineral Survey

INTRODUCTION:

Utah is a showcase geologic state, one where geologic phenomena are well displayed. Most transportation routes, especially in Utah, were developed because of the pristine geologic or geographic conditions that existed when the land was explored, settled, and developed. Utah is a state with many natural barriers to transportation, including mountain ranges, deeply incised canyons, monoclinal ridges, lengthy cliffs, and interior drainage mud flats. Utah can naturally be divided into three physiographic provinces: Basin and Range, Central Rocky Mountains, and Colorado Plateaus (figure 1). Each of these physiographic provinces displays a separate style of geology--in its stratigraphy, sedimentation patterns, and structure types. The province areas have been present for the last billion years and the boundaries that divide them have been present the same length of time. The boundary that divides the Basin and Range from the other two provinces has been very important throughout geologic time and has been called the Wasatch line, Wasatch hinge line, or Basin and Range-Colorado Plateau transition zone, among others. I don't mean to imply that the Basin and Range has always been a Basin and Range physiographic province, or that the Central Rocky Mountains have always been high mountain ranges, or that the Colorado Plateaus were always plateaus. I am implying that geologic environments and conditions across the lines were generally different throughout geologic history.

GEOLOGIC HISTORY:

Figure 2 is a generalized geologic map of Utah from Tooker and Stewart (1969). Starting in latest Precambrian time and continuing throughout the Paleozoic Era (900 to 245 million years ago), Utah was usually covered by an ocean; the area of the Basin and Range differed from that of the Central Rockies and Colorado Plateaus by subsiding and receiving sediments at a much greater and more continuous rate. In western Utah greater thicknesses of rocks were deposited than to the east, even though the entire state received mostly marine sedimentation,. In technical terms the rocks of the west were deposited in a miogeocline while the rocks to the east were deposited on a continental shelf. At times the sea or ocean retreated somewhat and placed the shoreline very near the present Wasatch hinge line. Then sediments were received only in western Utah and erosion may have occurred in eastern Utah. The Ordovician and Silurian periods lasted from 505 to 408 million years ago, and are not represented by rocks in eastern Utah. Starting 350 million years ago (Mississippian time) basins began to be formed on both sides of the Wasatch hinge line, but the basins that developed to the west received more and thicker layers of sediment than those in the east. Concurrently high areas developed, usually as submarine shelves between basins. In Pennsylvanian time mountain ranges, often called the ancestral Rocky Mountains, projected above sea level. These mountains were eroded and the sediment was carried to the basins. In the east these basins were often isolated from the open ocean and salt and gypsum were deposited in relatively great thicknesses. The unstable salt masses actually migrated in the Paradox Basin of eastern Utah to form thick salt anticlines. As the era of dinosaurs or the Mesozoic Era commenced, about 250 million years ago, the ocean began to retreat to the west and eastern Utah became a terrestrial landscape. Both the eastern and western parts of the state were still accumulating sediments and forming new rock formations, but those in the eastern part of the state were deposited above sea level as fluvial deposits. Eventually the ocean even disappeared from western Utah. In the middle of the Mesozoic Era much of Utah became a desert. However, two shallow seas successively extended southward from the north; at least one became restricted and salt and gypsum were deposited in central Utah. In the last period of the Mesozoic Era, the western part of Utah rose as a high mountain range as part of a classic mountain building event or orogeny. Rock layers deposited in the Paleozoic ocean 25, 50, and even 100 miles to the west were thrust eastward against the Wasatch Hinge line. The area of the Central Rockies and Colorado Plateaus remained low and received the erosional detritus from this mountain range. About 90 million years ago, while the mountains remained high in the west, another sea invaded Utah, the Mancos Sea, this time from the east. As the Sevier Orogeny mountain range continued to rise to the west it shed clastics; dumping coarse materials at the foot of the range, ever fining eastward, finally dumping gray muds into the sea. The deposits of this gray muddy sea are now known as the Mancos Shale. In the lagoons and deltas of the coast a dense vegetative growth flourished and Utah's coal deposits were formed and deposited. The Mancos Sea retreated after about 10 million years, but the eastern side of Utah continued to subside, especially the Uinta Basin, and continued to receive thick sediments. This receiving of sediments in eastern Utah continued to about 35 million years ago. Some were deposited by streams on flood plains and in channels, others in large lakes. At times large quantities of organic materials were deposited that were later transformed into oil shale, gilsonite veins, and similar deposits. By this time the mountain building to the west had slowed and erosion lowered the ranges considerably. The orogenic mountain building activity was replaced by volcanic activity. Huge stratovolcanoes rose in several places, spewing forth tons of volcanic ash, lava flows, and other volcanic debris. Some experienced catastrophic explosions which pulverized their tops and created huge calderas and craters. Some of the molten rock material did not reach the surface, but invaded the deep older rocks. As these deep plutonic and invaded rocks cooled, hot solutions mineralized them in many places. Precious and base metal deposits, iron deposits, and other valuable minerals were emplaced in principally in central and western Utah. In eastern Utah the volcanic activity was much more limited, but the famous laccolithic peaks of the La Sal, Henry, and Abajo Mountains were created. In addition local areas were uplifted or folded as domes, anticlinal mountains, anticlines and synclines, and monoclines. Such features as the San Rafael Swell, Uinta Mountains, and Circle Cliffs Uplift came into being during this 40 to 20 million year ago interval. After 20 million, and most importantly after 10 million years, the modern face of Utah began to emerge. To the west the crust was stretched along a general east-west axis from deep within the Earth. Long zones of north-south trending faults developed forming rows of grabens and horsts. The horsts were attacked by erosion and formed the north-south trending mountain ranges of the Basin and Range, whereas the grabens were filled with the erosional detritus and become the basins of the province. Some of these basins are known to contain detrital thicknesses of as much as 14,000 feet. Volcanism continued into this time, but with diminished emphasis. In the relatively recent past many of the intermontane basins were inundated by prehistoric Lake Bonneville, the deposits of which presently veneer the basin surfaces. The eastern boundary of the Basin and Range Province is marked by the Wasatch fault zone which is aligned along the Wasatch hinge line. The Wasatch fault scarps are quite steep and sharp and suggest that movement continues to the present day. The displacement along this fault is measured in many thousands of feet. The rocks to the east of the fault zone rose to create the Wasatch Range, which coupled with the Uinta Mountains, form the Central Rockies Physiographic Province. Some of north- trending faults, typical of the Basin and Range developed east of the hinge line, especially in the south half of the state. The blocks of rocks between these faults are elevated as horsts to form the High Plateaus of Utah. While basins and ranges were forming in the west the Colorado Plateaus area rose and began to be eroded. The Colorado River drainage system developed and incised deep canyons into its surface. The courses of the river and its tributaries were influenced by the hard and soft layers of previously deposited rock and plateaus, buttes and mesas developed as the erosional style. Monoclines, anticlines, and synclines formed during the previous interval created irregularities in the otherwise "layer cake" geology of the plateau country.

BASIN AND RANGE ROCK TYPES:

The rocks of the ranges of the Basin and Range physiographic province are dominated by those deposited in the ancient Paleozoic ocean. These are mostly hard gray limestone or dolomite with lesser amounts of sandstone, quartzite, and marine shale. These rocks are mostly well bedded and well indurated. They have undergone the intense deformation of the Sevier orogeny during the latter part of the Mesozoic Era, and are faulted, tilted, and folded, often to a remarkable degree. Volcanic igneous rocks, of the 40 to 20 million year interval, cover or intrude them in some mountain ranges. The sediments exposed on the surface of the basins are mostly Lake Bonneville deposits. They consist of unconsolidated silts, clays, sands, and gravels derived from the mountain ranges. In places older partially consolidated materials appear under the younger, but are similar in composition. They were deposited in ancestral lakes or by ancestral creeks and rivers. Locally, interbedded in these older materials, are volcanic ashes and tuffs, even some lava flows. The Bonneville sediments of the basins lie nearly flat, in their original angle of deposition, some of the older underlying deposits have been tilted and folded. Nevertheless some of the flat-lying very young Bonneville sediments have been displaced by recent fault activity. Most ground transportation routes favor the basins because of their flat nature. Former lake basins make the flattest ground of any. Additionally, Lake Bonneville eroded the hard well-consolidated bedrock of the ranges to provide road builders with an abundant supply of good gravel.

COLORADO PLATEAU ROCK TYPES:

The rocks presently exposed in the Colorado Plateaus are those deposited during the Mesozoic Era or age of dinosaurs. Those deposited on the shelf by the Paleozoic ocean are mostly covered, bared only in the deepest canyons. The Mesozoic rocks were deposited in mudflats, floodplains, stream channels, deltas, lagoons, shallow muddy seas, and as sand dunes on deserts. They are much more diverse than those in the mountain ranges of the basin and range west. Hence the rocks are alternatingly hard and soft, ideal for eroding into cliffs, mesas and buttes. Rock types include sandstone--the Colorado Plateaus probably have the largest accumulations of sandstone formations in the world--mudstones, claystones, shales, conglomerates, siltstones, and fresh-water limestones. The Colorado River and its tributaries are vigorously eroding the territory, doing their work by incision rather than by sweeping to and fro. Erosional and weathering debris do not remain long on the plateau surfaces and are cleaned off regularly by sheetwash and flash floods. Many surfaces are of bare rock. Ground transportation routes in the Colorado Plateau are along the strike of soft rock layers whose upper surfaces weather into thin soils. The hard units erode into cliffs and other barriers difficult to cross. Even so, the hardest of the plateau formations have not been through the intense mountain building activity of the western ranges. Hence they are generally flat-lying or slightly tilted. Locally, at monoclines they stand vertical, but these are exceptional places. These rocks are not as well indurated or cemented and hand specimens of the cliff-formers can often be abraded or crushed with the hand. The most popular formation in the Colorado Plateaus on which to build a road is the Mancos Shale, deposited by the muddy Mancos Sea. It erodes into a broad and extensive valley or lowland with few impediments. Working it is relatively easy when it is not wet. Even so, its soft nature and clay content make it difficult to maintain roads built upon it. The Mancos Shale and the many sandstone units of the Colorado Plateau are generally too soft to create good gravel deposits for construction purposes. Pediments formed at the base of cliffs often provide good gravels, but there are many places on the plateaus where they are not present or where they are composed of weak sandstone. Steep cliffs, coupled with the diversity of rock types, provide opportunity for the development of a host of geologic hazards, including toreva block faulting, landslides, rockfall, and slumping. These are especially prevalent in the High Plateaus of Utah where the higher altitudes invite heavy winter snowcover. During snowmelt the surface rocks become saturated and the hazards become active. The various rock types of the plateaus alternate in color and make this part of Utah a great tourist attraction. Deep canyons outlined with color bands, interesting erosional forms, rock arches and bridges are the basis for all of Utah's national parks, including Bryce Canyon, Zion, Capitol Reef, Canyonlands, and Arches. The latter park also displays some unusual salt tectonic features.

CENTRAL ROCKY MOUNTAIN ROCK TYPES:

The consolidated rocks of the Central Rocky Mountains are of all ages, ranging from those formed in Archean times (older than 2.6 billion years) to those formed less than a few thousand years ago. These are the same rock types as those found in the western basins and ranges and on the Colorado plateaus. Those of the Wasatch Range are intensely folded and thrust-faulted reflecting the Sevier orogenic activity, and cut by north- trending faults reflecting the Basin and Range activity. The north- trending faults have positioned the widest valleys which are filled with fluvial debris--sand, gravel, silt from the surrounding mountains--in the form of alluvial fans. These valleys and the larger canyons provide transportation corridors through the mountains. The Uinta Mountains portion of the Central Rockies is an east west-anticlinal range. Very old quartzitic rocks make up most of the core; the inner flanks contain the rocks deposited in the Paleozoic ocean, the outer flanks expose the Mesozoic formations with the diverse lithologies. The geologic problems of the diverse rocks of the plateaus are also present in the mountains in which they are exposed. Steeply dipping bedded rocks occasionally slide into the canyons blocking the transportation routes. The high mountains are covered with heavy snows over the winter; the moisture weathers the rocks to form thin mountain soils or colluvium that cling to the mountain surfaces at the maximum angle of repose. During the annual snowmelt these become saturated and present debris flow, mudflow, and other hazards. The very highest of these mountains are covered with glacial till, its peaks and arete ridges surrounded by cirques and U-shaped valleys reminiscent of Switzerland. The semi-arid climate of Utah limits the ability of nature to cover its rocks. They are therefore displayed as well as can be expected on this Earth of ours. Utah has representatives of most kinds of rocks, ages of rocks, environments of rock deposition, landforms, and structures. Its a good place to study geology, to go sightseeing, and a challenging place to build roads.

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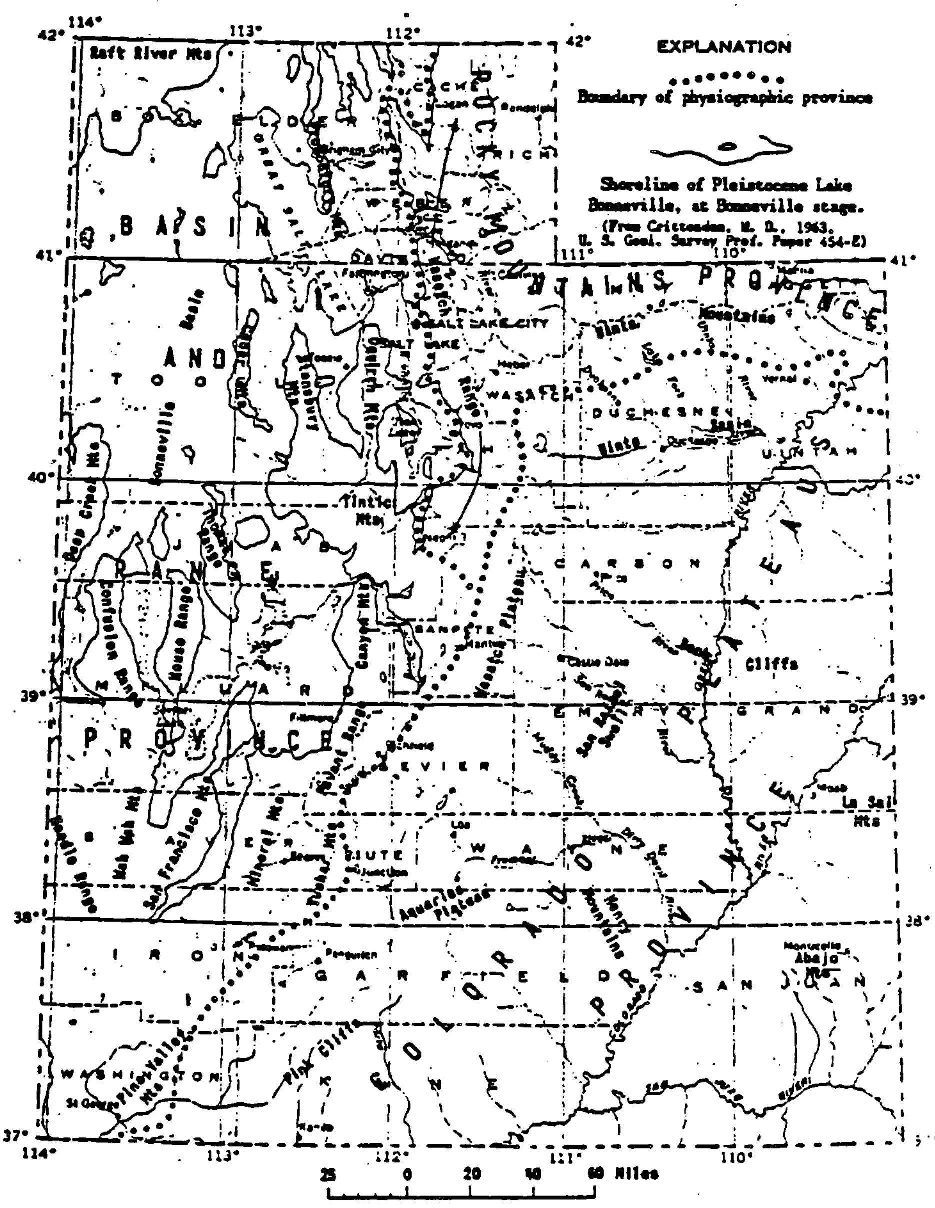

Figure 1: Physiographic provinces and principal features of Utah (Tooker and Stewart, 1969).

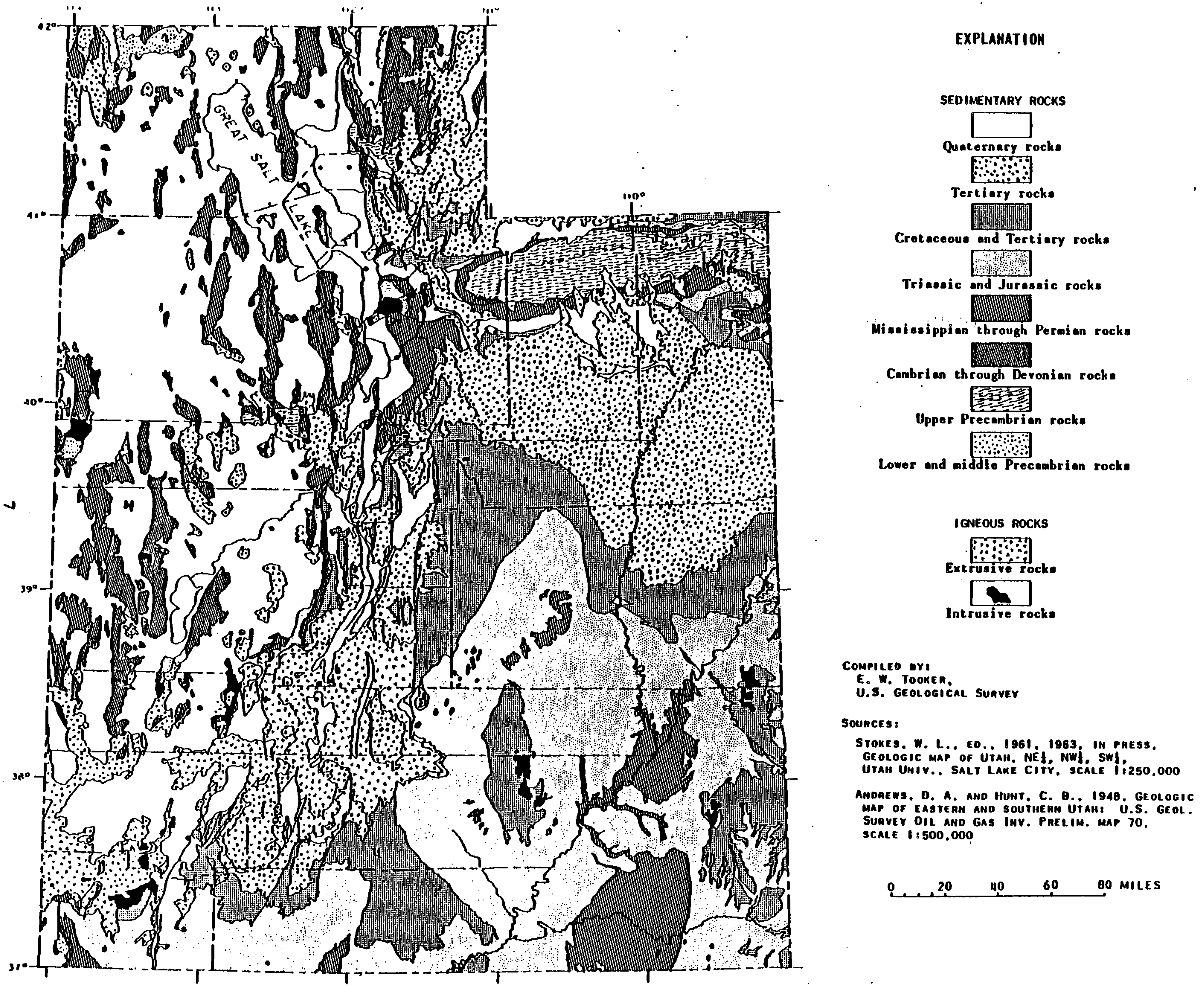

Figure 2: Geologic map of Utah (from Tooker and Stewart, 1969).

GEOLOGIC HAZARDS OF UTAH

by
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INTRODUCTION

The actions of wind, water, and other earth processes such as landslides and earthquakes, occurring throughout geologic time, have shaped Utah's diverse landscape. These natural processes potentially threaten life and property and may be termed geologic hazards. Recent geologic events in Utah, particularly in 1983 and 1984, have provided the state with a stern reminder of the geologic hazards with which it must co-exist, and have spurred increased efforts to understand and mitigate Utah's geologic hazards. Damage from the recent rise of Great Salt Lake, stream flooding and debris flows along the Wasatch Front and Plateau, statewide ground-water rises, and the Thistle landslide have had a major impact on the economy of the state and lives of its citizens, with the cost of repairs public estimated at over 500 million dollars. This is small, however, compared to the cost of damage that could result from a large magnitude earthquake occurring somewhere along the Wasatch Front.

The frequency of geologic hazards in Utah (especially debris flows and stream flooding) has returned to "average" since the early 1980s, due to a decrease in precipitation. Statewide, precipitation has been near or below average during the last two years, and the "wet cycle" which began in 1982 ended in 1986 (Mabey, 1987). However, the October 1987 mud floods following a wildfire northeast of Orem, and the June 1988 rock slides along Highway 95 near Blanding, attest to the ongoing nature and widespread geographic distribution of the State's hazards. These yearly occurrences are overshadowed by the likelihood that a large earthquake could occur in Utah at any time. In short, the geologic hazards affecting Utah, chiefly slope failures, flooding, earthquakes, shallow ground water, and adverse soil conditions, remain hazards today, and should concern every Utahn, and especially those associated with the planning, design, construction, and maintenance of Utah's highways.

SLOPE FAILURE

Slope failures are most common in areas of high precipitation, high elevation, steep slopes, and slide-prone geologic materials. These conditions exist in the Wasatch and Uinta Mountains in northern Utah and in the high plateaus and steep canyons of central and southern Utah. Both shallow-seated landslides such as debris slides and rock falls, and deep-seated landslides such as rotational slumps, occur in Utah.

Debris slides usually fail along the contact between the surficial weathered zone (soil, regolith, colluvium) and underlying bedrock, in response to positive pore-water pressures created by infiltration of water from precipitation, snowmelt, underlying bedrock springs, or other sources. Debris slides may become debris flows or debris floods with the introduction of additional water, usually when an advancing slide mass encounters a flowing stream. The resulting mixture is often fast-moving, and can present a hazard to people and structures downstream, where deposition of debris and flooding occurs. Rock falls consist of rock fragments that detach from parent bedrock along joints, bedding planes, or other zones of weakness. They occur in many areas of Utah that have steep slopes and bedrock outcrops, and are particularly common in southern Utah, where resistant but jointed sandstones like the Castlegate, Navajo, and Wingate form near-vertical cliffs (Fig. 1).

Deep-seated landslides generally fail along a contact between two different bedrock units, or within a particular slide-prone formation. Slumps form a headscarp and may progress downslope into a more viscous earth flow, or form a bulge at the toe. Damage can occur when structures are on or close to a

Figure 1. July 1, 1988 rock fall in Bloody Mary Wash north of Arches National Park. Boulders of Wingate sandstone damaged tracks of the Rio Grande Railroad. Photo courtesy of Genevieve Atwood.

slide. Secondary damage such as flooding can occur when landslides dam rivers or fail into reserviors. Deep-seated landslides are often slow-moving, and usually present a greater hazard to property rather than lives.

In Utah, most geologic units susceptible to landsliding (rockfalls excluded) consist of or contain abundant clay or shale. In northern Utah, including the Wasatch and Uinta Mountains, shale-bearing geologic units involved in landsliding include the Ankareh Formation, Salt Lake Formation, Wasatch Formation, Manning Canyon Shale, and Red Pine Shale. In central Utah, units include the North Horn Formation, Indianola Formation, and Mancos Shale. In the Wasatch Plateau region of central Utah, the most hazardous areas for landsliding are those where high, steep, north-facing slopes are underlain by the North Horn Formation (Godfrey, 1978). In southern Utah, landslides occur in the Tropic Shale, Chinle Formation, Carmel Formation, and Mancos Shale.

Although increasing the water content of geologic units on steep slopes is a common cause of slope failures, they can also occur without a significant increase in water content. Furthermore, landslides are not restricted to high mountains where precipitation is greatest. Slump-type slope failures commonly result from removal of support material at the base of a previously stable slope or at the toe of an existing landslide. This can occur naturally by streambank erosion, or as the result of construction activities. Landslides can also be initiated by earthquake ground shaking. Overwatering of slopes through lawn and field irrigation or leakage of underground pipes has contributed to many recent landslides in non-mountainous areas of Utah such as in Monticello (Case, 1988), St. George (Christenson, 1987), and Hoytsville (Klauk and Harty, 1988).

FLOODING

Two types of surface flooding occur in Utah: 1) overbank flooding along stream channels during intense cloudburst storms (flash flooding), long-duration rainstorms, or spring snowmelt, and 2) flooding of shorelines or low-lying areas from lake-level rises or ponding of water. Stream-channel flooding poses the greatest hazard to lives and property because it can occur suddenly, with little warning (Fig. 2). Flooding from lake level rises occurs more slowly, and usually poses little hazard to lives.

Generally, the most destructive flooding in Utah comes from short-duration, severe rainstorms of convective origin. These flash floods occur most often in summer months, between July 1 and September 10 (Marsell, 1967). They occur throughout the State, but are particularly common in southern Utah, where runoff from cloudburst storms is funneled through sparsely-vegetated canyons with great efficiency. From the time of pioneer settlement in 1847, to 1981, 1425 cloudburst floods have been

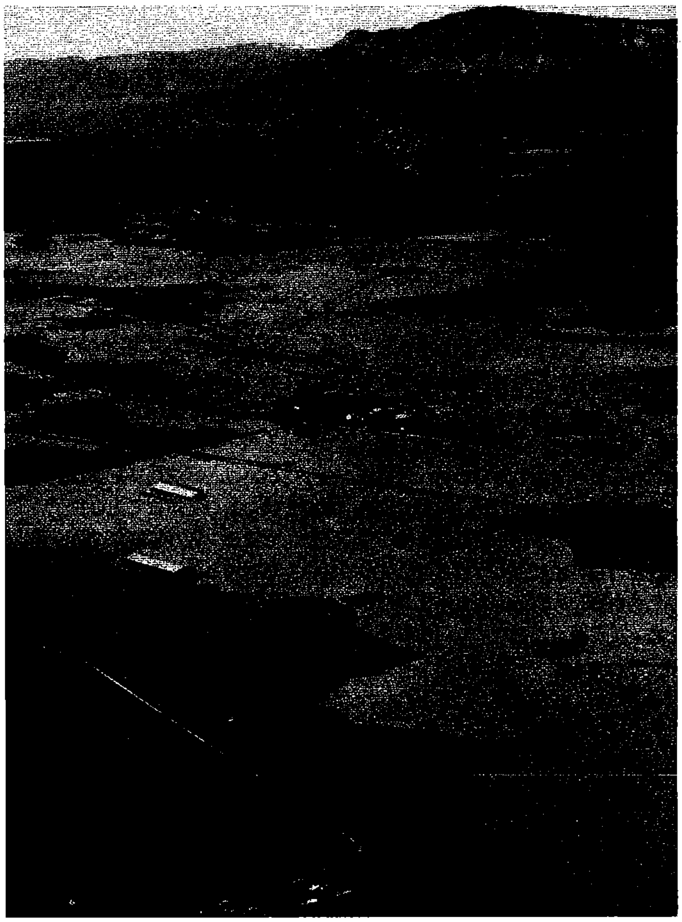

Figure 2. Flooding of the San Pitch River east of Wales in Sanpete County, 1983.

Figure 3. Flooding of the Jordan River control gate at the northern end of Utah Lake, 1983.

reported in Utah (Utah Division of Comprehensive Emergency Management, 1981). Of these, approximately 20 percent (294) were in Salt Lake and Utah Counties, which represent only a small percentage of the State's total area. These counties contain more than one half of the total State population, and thus observe and report more flood events than sparsely populated counties. Precipitation-frequency data for 22 stations throughout Utah derived by Richardson (1971, reported by Butler and Marsell, 1972), show that southern Utah is expected to receive more intense rainfall per unit of time than northern Utah. Therefore, southern Utah has the greater potential for flash flooding, but northern Utah has incurred more damage because of its greater population density (Butler and Marsell, 1972).

During the 1982-1986 wet period, Great Salt Lake and Utah Lake rose to record-high levels, causing hundreds of millions of dollars in damage to structures, property, and to Utah's economy. The recent decline in annual precipitation over Great Salt Lake's watershed, plus the pumping of water from the lake to the Great Salt Lake Desert beginning in 1986 has greatly reduced the hazard of flooding around Great Salt Lake. Dredging of the Jordan River and modification of the outlet structure on Utah Lake has likewise decreased the flood hazard of that lake (Fig. 3).

Low-lying areas such as the basins in western Utah can also present a flood hazard to structures, including transportation and utility lines. Flooding and ponding after rainstorms or seasonal snowmelt occurs in the Great Salt Lake Desert, and in the many dry and intermittent lakes located in the lowest areas of closed basins (Harty and Christenson, 1988).

SHALLOW GROUND WATER

Rises in the ground-water table usually occur in response to increases in precipitation or irrigation. Statewide, ground-water levels generally rose during the 1980s, and it is estimated that about 15 percent of Utah has ground water within 30 feet of the surface (Hecker and others, 1988). This caused numerous surface and subsurface flooding problems throughout the State. Basement and septic tank flooding occurred in many cities in Salt Lake County, as well as in Ballard (Uintah County); Moab (Grand County); Plymouth (Box Elder County); Erda, St. John, and Clover (Tooele County); and Fountain Green, Mt. Pleasant, and Ephraim (Sanpete County) (Hecker and others, 1988; Klauk, 1988). Wetlands and swamp areas north of Sevier Lake expanded and contributed to flooding of Highway 6/50 (Harty and Christenson, 1988). Rising ground water has caused a number of contamination problems in the State, including flooding of landfills (Davis and Salt Lake Counties) and transmission of gasoline from leaky underground storage tanks into sewer lines and basements (Sugar House, Moab, Kaysville, and many Wasatch Front communities) (Hecker and others, 1988). Shallow ground water has not been a problem only during the recent wet years; irrigation practices and overwatering have contributed to a long-term problem in many areas in the State.

In addition to flooding, shallow ground water can also negatively impact soils in construction areas. Soils with high clay or silt contents can become unstable in cut slopes when construction excavations penetrate the water table. Dewatering the site and installing subsurface drains is often necessary before construction can proceed.

EARTHQUAKES

Earthquake activity in Utah is pronounced in the Intermountain Seismic Belt (ISB), a zone trending generally north-south through the central part of the State. The ISB represents an area where the earth's surface is stretching and cracking in response to active deformations in the North American subplates (Smith and Sbar, 1974; Arabasz and Smith, 1979). Utah has been fortunate in historical time to have experienced few destructive earthquakes. Geologic evidence, however, indicates that large earthquakes have occurred in the past and will likely occur again in populated areas of Utah, particularly along the Wasatch Front. In the last 6000 years, recurrence of large earthquakes along the Wasatch fault in north-central Utah has averaged about once every 350 to 400 years (Schwartz, 1988). Of the geologic hazards that can affect Utah, none has the potential for creating more far-reaching, widespread damage than a large, 6.0 to 7.5 magnitude earthquake. The hazards associated with a large earthquake include ground shaking, surface fault rupture, and secondary effects such as soil liquefaction, slope failure, seiche generation, tectonic subsidence, and flooding.

Earthquake ground shaking is the most widespread and potentially damaging of the seismic hazards. The intensity of ground shaking in a given area depends on the source characteristics of the earthquake (magnitude, depth to focus), distance from the epicenter, geologic conditions in the area, such as subsurface composition and geometry (soil verses bedrock, geologic structure), and thickness and water content of unconsolidated deposits. Research indicates that within the Wasatch Front valleys, which are underlain by unconsolidated sediments, certain earthquake ground motions may be amplified up to 10 times that of adjacent bedrock areas (Hays and King, 1982). This amplification of ground motion results in increases in expected seismic intensity and structural damage (Algermissen and Steinbrugge, 1984).

Ground shaking is not limited to causing direct damage to structures; it can also initiate soil liquefaction, which occurs when seismic shaking causes certain types of soils (sands and silty sands) to lose strength and liquefy due to a sudden increase in pore-water pressure. Soil liquefaction may cause various types of ground failures, some of which can occur on slopes as low as 0.5 percent (Youd, 1978). In addition to liquefaction, earthquake ground shaking may also initiate more conventional types of slope failures, such as rock falls, slumps, earth flows, and rock and soil avalanches. Keefer (1984) determined minimum Richter magnitudes required to cause various types of slope failures during an earthquake. They range from a minimum of 4.0 for rockfalls to 6.5 for soil avalanches. Rock falls and rock slides were reported after the 1901 Richfield, 1921 Elsinore, 1934 Hansel Valley, and 1962 Cache Valley earthquakes (Neumann, 1936; Williams and Tapper, 1953; Coffman and Von Hake, 1981; Keaton, 1986), which had magnitudes between 5.7 and 6.0.

Earthquake ground shaking can also generate seiches (waves) in standing bodies of water such as lakes and reservoirs. Of particular concern is flooding due to dam failure caused by overtopping of dams by reservoir seiches. Seiches were reported along the southern shoreline of Great Salt Lake at Saltair and at the Lucin cutoff during the magnitude 6 Hansel Valley earthquake of 1909 (Williams and Tapper, 1953). At that time, Great Salt Lake was at an elevation of 4202 feet (Woolley, 1924, plate II), or about six to seven feet lower than at present (approximately 4208 feet).

All historical earthquakes in the Great Basin with magnitudes of 6.3 or larger have been accompanied by surface faulting (Bucknam and others, 1980). This magnitude is generally taken as the threshold for earthquakes capable of generating surface rupture along normal faults in Utah. Future surface rupture is expected to occur along existing faults that have been active in Quaternary time (Fig. 4). The hazard from fault rupture and related deformation is mainly to structures built on or near the fault, and to lifelines that cross the faults (road, utility, sewer, water). Flooding can also occur during faulting, by diversion of surface waters (streams, lakes, aqueducts, and canals) and pipeline rupture.

Surface deformation caused by faulting commonly occurs in close proximity to faults, but there is evidence that it may also occur on a larger scale, in the form of regional tilting of basin floors toward faults along basin margins (tectonic subsidence). Geologic evidence suggests tectonic subsidence may have occurred along the Wasatch fault (Keaton, 1987). Such differential lowering of the ground surface could produce widespread flooding by lakes (particularly Great Salt Lake and Utah Lake), ponding of shallow ground water, and disruption of facilities such as wastewater treatment plants, sewer systems, and water distribution systems that rely on gentle slopes to function properly (Keaton, 1987).

ADVERSE SOIL CONDITIONS

Many areas of Utah are underlain by soils that can cause damage to building foundations and roads if not identified and considered in engineering design. Such soils include expansive,

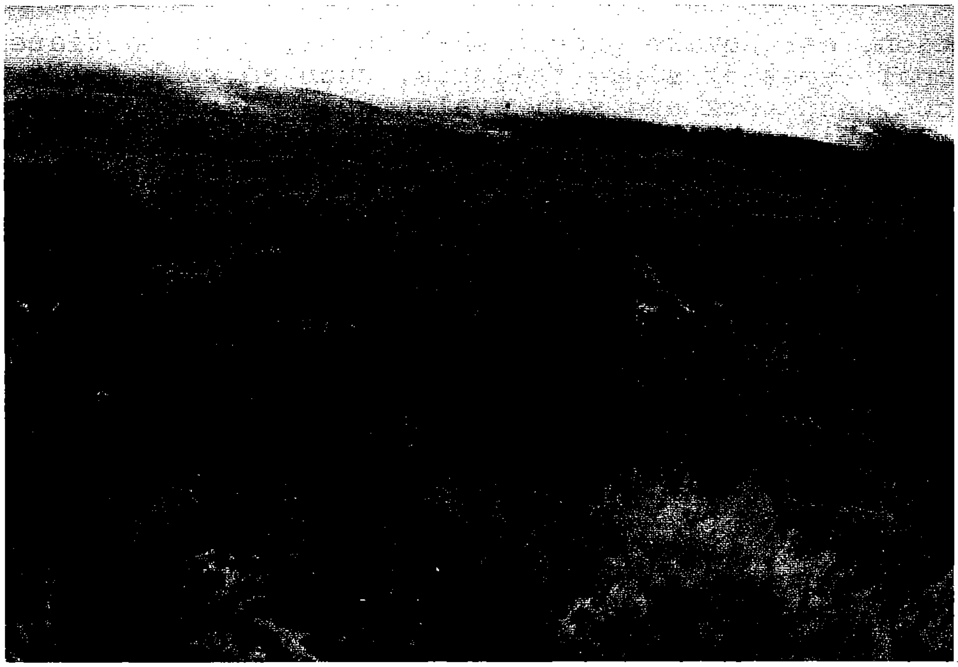

Figure 4. Fault scarp accompanying the 1983 Borah Peak, Idaho earthquake, magnitude 7.3.

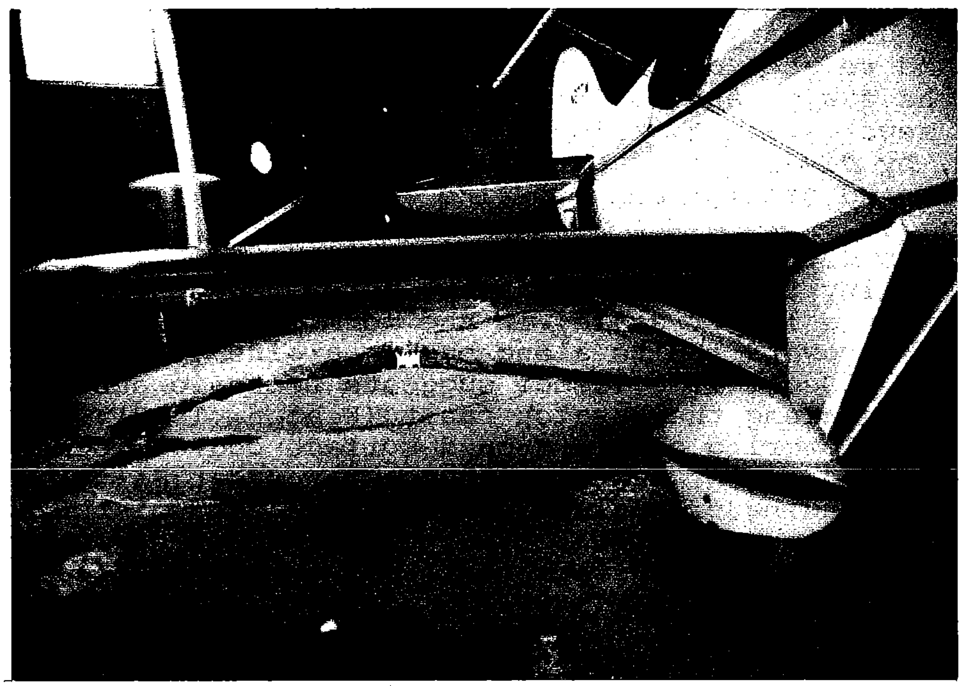

Figure 5. Expansive soils of the Mancos Shale damaged a hanger floor at the Moab airport.

collapsible, and gypsiferous soils. Foundations can crack or shift when volumetric changes occur in these soils with changes in moisture content.

Alternate wetting and drying of soils containing a high percentage of clays, particularly sodium-rich clays, can cause the soil to expand and contract, thus damaging structures built on these soils (Fig. 5). Such expansive soils are usually derived from shale formations, many of which are also susceptible to landsliding. Most of these shale formations are found in the Colorado Plateau region of eastern and southern Utah, and include the Mancos Shale, Tropic Shale, Chinle Formation, Brushy Basin Member of the Morrison Formation, and the Green River Formation. Shale is also present in the mountainous areas of north-central Utah, where the most troublesome formation is the Manning Canyon Shale. Approximately 10-15 percent of the State is underlain by shale that weathers to expansive soils. An unknown, but probably large part of Utah is subject to expansive soils derived from weathered volcanic tuffs, predominantly in central and south-central Utah, and from alluvial clays such as those in the Great Salt Lake Desert and the central parts of many Basin and Range valleys. Construction in areas of expansive soils often cannot be avoided, but the hazard they present can be mitigated by identification and proper engineering.

Collapsible soils, also known as hydrocompactible soils, experience a volumetric decrease when wetted. They are found chiefly in rapidly-deposited, geologically young materials in moisture-deficient environments, such as alluvial-fan and debris-flow deposits. These materials contain numerous void spaces, and percolating water from precipitation, irrigation, or lawn watering causes strength reduction of the soil resulting in collapse. Damage to buildings and roads can develop when ground subsidence from hydrocompaction occurs over a sufficiently large area. Subsidence may also occur in certain soils that contain gypsum due to dissolution of that mineral by infiltrating waters and subsequent collapse of the internal soil structure. Although it is not known how large an area of Utah is covered by collapsible soils, the most widespread documented occurrence of this problem has been along the base of the Hurricane Cliffs in southern Utah, particularly in Cedar City and Hurricane. Collapsible soils have also been identified in or near Scipio, Nephi, Kanab, Vernal, Monroe, Richfield, and in various areas of Utah County.

OTHER HAZARDS

Several other geologic hazards are present in Utah that are less widespread and/or less of a threat to life and property. However, on a site-specific basis, their identification is as important as the major hazards. These hazards include active sand dunes, ground subsidence from ground-water withdrawal or organic soil decomposition, accelerated soil erosion, mine collapse, abandoned but open mine shafts, sinkholes, radon, and snow avalanches.

Active dunes are found chiefly on the wide, flat floors of western Utah basins, and in some areas of the Colorado Plateau in southeastern Utah. Shifting dune sand has necessitated temporary road closings due to poor visibility, and has required increased road maintenance as sand must be removed to keep roads open. In addition, dunes stabilized by vegetation can be reactivated if the vegetation is damaged.

Ground subsidence and ground cracking from ground-water withdrawal has occurred in the Milford area. North of Enterprise in the southern part of the Escalante Desert, ground-water pumping for irrigation has been almost continous since the 1920s, causing decreases in the elevation of the water table by as much as 70 feet. Ground subsidence in this area has yet to be documented, but this and other areas of regional ground-water withdrawal are candidates for a future occurrence of this hazard.

The east shore area of Great Salt Lake contains some of the largest accumulations of peat in the State. Ground subsidence in peat bogs can pose a hazard as the organic materials decompose, release gas, and compress under the weight of overlying structures. In addition, the methane gas produced is highly combustible and can concentrate in structures built over these soils.

Accelerated erosion of soil by wind or a combination of wind and water and/or poor farming or rangeland practices has occurred in many areas of the State, including the Blue Creek-Howell (Box Elder County), Rabbit Gulch (near Duchesne), Montezuma Creek (San Juan County), and Muddy Creek (near Orderville) watersheds (R.C. Rasely, U.S. Soil Conservation Service, personal commun., July, 1988). Soil piping, a form of subsurface erosion that is most common in fine-grained Holocene alluvium, has also occurred in many areas of the State. One notable case was observed on agricultural land near Roosevelt in November, 1987. The collapse of an underground pipe in fine-grained alluvium along a stream bank created a 10-20 ft deep gully 150 ft long and 30 ft wide in a farmer's field (Rasely, 1988). Piping is presently active on the floodplain of Montezuma Creek in southeastern Utah (Fig. 6), and has locally undermined gravel roads in the area.

Collapse of underground mine tunnels has caused ground subsidence in the State, but to date has not caused much damage to structures or loss of life. By contrast, a number of people have lost their lives by falling into abandoned mine shafts that were not properly sealed.

Sinkholes can be life-threatening if they form quickly, but more often they only damage land and structures. Sinkholes generally form in limestone and dolomite, in response to

Figure 6. Soil piping and collapse near Montezuma Creek, San Juan County.

chemical dissolution of the rock's calcium carbonate by infiltrating precipitation and subsequent collapse of overburden material. Sinkhole formation has been reported in northern Utah, particularly in the Uinta and Wasatch Mountains and in the Bear River Range. Sinkholes due to dissolution of gypsum have occurred in agricultural fields in the Uinta Basin near Vernal.

The hazard from radon, a naturally occurring decay product of radium, in turn a decay product of uranium, has increasingly become a concern in the United States, including Utah. Although known to present a radiation hazard upon decay, only recently has it been shown to occur in buildings in higher than expected concentrations. Geologic factors affecting radon gas concentration include rock or soil type, permeability and porosity of surrounding rock or soil, and water saturation of soils that overlie the potential radon source. Rock types considered by the Environmental Protection Agency to potentially contain high radon concentrations include metamorphic rocks, granites, and organic-rich black shales. For a radon hazard to be present, Tanner (1986, in Sprinkle, 1988) cites four conditions that must be met: 1) radium in the ground, 2) ease of radon movement in the ground, 3) porous building materials or openings below grade, and 4) lower atmospheric pressure in the building. At present, not much is known about the extent of the radon hazard in Utah. However, radon monitoring is currently taking place in 750 homes throughout Utah, in areas where elevated levels of radon are suspected to occur. In addition, generalized maps identifying rock types in Utah associated with above average concentrations of uranium and radium, and point source areas such as uranium mines, mills, geothermal and thermal areas, and fault zones have been compiled (Sprinkle, 1987, 1988).

Snow avalanches have caused much damage and many deaths in Utah, principally in the Wasatch Range of northern Utah. Volcanos are present in western Utah, from Delta south to St. George. All are currently inactive or dormant, but some have been active within the last 1000 years. The greatest hazard from volcanic eruptions is considered to be from air-born ash settling in the State from active volcanos west of Utah (Christenson, 1986).

SUMMARY

Natural geologic processes generally receive little attention except when they become hazards to life and property. The major geologic hazards affecting Utah include slope failures, flooding, shallow ground water, earthquakes, and adverse soil conditions. Slope failures have caused much damage in recent times, as evidenced by the Thistle landslide of 1983. Many populated areas at the base of the Wasatch Mountains and Wasatch Plateau have suffered damage from debris flows and flash floods. Throughout the State, shallow ground water has caused flooding of basements and septic tanks, and has aided subsurface migration of pollutants. Likewise, problem soils exist in many areas, and require specialized engineering designs and techniques to be mitigated properly. Utah has yet to experience a large, destructive earthquake in a populated area in historical time, but the potential is great for widespread damage should one occur. The frequency of occurrence of some of these hazards, such as slope failures and flooding, was greatly increased during the wet cycle of 1982-1986, but has since returned to a more "average" rate. These and the other geologic hazards will continue to occur, and should remain a concern to every community in Utah.

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GEOLOGIC ENGINEERING ON THE NEW INTERSTATE ROUTE H-3 IN HAWAII

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ABSTRACT

Oahu is the most populous island in the State of Hawaii. Since the transportation needs of Oahu are increasing, improved and new facilities are required to accommodate physical, economic and social growth. The State of Hawaii Department of Transportation has accordingly been developing the highways to meet the island's travel demand projections.

The H-3 Project is planning, design, and construction of a 10-mile-long new highway that traverses the Koolau mountain range. The need for additional Trans-Koolau transportation capacity is based on travel demand projections matched against the capacity of the only two other existing Trans-Koolau

facilities, namely, the Likelike and Pali Highways. The proposed alignment of H-3 traverses North Halawa Valley on one side of the Koolaus and Haiku Valley on the other side.

The project includes a tunnel through the Koolau mountain range. The portals are referred to as the Halawa Portals and Haiku Portals, the former closest to Honolulu. The 1-mile-long twin-bore Trans-Koolau Tunnels will have two lanes in each direction. The total roadway width in each bore will be 38 feet. Other structures along the alignment include a short cut-and-cover tunnel at Hospital Rock, 2 miles of long-span segmental viaduct, and many approach cuts and fills.

The volcanic rock and soils along the alignment present a variety of foundation and excavation problems. The Hawaiian islands were formed from volcanic materials deposited generally in fairly thin basaltic layers. The stratified deposits weather at different rates. Volcanic materials generally weather to "MH" soils.

Relatively heavy rainfall supplies the groundwater which gets trapped in compartments formed by dikes within the Koolau mountain range. This source of water is used to meet part of the water supply needs of the island. The project must not adversely affect this valuable water resource. In general this means the tunnel has been sited above ground water levels within the mountain.

With a focus on the tunnel, the paper overviews the challenges of exploring the undeveloped project alignment, subsurface conditions encountered, and geologic impacts on the design and construction of proposed highway structures.

PROJECT DESCRIPTION

The H-3 Project is a new highway that traverses the Koolau mountain range to provide a new route from leeward (south) Oahu to windward (northwest) Oahu (Figure 1). Additional Trans-Koolau transportation capacity is needed because travel demand projections exceed the capacity of the only two other existing Trans-Koolau facilities, namely, the Likelike and Pali Highways. The proposed alignment of H-3 traverses North Halawa Valley on the leeward side of the Koolau Range and Haiku Valley on the windward side.

This four-lane interstate facility will have two lanes in each direction. The alignment starts at Halawa Interchange, traverses through North Halawa Valley at grade, passes on to long-span segmental viaducts in rugged areas near the tunnel and finally enters the Trans-Koolau tunnels. Exiting the tunnels in the head of Haiku Valley (west side), the facility runs easterly on a viaduct structure. A short cut-and-cover tunnel is called Hospital Rock. From there it continues at-grade to an interchange with Likelike Highway, skirts the boundary of Ho'omaluhia Park then proceeds on to the existing Halekou Interchange.

The typical roadway cross-section consists of two 12-foot lanes with a 10-foot right shoulder and a 4-foot left shoulder for a total width of 38 feet. The cross-section remains constant for the roadway, viaduct structures, and tunnels.

The total length of this segment of H-3 is approximately 10 miles, and the latest construction cost estimate for contract items is about $750 million. The current status of design and construction are discussed in the following paragraphs.

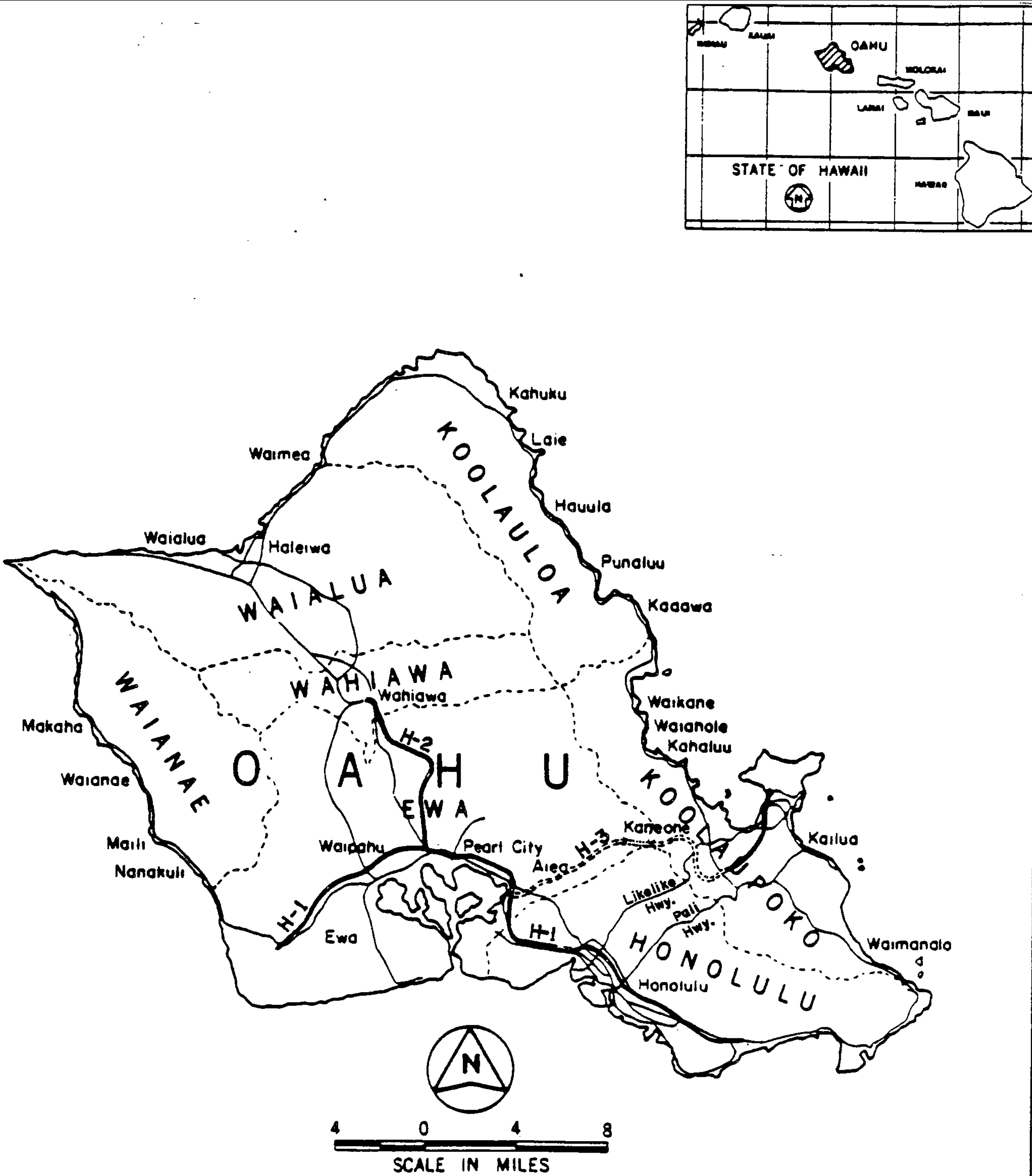


FIGURE 1. LOCATION OF INTERSTATE ROUTE H-3 PROJECT ISLAND OF OAHU, HAWAII

Current Status of Construction

Construction of H-3 began with the Halekou Interchange last year (June, 1987) after lifting of a court injunction that had been long delaying work. Since then, construction of access roads to each side (or portal) of the Trans-Koolau tunnel has begun. The present construction status is as follows:

North Halawa Valley Access Road. Construction is underway on an access road up North Halawa Valley. Construction of the road started November 2, 1987. The trailblazer road has reached the portals of the Trans-Koolau Tunnel at the head of North Halawa Valley. The completed access road will be approximately 5 miles in length with 16 foot-wide paved surface (some 24-foot width for passing) and bridge crossings. The purpose of the road is to afford access into the densely vegetated valley for topographic surveys, geologic exploration, and construction of H-3. The road later will permit construction access to the viaduct foundations and Halawa portal of the Trans-Koolau tunnel. Completion date for the access road is expected to be January of 1989.

Halekou Interchange. Construction of the interchange started on June 16, 1987, after lifting of the injunction. In December, 1987, the section of H-3 from the Halekou Interchange to the Kaneohe Marine Corps Air Station (built over 10 years ago but not used) was opened to public traffic. Completion date for the Halekou Interchange is expected to be November, 1988.

Haiku Access Road. Construction bids for this project were opened on June 30, 1988. The 16 to 24 foot paved road 2.1 miles in length will be built for the purpose of providing access for the construction of the Hospital Rock cut-and-cover tunnel, the Windward Viaduct, the Haiku Valley Bridges, and part of the Trans-Koolau Tunnels. Construction started in October, 1988.

Current Status of Design

Several contract segments are currently under design. Highlighted below are the tunnel portions.

Trans-Koolau Tunnel. The Trans-Koolau Tunnel is a twin-bore vehicular tunnel with excavated dimensions of about 48 feet wide and about 35 feet in height and is approximately one mile in length. Portal buildings will house ventilation fans. A separated control center building will also house emergency equipment. The latest technology will be used in the design of the tunnel and control equipment. There are expected to be 2 major tunneling contracts with construction during 1990 to 1994.

Trans-Koolau Exploratory Tunnel. The 13-foot exploratory tunnel will be driven during final design of the Trans-Koolau Tunnels. The exploratory tunnel will reveal in detail the types of geologic conditions through which the Trans-Koolau tunnels must be driven, including the extent and characteristics of any soil or soil-like materials, depth of rock weathering, character and variability of the rock in situ, general groundwater regime, and amount and location of trapped dike water. This information is vital in developing a design compatible with anticipated tunneling conditions. It will provide key information for preparing bids and will provide a basis for contingency measures for potential construction problems. The project was advertised for construction bids in September, 1988. Bids were opened in October, 1988, and construction is expected to begin in January, 1989, when the North Halawa access road is completed.

GEOLOGIC SETTING

Regional Geology

The earth's surface is understood to be broken into about ten major plates and several smaller plates, each several miles thick, which are all moving in various directions relative to one another. The Hawaiian Islands sit almost in the middle of one of the largest of the plates, the Pacific plate. A source of heat (hot spot) deep within the earth continually generates new magma (molten rock), which feeds the Hawaiian volcanoes. The hot spot has remained relatively stationary for many millions of years, while the Pacific plate slowly moved over it, toward the northwest, thus producing the linear chain of volcanoes that created the islands. Thus the islands to the northwest are older than those to the southeast.

As rock erupted from the interior of the earth and hardened, it gradually piled up, layer upon layer, to build the great mountain range that constitutes the Hawaiian Islands. Each island is the top of an enormous volcanic mountain, modified by stream and wave erosion. On the older islands, streams have cut deep canyons and waves have removed a large proportion of the original volcanoes. In comparatively recent times, a brief revival of volcanic activity occurred on the more northerly of the major islands, including Oahu.

Physiography

The island of Oahu has four major geomorphic provinces (Figure 2): the Koolau Range, the Waianae Range, the Schofield Plateau, and the Coastal Plains (Macdonald, et al., 1983). The Trans-Koolau Tunnels will be constructed in the Koolau Range.

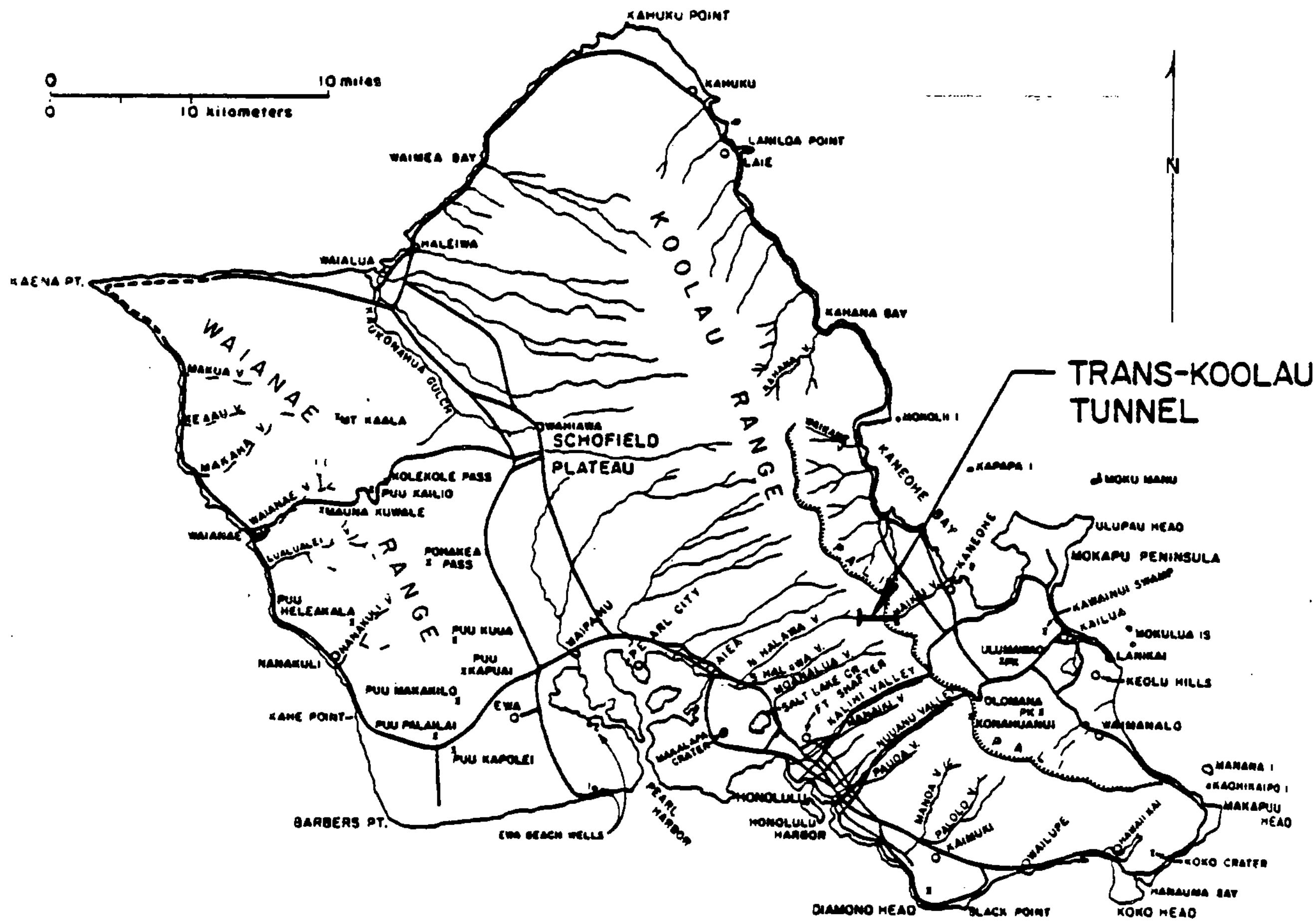


FIGURE 2. GEOMORPHIC PROVINCES, ISLAND OF OAHU, HAWAII

The Koolau Range forms the eastern part of the island. Puu Konahuanui, the highest point, is at an elevation of 3,105 feet. The range is 37 miles long and is deeply eroded by streams, and in places it has high sea cliffs along the sea shore. It consists predominantly of thin, narrow basaltic lava flows, piled one upon the other like shingles, with minor amounts of volcanic ash and numerous dikes.

The Pali (or great cliff) on the windward side of Oahu, with its nearly vertical, fluted wall, is 22 miles long and 2,000 feet high. Its origin is complex, but its present form is due chiefly to stream erosion and subsequent filling of the valley floors by stream deposition while the headlands were battered back by the sea. Great amphitheater-headed valleys cut their way to the top of the Koolau Range and beyond in places, so that some of the leeward valleys are beheaded. The Haiku portals are approached through one of these amphitheater-headed valleys called the Haiku Valley.

Types Of Lava

Most lava flows in Hawaii can be classified geologically either as "pahoehoe" (pah-hoay-hoay) or "aa" (ah-ah). According to Cas and Wright (1987), "Pahoehoe is generally a very fluid and fast flowing lava. Generally small, highly mobile flows advance as a coherent unit with a smooth rolling motion. Larger, less mobile flows advance by protrusion of bulbous 'toes' of lava...Aa Flows...advance much more slowly. The jagged flow-front creeps forward and steepens until a section becomes unstable and breaks off...Internally, pahoehoe lavas are characterized by large numbers of smooth, regular spheroidal vesicles. Many flows contain more than 20% vesicles, though it is not uncommon to find parts of flows with 50% vesicles...Internally, aa lava is characterized by irregular elongate vesicles that are drawn out in response to internal flow and a stratification consisting of a solid massive lava body sandwiched between layers of fragmented clinker that may be welded together..."

Geologic Structure

Lava flows are usually broken by cracks or joints. In most Hawaiian lava flows the joints formed normal (perpendicular) to the surface by cooling. The joints are typically irregular and are broken by joints resulting from contraction in other directions, and from continued movement of the flow as it consolidated. They therefore produce little or no columnar appearance that is typical in other basalts. The lava is simply broken into many irregular joints or rectangular blocks. The complex of irregular joints is important in governing the way in which the rock breaks during excavation, and in allowing the passage of groundwater through the rock.

Vesicles, lava tubes, intrusive dikes, and rock weathering are integral parts of the geologic structure of lava flows. More information on these subjects is given in the next chapter.

GEOLOGIC FEATURES OF ENGINEERING SIGNIFICANCE

Geologic features of significance at the tunnel site include displaced or transported soils, residual soils (soils derived from in-place weathering of the parent bedrock), weathered rock, and unweathered rock. In this chapter these features are presented in the following manner. First, the transported soil deposits which will affect a limited amount of the construction are discussed. Second, site specific data are given on the Koolau series basalt flows. Third, the very important effects of weathering are described. Finally, groundwater conditions are described.

Transported Soil Deposits

All of the contracts have been or will be constructed partly within or adjacent to transported soil deposits. Soils displaced and deposited by streams are called alluvial deposits or alluvium. Soils displaced by gravity on relatively steep slopes are called colluvial deposits or colluvium.

Alluvium. Alluvium is generally associated with the streams that traverse the North Halawa and Haiku Valleys. The stream deposits vary greatly from fine grained to coarse-grained soils, including boulders. In places, the alluvium may be underlain or overlain by colluvium. In all cases, alluvium is derived from the Koolau Series basalt flows, which are discussed later.

Colluvium. Colluvium consists of transported soils that have been displaced by gravity from their original locations on relatively steep slopes. These soils resemble residual soils but are generally less dense, more moist, and weaker. Grain size distribution and Atterberg limits vary, but the colluvium is generally classified as a highly plastic silt (MH soil). Significant amounts of clay, sand, gravel, cobbles, boulders, and weathered rock pieces may also be present. Colluvium deposits are also derived from the Koolau Series basalt flows as described below.

Koolau Series Basalt Flows

The Koolau Series are the oldest lava flows within the Koolau Range. A description of the Koolau flows and rock characteristics, excerpted from Wentworth (1951), emphasizes many of the attributes of these basaltic lavas:

> "The Koolau formation consists almost wholly of basaltic lava flows erupted from numerous vents along a fissure line over 30 miles long. The flows average 10 feet or less in thickness, and the entire mass, from the deepest exposures some 2,000 feet below the dome surface to its top, shows exceptional uniformity...

"Both pahoehoe and aa lava flows are common in the Koolau dome, with a preponderance of the aa flows in the better-exposed lower flanks of the mass... It is important to emphasize that the aa flows include both the clinker phase with which most people are familiar and also the dense interior which is a component part of all aa flows... The lava formations are marked by various features typical of basaltic flows (Wentworth and MacDonald, 1953). There is a close-spaced jointing such that even in the thicker aa masses or in the pahoehoe it is impossible to produce any considerable fraction of 1- or 2-ton rocks. Regular polygonal or columnar jointing is, however, very rarely seen, and then only in an occasional thick flow."

"...The contact of lava flows, one on another, is marked by great irregularity occasioned by the broken character of the pre-existing surface and by the chilling and increase in viscosity of the advancing lava. Only between the successive thin units of pahoehoe flows is there any approximation to intimate molding of one flow on another so as to make what could be called a sealed joint."

"Elsewhere, and generally throughout the mass, the contacts show numerous openings, loose blocks, and abundant bridging, which contribute to the high permeability of the lava formation."

The features of major engineering significance within the Koolau Basalt are the pahoehoe flows, the aa flows, clinker, ash, cinder, scoriaceous rock, dikes, and lava tubes. These are each discussed separately below. However, it is essential to recognize that these features naturally occur in variable and irregular distributions.

Pahoehoe. Pahoehoe is a vesicular basaltic. The vesicles generally are rounded with smooth interiors. The vertical thicknesses of individual flows generally vary between about 10 and 70 feet with an average of about

30 feet. Zones of pahoehoe encountered by recent project borings are generally described as moderately weathered to unweathered, low hardness to very hard, and intensely to widely fractured with RQD* values typically between 50 and 100 percent. These descriptive terms are defined in Table 1.

Aa. Aa is basaltic, vesicular lava typified by a rough, jagged, clinkery surface. The vesicles often are abundant and elongate and the vesicle walls appear irregular and rough. The vertical thicknesses of individual flows generally vary between about 1/2 and 40 feet with a predominance of values between about 1 and 5 feet and an average of about 6 feet. Zones of aa encountered by recent project borings are generally described as highly to slightly weathered, low hardness, and intensely to closely fractured with RQD* values typically between 40 and 80 percent.

Clinker. Clinker is formed by a cooling crust of slow moving aa lava flow. As the flow is pushed forward, the crust breaks and rolls over the front of the flow. This crust is then incorporated into the base of the flow as a clinker layer. The result is a discontinuous layer on the top and bottom of flows that is composed of blocky loose and partially welded lava fragments. Material size ranges from coarse sand to large gravel. Fragmentation of clinker observed in drill cores sometimes results, in part, from coring operations.

Clinker often weathers preferentially because of its large surface area and great permeability of the clinker layers. Clinker may contain clay materials and weathered basalt. This phenomenon is often enhanced below the water table or at elevations influenced by fluctuating water table levels.

Clinker zones were encountered in the H-3 project borings. The vertical thicknesses vary between about 1/2 and 20 feet with a predominance of values between about 1/2 and 4 feet and an average of about 3 1/2 feet.

Clinker zones encountered in recent project borings were generally described as extremely to highly weathered, soft to low hardness, and crushed to intensely fractured with RQD* values typically between 0 and 50 percent.

Ash, Cinder and Scoriaceous Rock. Ash characteristically consists of inorganic, uncemented, pyroclastic material consisting of gravel size fragments or smaller. The ash soils generally are encountered near ground surface and have relatively low strengths and high plasticity. Thicknesses vary between about 5 and 15 feet.

Cinder consists generally of partly cemented, glassy, vesicular volcanic ejecta, also gravel size or smaller. Cinder and weathered cinder soils are relatively strong and stable. However, when disturbed their inherent cementation may be lost causing a reduction in strength.

Scoriaceous rock consists of pyroclastic, volcanic slag ejecta characterized by dark color, high vesicularity, and both glassy and crystalline textures. In borings these materials are generally encountered in a highly or extremely weathered state to the extent that often they can not be differentiated from clinker.

Lava Tubes. The feeding rivers of pahoehoe flows quickly crust over and develop more or less continuous roofs. Thenceforth the lava stream flows within a tunnel of its own making. If the tunnel crust or wall is breached, lava runs out leaving an empty tunnel. This tunnel is known as a lava tube. Wentworth (1951) points out that, "Lava tubes are commonly encountered in any extensive tunnel through the Koolau mass and are exposed in outcropping cliffs. Most commonly these are 1 or 2 feet in diameter, but a number are known that are 20 feet or more wide and generally somewhat less in height..."

TABLE 1
DEFINITIONS OF TERMS USED TO DESCRIBE ROCK
H-3 TRANS-KOOLAU EXPLORATORY TUNNEL

WEATHERING:

Degree of weathering as defined here is physical disintegration due to chemical alteration of the minerals in the rock. Terms and abbreviations used to describe weathering are:

EW Extremely Weathered - The original minerals of the rock have been almost entirely altered to secondary minerals, even though the original fabric may be intact (Saprolite)

HW Highly Weathered - The rock is weakened to such an extent that a 2-inch diameter core can be broken readily by hand across the rock fabric (Saprolite)

MW Moderately Weathered - Rock is discolored and noticeably weakened, but a 2-inch diameter core cannot usually be broken by hand across the rock fabric.

SW Slightly Weathered - Rock is slightly discolored, but not noticeably lower in strength than fresh rock.

UW Unweathered (fresh rock) - Rock shows no discoloration, loss of strength, or any other effect of weathering.

HARDNESS:

S Soft - Reserved for plastic material alone.

F Friable - Easily crumbled, pulverized or reduced to powder by hand.

LH Low Hardness - Can be gouged deeply or carved with a pocket knife.

MH Moderately Hard - Can be readily scratched by a knife blade; scratch leaves heavy trace of dust and scratch is readily visible after the powder has been blown away.

H Hard - Can be scratched with difficulty; scratch produces little powder and is often faintly visible.

VH Very Hard - Cannot be scratched with pocket knife.

TABLE 1

DEFINITIONS (continued)

FRACTURE SPACING:

The general fracture spacing is noted according to the following criteria:

C Crushed - 5 microns (mechanical clay) to 0.1 foot.

IF Intensely Fractured - 0.05 to 0.1 foot (contains no clay).

CF Closely Fractured - 0.1 to 0.5 feet

MF Moderately Fractured - 0.5 to 1.0 feet

WF Widely Fractured - 1.0 to 3.0 feet

VF Very Widely Fractured - +3 feet.

RQD (ROCK QUALITY DESIGNATION):

The percentage of each coring interval that is sound, competent (UW and SW) lengths of core more than 10 cm (4 inches) long, as measured along the centerline of the core. This is the strict definition of RQD.

A variation is RQD* (DEERE AND DEERE, 1988). This is same as RQD except moderately weathered (MW) rock pieces are also counted. Implicit to the definition of RQD or RQD*, a value of zero (0%) means that the core run either did not meet the criteria above or was too weathered (EW or HW) to calculate a value.

Intrusive Dikes. The Koolau Range is an erosional remnant of the Koolau volcanic dome, which extended along a rift zone 30 miles long (Hirashima, 1971). The rift zone is marked by closely spaced multiple dikes, termed a dike complex. The marginal dike zone differs from the rift zone in that the dikes are widely spaced. Dikes in the marginal dike zone are spaced tens or hundreds of feet apart; in the rift zone they are spaced inches or a few feet apart.

The vertical basalt dikes are parallel or nearly parallel to the rift zone and are less permeable than the rocks they intrude. Hence, each pair of parallel dikes form two walls of an elongated groundwater reservoir. For more detailed discussions on the rift and marginal dike zones of the Koolau Range, see the report of Takasaki, Hirashima, and Lubke (1969).

The thickest dikes are in the northwestern rift zone where the H-3 exploratory tunnel and main tunnel will be constructed. Most of the dikes strike northwest and are believed to be between 3 and 7 feet in thickness, but dikes thicker than 10 feet may exist in the project area.

Persistent trade winds, which blow from the northeast across the Hawaiian Islands, bring large quantities of moist air to the Koolau Range. When these winds are deflected up and over the range, the moisture in the air condenses and falls as rain. The rain that infiltrates the porous ground percolates through the lava to dike-water reservoirs. Where dike reservoirs have been breached by stream erosion, ground water escapes into streams.

Bedrock Weathering Characteristics

Residual Soil. On the leeward side of the Koolau Range, the slopes are gentler than on the windward side and very deep weathering has occurred.

The rocks have been subjected to such extensive chemical weathering that they sometimes can be cut readily by hand-excavation tools. Relict joints of the basalt flows still exist even in areas of extensive weathering. A general description of these residual materials can be obtained from the investigations and experience gained from the construction of the Wilson Tunnel (Peck, 1981).

Residual soil is present throughout the site and is derived from the in situ decomposition of the parent vesicular basalt bedrock. The major component of this material is soil-like, although it may contain pieces of friable, weathered rock. The upper zone of residual material has decomposed to such an extent that no remnant or relict rock structure is visible. Grain size distribution and Atterberg limits vary but the residual materials when disaggregated generally classify as highly plastic silts (MH), grading to silty sands (SM). Significant amounts of clay, gravel, cobbles, and weathered rock pieces may also be present. Residual soil without relict structure generally behaves from an engineering standpoint like a plastic silt.

<u>Weathered Rock</u>. Weathered rock is also derived from the in situ decomposition of the parent vesicular basalt bedrock. This material generally consists of soil, weathered rock, and unweathered rock components in a heterogeneous matrix with respect to weathering. The material exhibits wide variations in density, moisture content, and strength. Usually much of the cohesion of the parent rock is retained, and remnant rock structure including compositional banding and relict jointing is exhibited. Rock terminology is used to describe this material, with the amount of weathering being an important factor. The weathering categories are given in Table 1. The term "saprolite" has been used for this project to mean extremely or highly weathered basalt. Although saprolite is technically considered severely weathered rock, laboratory soil testing can be used to

classify disaggregated materials. Grain size distribution and Atterberg limits vary but saprolite materials when disaggregated generally classify as highly plastic silts (MH), grading to silty sands (SM). The results of grain size distribution or sieve tests are highly dependent on the length of time the samples are tested. When the sieves are vibrated longer more disaggregation takes place and therefore a greater percent of particles in the clay and silt size result. Atterberg limit test results on sample fractions passing the #40 sieve are shown in Figure 3 for samples tested from the Halawa tunnel borings near the portal.

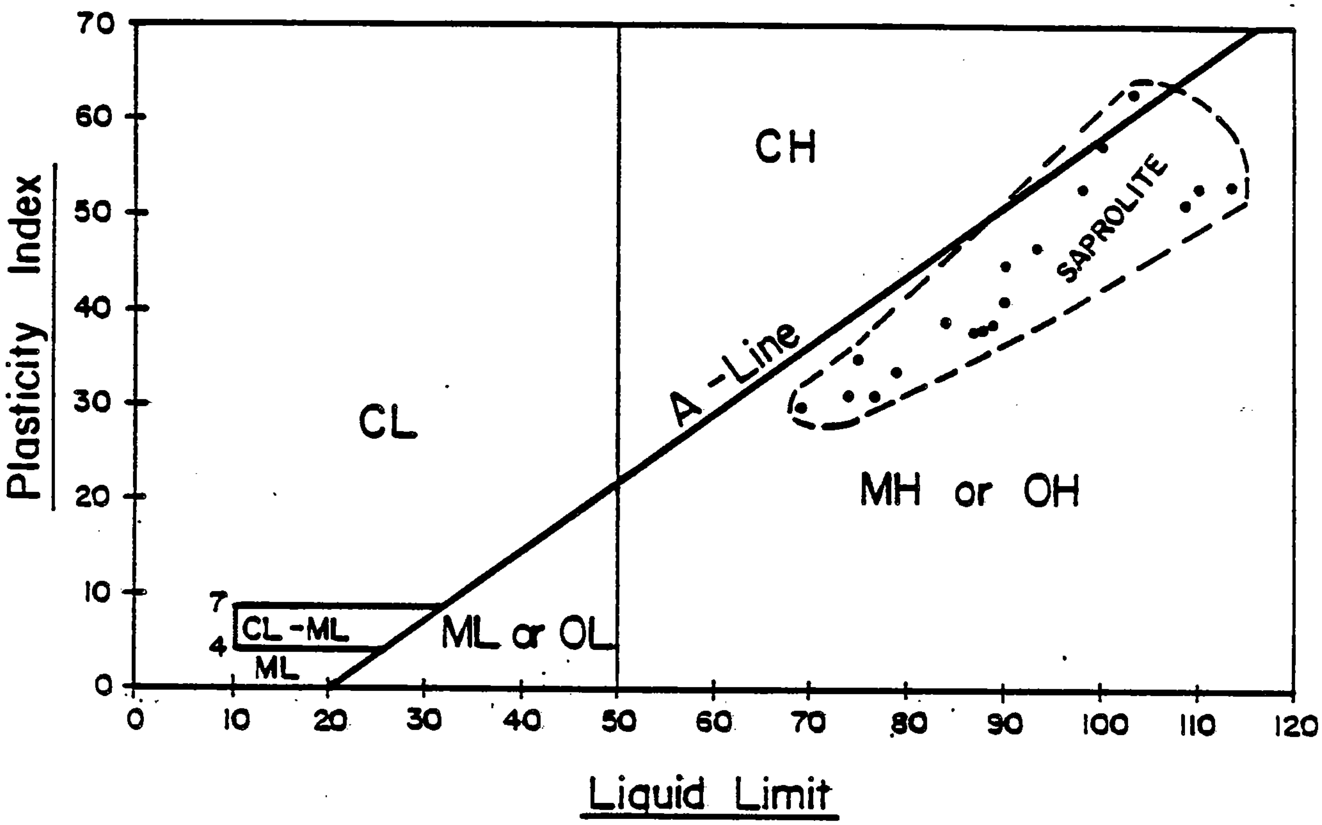
The weathered rock zone, including saprolite, is generally thicker and more extensive than the residual soil zone. Strength is highly variable and dependent on the type of basalt (pahoehoe, aa, or clinker) and the degree of weathering (Figure 4).

Unweathered Rock. Unweathered rock (UW) is unaltered basalt generally with no discoloration, loss of strength, or any other effect of weathering. Rock-mass behavior is generally governed by the frequency and characteristics of joints. RQD values for unweathered rock generally range between 70 and 100 percent. The intact rock between joints is generally moderately hard to very hard (See Table 1 for definitions). Unweathered rock zones range from relatively massive (with widely spaced, non-persistent jointing) and well interlocked to relatively loose, non-interlocked, and very blocky. Welded clinker zones may also occur as unweathered rock zones.

Groundwater Conditions

The permeability of basalts exceeds that of most other rocks. The relatively high permeability is due to the presence of the following features:

- Interstitial spaces in the clinker
- Cavities between flows



NOTE: TESTING FOLLOWED ASTM STANDARDS, FOR WHICH TESTING IS ON MATERIALS PASSING NO. 40 SIEVE.

FIGURE 3. ATTERBERG LIMITS FOR SAPROLITE

TUNNEL BORINGS NEAR NORTH HALAWA VALLEY PORTAL

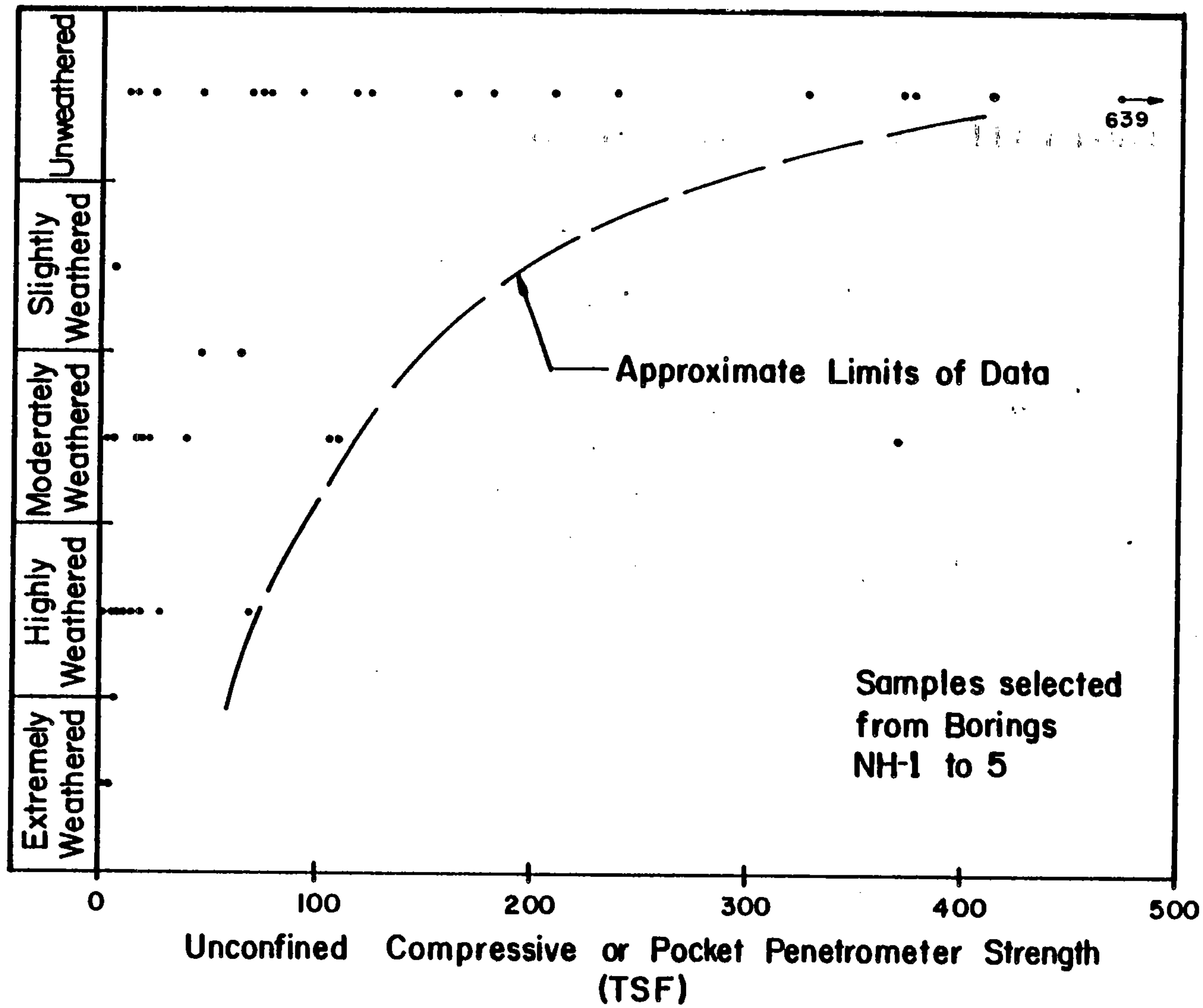


FIGURE 4. EFFECT OF WEATHERING ON STRENGTH
TUNNEL BORINGS NEAR NORTH HALAWA VALLEY PORTAL

- o Cooling or shrinkage cracks
- o Lava tubes
- o Gas vesicles
- o Joints produced by tectonic forces after the flows come to rest
- o Tree-mold holes.

Some lava tubes are 20 feet in diameter, and when they occur in the zones of saturation, they are capable of transmitting large quantities of water.

Five pump-in permeability tests were performed at the North Halawa tunnel portal using packers to provide in situ permeability data on widely jointed, unweathered rock and highly weathered, fractured rock. Permeability tests run in a few of the borings yielded permeability values on the order of 10^{-3} centimeters per second. Permeability appears to be inversely proportional to weathering.

Monitoring data from observation wells in the vicinity of the planned tunnels at the Halawa portals indicate groundwater levels between about elevation 975 and 1030 during the period of March to May, 1988. However, ground water is highly controlled by geologic structure within the mountain and little data exists to predict groundwater levels through the Koolau Range along the tunnel alignment as discussed in the following paragraphs.

The higher portions of the Hawaiian mountain ranges have rainbelts because they protrude into the tradewinds. Because the basalt is so permeable, the fresh rain water flows into the island mass and becomes trapped within vertical compartments bounded by dikes which is shown schematically in Figure 5. A large area of Oahu holds water trapped by dikes. To recover this water as part of the commercial and domestic water supplies, several tunnels have been driven into the region near the Trans-Koolau highway tunnel site although at lower elevations.

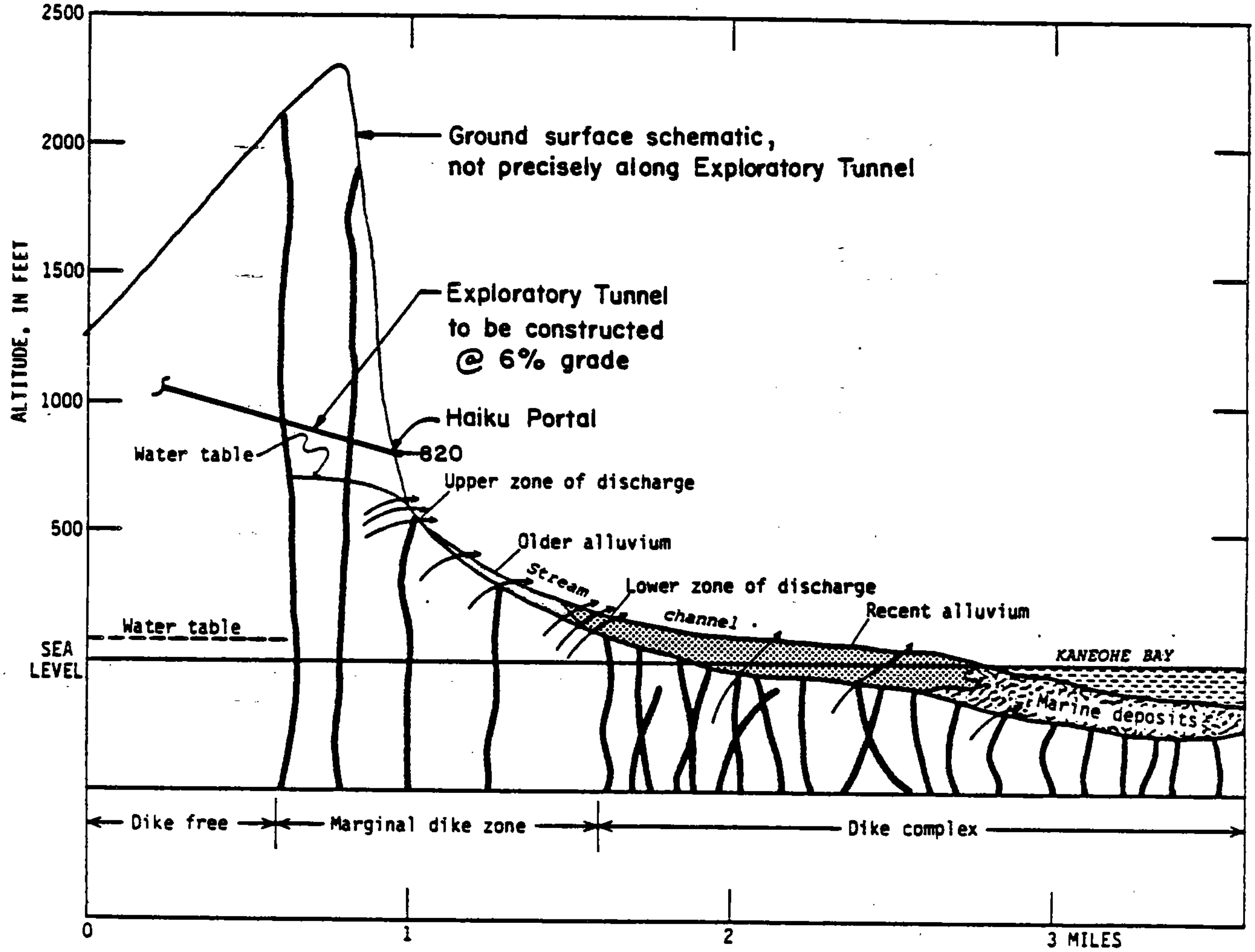


FIGURE 5. SCHEMATIC DIAGRAM OF POSSIBLE WATER LEVELS ALONG H-3 TUNNEL ALIGNMENT (MODIFIED FROM TAKASAKI AND MINK, 1985)

According to Takasaki and Mink (1985), near the crest of the Koolau Range the groundwater surface may be as high in elevation as 1000 feet in the northwest part of the Range and approximately 400 feet in the southeast. This situation is depicted in Figure 6 upon which is shown the projection of the Haiku portal of the exploratory tunnel at elevation 820 feet.

Streams And Waterfalls

Two major streams flow through the H-3 site: one along the North Halawa Valley and one along the Haiku Valley. Several tributary streams cross the alignment and feed these major streams. The valleys are remnants of differential erosion by streams mainly fed by waterfalls at higher elevations. Several waterfalls can be observed after heavy rainfall on both sides of the Koolau ridge.

No direct correlation between rainfall and stream flow has been made for the Halawa portal site. However, stream flow has been observed to increase markedly after heavy rains and subside to insignificant flows during drier periods. At best, stream flow quantities are erratic yet somewhat dependent on rainfall.

SUMMARY OF GEOTECHNICAL CONDITIONS

In summary, there are seven principal geologic units which affect H-3 construction: alluvium, colluvium, ash, cinder, residual soil, weathered rock, and unweathered rock. The character and behavior of these materials must be evaluated for each individual site. However, some generalities can be made based on the limited subsurface exploration and construction that has taken place to date.

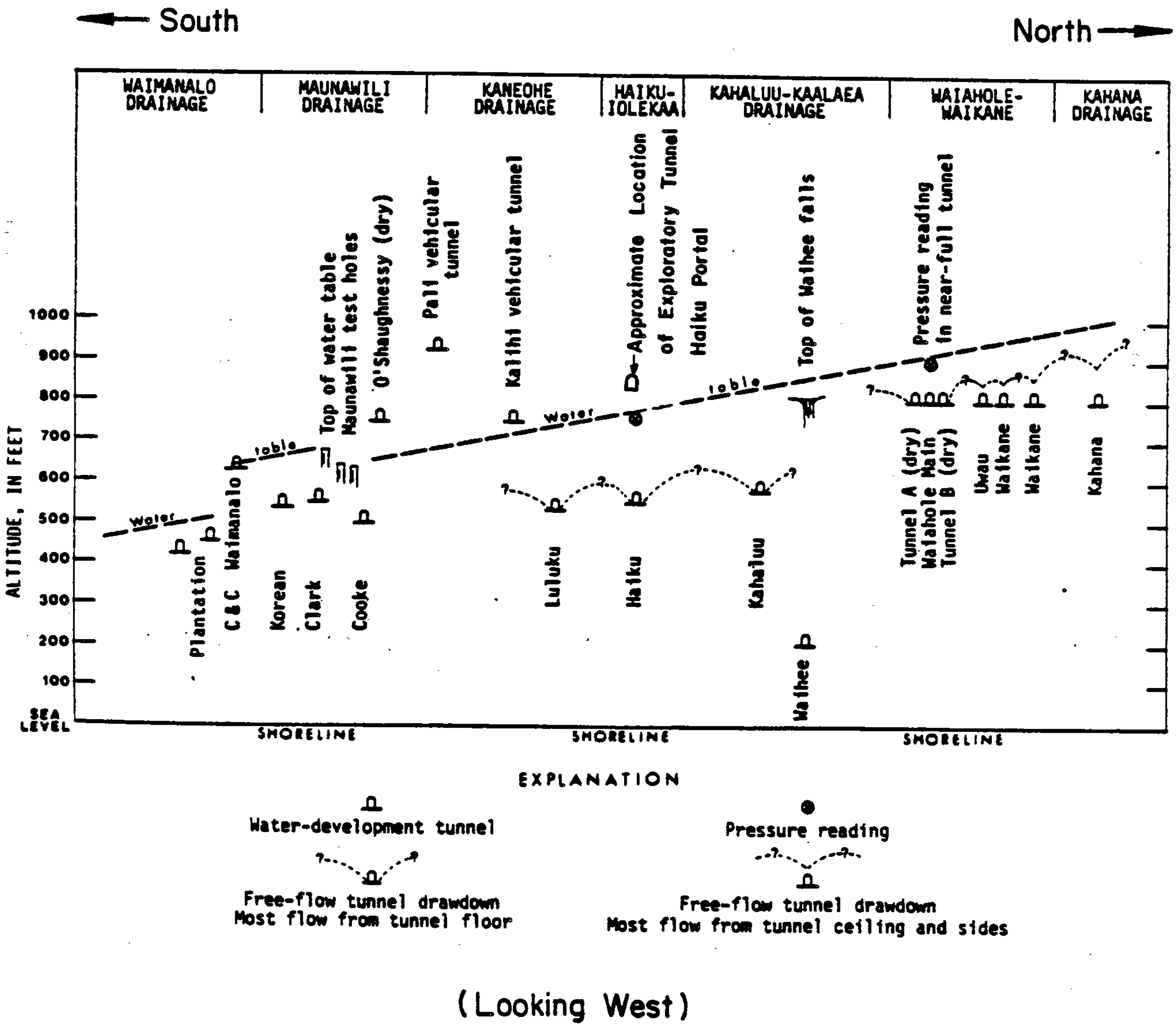


FIGURE 6. WATER LEVEL OF DIKE-IMPOUNDED GROUNDWATER (MODIFIED FROM TAKASAKI AND MINK, 1985)

Alluvium and colluvium deposits are generally acceptable for use in fills when they are reasonably well graded and compact. However, they are often poorly graded, highly variable, and not well compacted. In that case, they may pose problems due to erratic bearing capacity, high compressibility, low cohesion, boulders, and highly permeable zones.

Ash deposits are generally poor materials for any geotechnical purpose except perhaps landscaping. Depending on the plasticity index of the material, removal and replacement is typically the best option for most applications. Cinder is typically somewhat better than ash but may require densification and protection from erosion.

Residual soils are acceptable for several uses. However, in the presence of water and heavy construction traffic, the soils can degrade severely to such an extent that no work can be done until the soil dries out or a protective subgrade is placed. Therefore, it is important to direct water away from exposed soil and minimize disturbance by construction equipment. The Halawa side of the Trans-Koolau Tunnel and Exploratory Tunnel will be constructed partially in residual soil. Special precautions and excavation sequencing will be necessary to seal and reinforce these soils during tunneling.

Weathered and unweathered rock generally present their own geotechnical considerations. Compared to transported or residual soils, these materials typically have high allowable bearing pressures for foundations and end-bearing piles. Relatively steep cuts can be made and excavated materials generally make good fill. Tunnel excavation and support will require detailed design. Frequent clinker and saprolite zones occur. When these zones are weak and extensive, proper steps must be taken to mitigate their effect on ground mass behavior.

RECENT H-3 CONSTRUCTION EXPERIENCE

Construction has begun on the Halekou Interchange and the North Halawa Access Road. Additionally, the Halawa tunnel portal subsurface exploration required hand-excavated pads from which some observations were made. This construction experience is elaborated on below.

North Halawa Access Road. Temporary access roads to both tunnel portals are required for construction of the tunnels as well as other structures along the H-3 alignment. All construction traffic will be required to use these roads to the tunnel site. Construction of the 5-mile-long North Halawa Access Road includes cuts and fills, bridges, pavement, drainage, and erosion control measures. Grades on the roads are up to about 10 percent. The pavement consists typically of a combination of aggregate sub-base and asphalt pavement. The roadway width ranges between 16 and 24 feet.

Sequencing of the work began with a trailblazer road which was laid out ahead of the contractor's operations by the engineer and surveyors. The contractor cleared foot trails and survey sight lines for the establishment of the trailblazer alignment line and grade. A group of archaeologists worked in advance of roadway construction to survey and map all archaeological sites within the path of the trailblazer/access road corridor. In certain instances, the contractor's operations were temporarily suspended or re-routed around the sites to provide adequate time for the archaeologists. The significance and effect of the sites were determined by the FHWA and concurred by the State Historic Preservation Office. The sites determined significant (eligible for inclusion in the National Register of Historic Places), and which required further studies but did not require preservation, were excavated by the archaeologists in accordance with the FHWA/SHPO approved data recovery plans. The FHWA and HDOT paid all costs.

After the trailblazer road was constructed an evaluation in the field was made as to the best way to construct the full width of the road using a combination of rapidly constructible lattice bridges (the contractor's choice), cuts, and fills. Geotextile fabric walls were available in the contract but none were used in the field.

The major geotechnical problem encountered in the field was the abundance of water at the site due to record rainfall. Construction traffic combined with the wet conditions caused the trailblazer road substrata to soften making construction increasingly difficult. Slow progress was made during the rainy early part of 1988 because of these conditions. The effect of construction traffic is highlighted by the contrast between the cuts which stood well over time, as opposed to the roadway at the bases of the cuts which required frequent redressing and stabilization.

North Halawa Valley Tunnel Portal Exploration. Borings near the Halawa portal were completed in April, 1988. The need for subsurface geologic information was greatest at the Halawa portals because of the unknown extent and variation of residual soil and weathered rock materials at the site, which are sometimes soil-like and weak. Information on this condition was critical for making preliminary design decisions for the highway tunnel and portal structures, as well as the exploratory tunnel. The North Halawa Access Road had not yet been completed at that time, and helicopters had to be used to transport personnel, equipment, and supplies.

Five boreholes were drilled at the Halawa portals, totaling 910 linear feet. The number of borings for this phase of site exploration were limited because of the high costs associated with site inaccessibility.

Other key aspects of this specialized work included: triple-tube type coring to recover continuous samples of both soil and rock-like materials; inclined drilling to intercept the tunnel horizon from very few set-up locations (pads excavated by hand); and drilling with foam to reduce water requirements and the costly hauling of water by helicopter.

The paths and pad cuts for the drilling sites experienced the same performance characteristics as the North Halawa Valley Trailblazer Road. The cuts up to 10 to 15 feet high stood well with no reinforcement. However, the side hill paths softened progressively due to foot traffic and frequent rain.

CONCLUSIONS

1. The H-3 project will be constructed predominantly with and within weathered basaltic materials that have been displaced by streams (alluvium) and gravity (colluvium) or weathered in-place. The basaltic materials include pahoehoe, aa, clinker, ash, and cinder. Weathering has a significant effect on strength and plasticity.

2. Most of the soil/rock materials are usable from a geotechnical standpoint except the ash zones which typically need to be removed and replaced.

3. Variability of grain size and weathering are extremely important factors that must be considered in the design of foundations, cuts, fills, and tunnels. Ground behavior is controlled by the particular properties of the individual flows and other geologic units as well as their arrangement and combined properties.

4. The most severe construction problem that has been observed to date is the sensitivity of weathered basaltic soils to working and reworking by

construction traffic in the presence of water. Adequate drainage and protection of the soil from direct construction traffic has mitigated these problems to a large extent.

ACKNOWLEDGEMENTS

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Aesthetic and Safety Issues for Highway Rock Slope Design

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ABSTRACT

During the design phase of most new highway projects, rock slope planning is not typically carried out at the "state-of-the-art" level of rock mechanics. This is not a reflection upon the slope designers but more an admission of constraints inherent to highway slope design. These constraints often include the inability to fully characterize the geologic regime due to structural complexity or to insufficient data, an incomplete history of time-dependent variables such as ground water levels, and difficulty in determining appropriate design strength parameters for lengthy corridors.

To overcome these investigative and analytical deficiencies, rock slope designs generally incorporate one or more of:

- o careful excavation procedures (e.g. controlled blasting)
- o flexibility for design changes and stabilization during construction (e.g. rock bolting, shotcrete)
- o tolerance for minor slope failures (e.g. designed ditch geometry)

In recent years, there has been a movement to provide for aesthetically pleasing highway designs motivated by the landscape and environmental professions. This has produced admirable results when applied to structures such as overpasses and bridges and to soil slopes. But when aesthetic limitations are applied to rock slope designs, a conflict occurs with many of the tools which the slope designers rely upon. For example, the dislike by some of pre-shear blast hole imprints on rock cuts has caused some projects to modify well-proven, controlled blasting procedures. These procedures have been developed over many years to minimize damage to the rock which will ultimately form the permanent slope. Unless costly additional design precautions are taken in the absence of controlled blasting, the risk of slope failure and rockfall is increased, maintenance costs are greater and the safety of highway traffic is compromised.

Given this divergence of opinions how can the common goal of designing a safe and aesthetically pleasing roadway be accomplished? Firstly, the highway slope designers and the landscape architects must establish and maintain dialogue throughout the feasibility and design phases of a project. In some cases, methods can be developed to satisfy both aesthetic and engineering requirements, while in others compromise will be required. Secondly,

significantly increased budgets must be allocated for both the geotechnical design and construction for most projects that have aesthetic limitations.

INTRODUCTION

The history of highway rock slope designs which incorporates both rational rock slope design criteria and controlled blasting is relatively short; dating back to about 1960. The earliest project with which the authors were involved was the Skagway-Carcross Highway constructed in 1974 in Alaska.

From such beginnings, a rock slope design and excavation methodology has developed which attempts to minimize both the rock excavation costs and the risk of rock failure to the travelling public. Because the methodology cannot precisely quantify all the variables during the investigation in order to provide the optimum slope design, a number of design and construction techniques have been developed which either prevent or minimize the consequences of rock slope failure.

In recent years, certain rock slope designs are being required to meet historical standards for safety as well as newly imposed criteria for aesthetic appeal. In many cases these objectives are mutually exclusive and if a choice is to be made between aesthetic and safety requirements, then the question becomes one of compromise.

This paper summarizes the engineering procedures now being used for rock slope design and the extent, if any, to which these procedures can be modified to meet aesthetic considerations without compromising the safety of the highway. The parallel issues of whether the travelling public desires aesthetically pleasing rock cuts and whether they are willing to pay for them are not addressed; although the authors' opinion in this regard is captured in Figure 1.

CONSEQUENCES OF ROCK SLOPE FAILURE

The motivation for stable highway rock slopes is twofold; safety and cost.

Safety Implications

The safety issue is pointedly illustrated by the following court cases which show that, in most jurisdictions, highway authorities can be held liable for both negligent rock slope design and for inadequate rock slope maintenance.

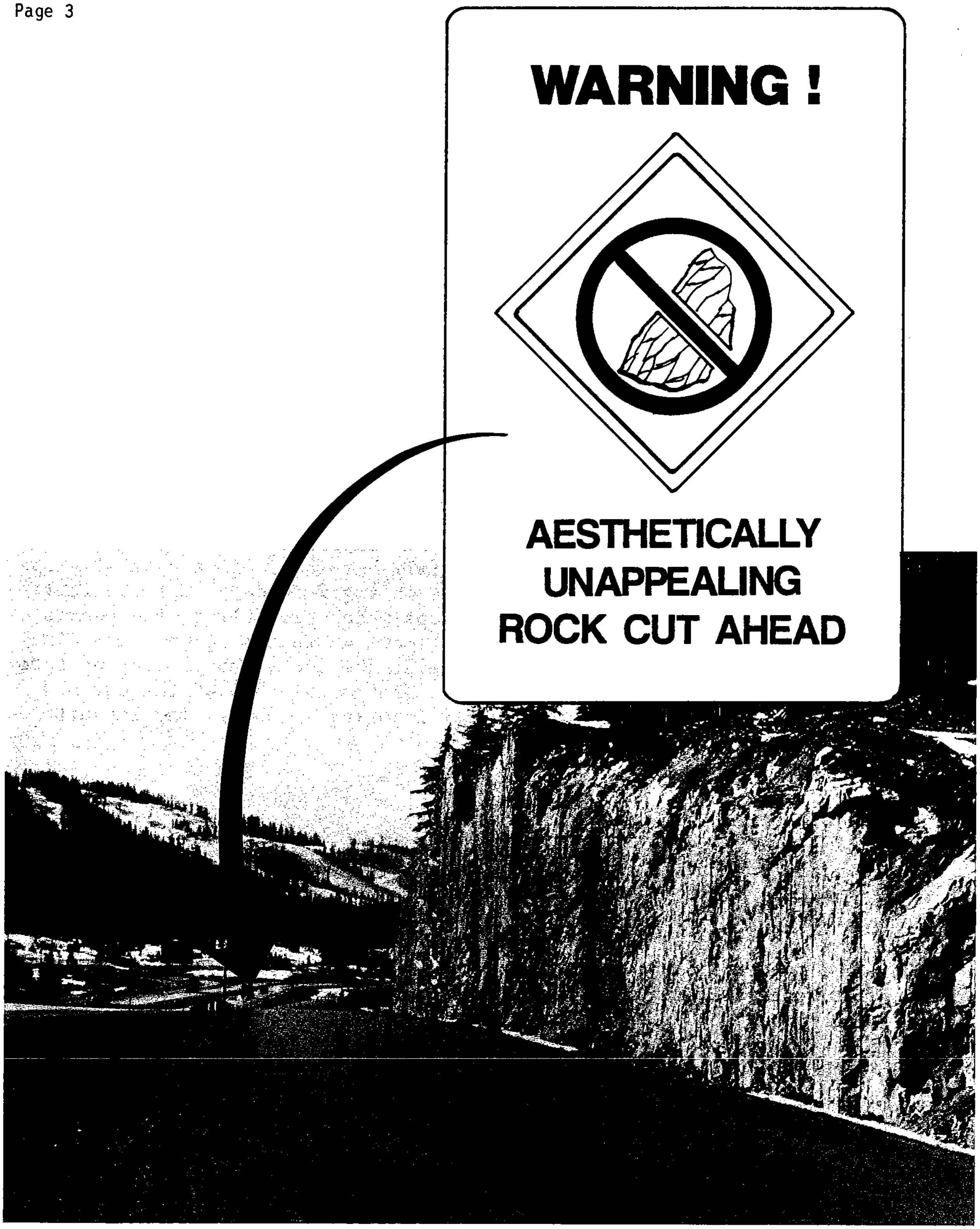


Figure 1
Point of View

State of Washington State Route 14

This accident occurred approximately 30 miles east of Vancouver Washington in November, 1985. An existing rock slope approximately 60 feet high failed resulting in a large debris pile that covered both the eastbound and westbound lanes of the highway and included blocks as large as 15 feet. Minutes after the failure, an unloaded logging truck travelling eastbound slammed into the failure mass, destroying the truck and seriously injuring the driver. Traffic on the highway had to be diverted through Oregon for several days while state maintenance crews removed rock debris from the roadway, some of which had to be drilled and blasted. The driver filed a lawsuit against the Washington State Department of Transportation, but an out of court settlement was reached in the amount of $175,000.

State of Oregon Interstate 84

This incident took place approximately 53 miles east of Portland on Interstate 84. During a heavy rain, a rockfall initiated off a natural slope approximately 100 feet above the top of an existing rock cut. The rockfall impacted on an intermediate bench on the slope which had filled with debris, causing a two foot boulder to be kicked out about 40 feet horizontally. The boulder flew across the 30 foot fallout zone at the toe of the slope and into the outside travelling lane. The rock went through the roof of the car fatally injuring two children and seriously injuring a third. The lawsuit which followed was settled out of court.

Just vs. Her Majesty the Queen

The incident for this case took place in 1981 on the Squamish Highway north of Vancouver, British Columbia. Traffic was being held up due to snow clearing efforts following a heavy, wet snowfall. A rock from just beyond the right-of-way was loosened by the prying action of wind on a large tree. The rock bounced down the slope and through the roof of a small car waiting in the traffic delay, fatally injuring one person. The plaintiff held that the highway department should have removed the rock during the course of scaling the right-of-way slopes below. Although the initial case was decided in favour of the highway department, an appeal of the judgement is pending.

As can be seen in the examples, rock slope failures tend to be sudden, unpredictable events the consequences of which can be tragic and involve high liability exposure when the travelling public is involved.

Cost Implications

The potential cost impacts for poorly designed rock cuts are twofold; liability exposure and maintenance. As shown by the case cited above, settlements for personal injury can reach several hundred thousand dollars.

An accident with major environmental impacts could potentially cost many millions of dollars, an event of ever greater probability given the amount of hazardous material being transported on today's highways. The second cost impact is that incurred by the owner on an ongoing basis associated with the maintenance of the transportation facility. Although these costs are not well documented, the cumulative monetary impact of rockfall cleanup, selective stabilization of existing rock slopes and damage to pavement surfaces must be staggering when related to the initial overall cost per mile of the transportation facility. It has been stated that the cost to excavate a large rock slope failure can be ten times that of the original excavation cost (Piteau and Peckover, 1978).

UNCERTAINTIES IN ROCK SLOPE INVESTIGATIONS

In comparison to more traditional studies such as open pit slope design, designers must address three factors unique to highway rock slope investigations:

- Studies for highways must characterize extensive linear corridors (one-dimensional areas).
- Near surface rock encountered for typical roadway cut slope heights is often highly variable due to effects such as stress relief, weathering etc.
- Highway slopes have an indefinite design life during which even a small scale failure can be potentially life-threatening.

In the past twenty years, highway designers have adopted and modified proven rock mechanics practices from other disciplines for application to highway rock slope investigations. These techniques and their limitations are briefly summarized below.

Geologic Structure

Perhaps the most important criteria for a rock slope designer is the relationship between orientation of the natural discontinuities within the rockmass and the orientation of the proposed rock excavation. Structure refers to faults, shears, joints, bedding planes, foliation and the like which are collectively referred to as discontinuities. Geological mapping or core drilling is essential to characterize rock structure which occurs in sets (a family of discontinuities each with a similar orientation) as well as that which occurs as unique features (e.g. faults).

Mapping

The objective of mapping is to determine the dip and dip direction (or dip and strike) of a representative number of discontinuities for each rock cut. The data is usually presented in the form of a stereonet on which analyses can be performed to determine the kinematic feasibility of failure (Hoek and Bray, 1977). Other properties of discontinuities which should be recorded during mapping include length, spacing, surface form, roughness and infilling materials.

For realignment projects in mountainous terrain, it is likely that suitable rock exposure for mapping will be present along the old alignment. For new alignments, natural outcrops may be few in number and, due to a number of factors, of poor quality for mapping. In such cases, statistically relevent sampling of discontinuity orientations may be difficult and it may be necessary to resort to core drilling.

Drilling

Geotechnical drilling for the investigation of highway rock cuts is typically carried out as shown in Figure 2. Spacing between holes is often in the range of 300 to 400 feet with hole depths extending 5 to 10 feet below grade. At critical locations or for deep cuts, those in excess of 50 feet, two holes may be drilled on a section prependicular to the alignment to provide a cross-section to aid in stability analyses.

The objectives of the drilling include:

- o Sampling for rock quality tests.
- o Sampling for strength testing.
- o Logging and index testing to determine overall rock mass quality.
- o Determination of the rock mass structure at depth.

A discussion of methods to determine rock mass structure through core orientation is given by Hoek and Bray (1977). An example of a low cost core orienting technique is the clay imprint method which utilizes an eccentrically weighted barrel in an inclined borehole. The orientation is determined at the beginning of a core run by taking an imprint of the core stub at the bottom of the hole. The success of the method depends not only on obtaining a good impression of the hole bottom but also on the ability to piece the core back together since the reference line cannot be transferred across shear zones or highly broken zones. The orientation of discontinuities such as joints and bedding can be determined with moderate success but the orientation of a unique major feature, such as a fault, cannot usually be determined from core measurements in a single hole. Even if multiple holes are drilled, solution of the "three point problem" requires detailed geological evidence that the

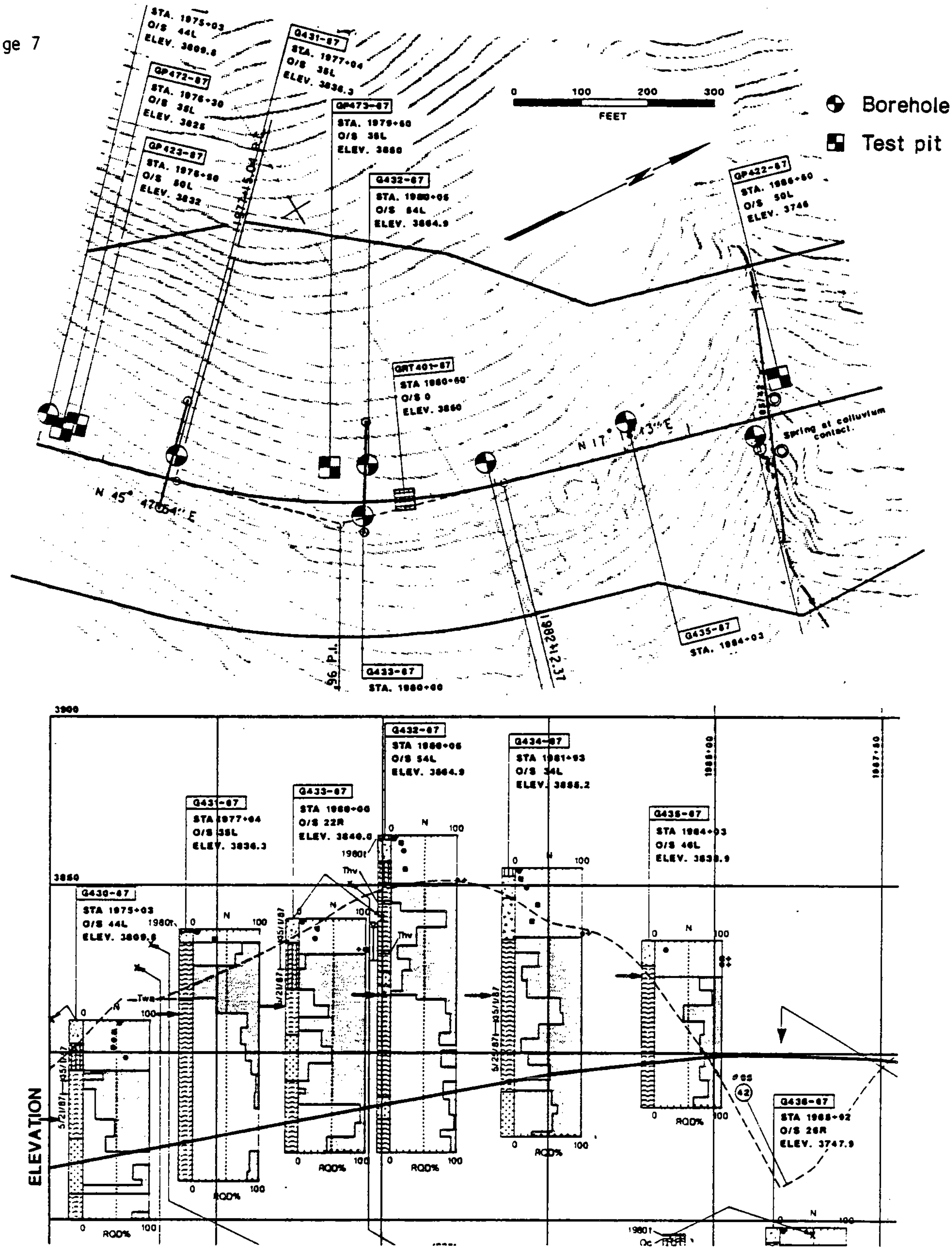


Figure 2
Typical Exploration Plan for Highway Investigations

three points represent the same feature and that the feature is planar. A typical success rate for core orientation under optimum conditions is 50 to 70 percent at a cost surcharge over conventional core drilling of 50 percent.

Geotechnical core drilling suffers many limitations including:

- A serious scale problem extrapolating the engineering properties of core sized samples, obtained from widely spaced boreholes, to the behavior of the overall rock mass.

- Biased structural sampling in that coring does not recover representative numbers of those discontinuities which are oriented sub-parallel to the hole.

- Lack of data on the length of discontinuities which cannot be determined from core data.

- Difficulty in determining the orientation of unique features such as faults.

To summarize, the best that can be hoped for in most evaluations of rock structure for highway slope design is to get a feeling for "the big picture" and to avoid fatal flaws in the design. Due to the limitations of structural mapping and drilling, it is inevitable that some structural unknowns or structural anomalies will either have to be accommodated in the design or be rectified during construction.

Groundwater Measurements

The presence of groundwater within the excavation is probably the most important primary factor in the long term stability of a rock slope. Groundwater in a rock slope can reduce stability by:

- Reducing the frictional strength component of potential failure surfaces (see Eqn 1).

- Increasing the forces tending to cause failure when a tension crack becomes partially or completely filled with water.

- Increasing the rate of slope degradation and erosion.

- Promoting rockfall during periods of freeze-thaw cycles.

For these reasons, the distribution and magnitude of pore pressures in a rock slope is critical to the stability of that slope. To determine and predict these pressures in advance of excavation requires knowlege of the natural pre-existing pore pressures, the overall climatic effects and the permeability characteristics of the rock mass.

A typical highway investigation program generally provides that open standpipes be installed in every second or third borehole along the alignment. A open standpipe can be a misleading indicator of groundwater conditions in that it averages all the pressures intersected by the hole and yields little or no information on whether the hyraulic gradient is upward or downward. In some instances this can be overcome by installing sealed piezometers, preferably two or more in close proximity, so that the direction of gradient can be determined.

The most serious deficiencies in groundwater investigations for roadways include:

- o Lack of data for a sufficient time period to include extreme seasonal and storm effects.
- o Difficuly in estimating the effects of rock permeabilty on the post-excavation pore pressures.

Typically, these deficiencies are accommodated with conservative assumptions for worst-case groundwater conditions and with design provisions for slope drainage.

Shear Strength of Discontinuities

The estimation of shear strength along rock discontinuities is necessary for the stability analysis of slopes or portions of slopes controlled by structural features. Shear strength is comprised of two components, the frictional component and the cohesive component which are related by the familiar expression:

$$\tau = (\sigma - \mu)\tan\phi + c \qquad \dots\dots\dots \ (1)$$

where: (typical units)

τ = shear strength (psf)
σ = normal stress (psf)
μ = water pressure (psf)
c = cohesion (psf)
ϕ = angle of internal friction (dimensionless)

Many highway slope designs evaluate the shear strength of discontinuities based on **qualitative** evaluations of the discontinuities in outcrop and from rock cores, and therefore the shear strength for practical purposes, is based on experience and reasonably conservative values obtained from established design tables.

When the shear strength of the discontinuity becomes critical to the rock slope design, laboratory testing can be conducted on the discontinuity or on it's infilling. Determination of these shear strength parameters through laboratory methods requires four important steps:

- Selection and retrieval of an undisturbed and representative sample containing the discontinuity. This is not a trivial exercise in rock as the drilling or sampling procedure often destroys the weaker samples leading to a non-conservative strength bias for the samples which can actually be tested.

- Testing of the sample(s) utilizing shear box, triaxial or other equipment as appropriate.

- Interpretation of the results to select design strength parameters for the field scale feature. The failure surface for the tested sample may have a size of a few square inches but this must be extrapolated to a potential slope failure surface with a surface area of hundreds or thousands of square feet.

- Prediction of the effects of construction which can often modify the actual strength values which can be mobilized. For example, uncontrolled blasting can destroy delicate infillings or cementation along joints thereby effectively reducing the cohesive component of the shear strength to zero. Similarly, movement along discontinuities during blasting can reduce the angle of internal friction from a peak value to a residual value.

As an alternative to laboratory testing, slope failures of a similar nature can be back analyzed to determine the acting strength parameters at the time of failure (assuming a Factor of Safety of 1.0). If the actual conditions at failure, such as earthquake accelerations, pore pressures etc. can be estimated, this method of strength determination is often superior to laboratory testing.

Whatever method of strength determination is adopted, the application of the results to features with a large areal distribution, such as a roadway alignment, is tenuous with respect to engineering precision.

Rock Mass Strength

In some cases it is necessary to design rock slopes in a rock mass which is highly fractured but lacks structural control for the face inclination. In this circumstance it is necessary to develop a model for the shear strength of the fractured rockmass. A method of accomplishing this is to utilize a strength criterion such as that developed by Hoek and Brown (1980). This criterion incorporates the broad lithogic rock type, the strength of the intact rock and the degree of fracturing in the rock mass to develop a non-linear shear strength equation.

In the typical application of such methods, a general shear strength model is developed. Application of such a model to lengthy alignments will surely result in some zones in which the strength model is inappropriate. Such zones must be recognized during constuction and appropriate design modifications made to suit actual conditions.

UNCERTAINTIES IN ROCK SLOPE ANALYSES

Just as unavoidable deficiences exist in rock slope investigations, so to are there deficiences in the analytical procedures.

Failure Models

In order to analyze rock slopes for stability, a number of models have been developed for which mathematical solutions can be derived. These models can be separated into structurally controlled and non-structurally controlled models. Examples of the former include plane, wedge and toppling failures, while the latter includes circular failure (Figure 3).

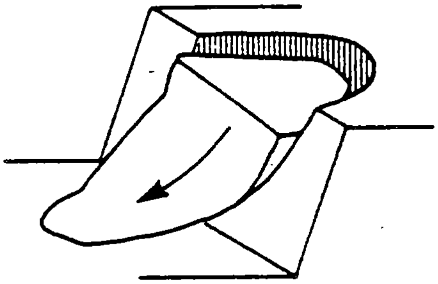
Of these analytical methods, only the wedge analysis takes account of the three-dimensional shape of the failure mass. The other methods assume that the slope is infinitely long so that a two-dimensional analysis is representative. Recent analytical methods have been developed for three-dimensional anaylses, but these are not in widespread use.

Limit Equilibrium Analyses

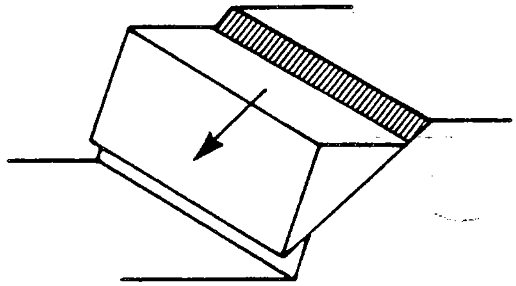
Analytical methods usually employ the limit equilibrium method of analysis which assumes that all resisting forces are fully mobilized at the same time. For progressive type failures, portions of the failure surface can locally become over-stressed and fail, thereby transferring the unbalanced forces to other portions of the failure surface. This process can continue up to the point of failure and clearly the limit equilibrium analysis is imprecise and non-conservative for this mode of failure. Design factors of safety should be sufficiently large to account for these time-dependent modes of failure.

Seismic Considerations

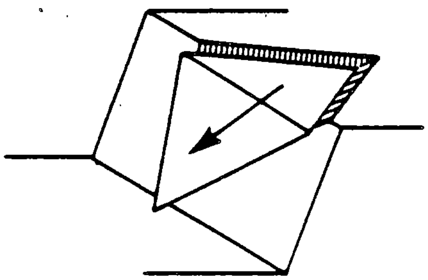
It is often necessary to incorporate seismic accelerations into rock slope design. This is accomplished by applying a horizontal force equivalent to some percentage of the gravitational acceleration acting at the center of gravity of the failure mass. The horizontal force is determined based on the magnitude of the earthquake for the desired return period, the distance from the epicenter and the amplitude of the ground motion. This data is often



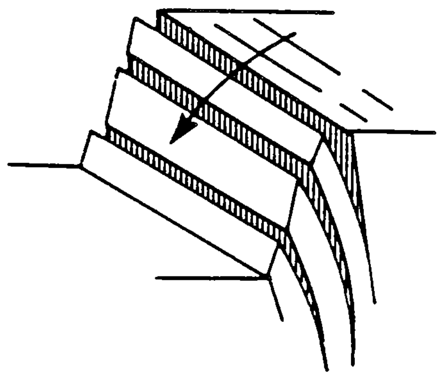
a. Circular failure in overburden soil, waste rock or heavily fractured rock



b. Plane failure



c. Wedge failure



d. Toppling failure

(After Hoek and Bray, 1977)

Figure 3
Types of Slope Failure in Rock

based on extrapolations of historical data rather than actual measurements. Recent experience suggests that pseudo-static analyses have been overly conservative and that further investigation into this field is required.

Rockfall Control

Rockfall control has been designed based primarily upon empirical data collected by Ritchie (1963) in Washington State. His research study consisted of determining the trajectories of bouncing rock for a variety of slope height and slope angle combinations. From these field tests Ritchie developed a set of guidelines which are still in widespread use for the design of rock fallout zones beneath cut slopes (Table on Figure 4). A study of rockfall mitigation in California carried out by Caltrans in 1985 verified the ditch dimensions derived by Ritchie. In the past 20 years, the Washington State Department of Transporation has effectively mitigated rockfall problems on cuts where Ritchie's criteria have been applied.

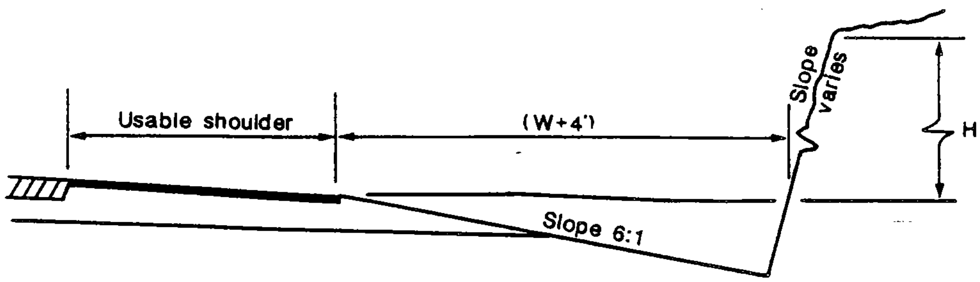
The main limitations of Ritchie's design charts is that they cannot generally be applied to design fallout zones for rockfall originating from mountain slopes with non-uniform inclinations or with multiple material types. In recent years, several workers have attempted to apply analytical methods to predict rockfall trajectorties, notably Piteau and Clayton (1977), Bowen (1987) and others. These methods must still be considered to be in the developmental stage until further field verification is carried out.

MITIGATION OF DESIGN UNCERTAINTIES

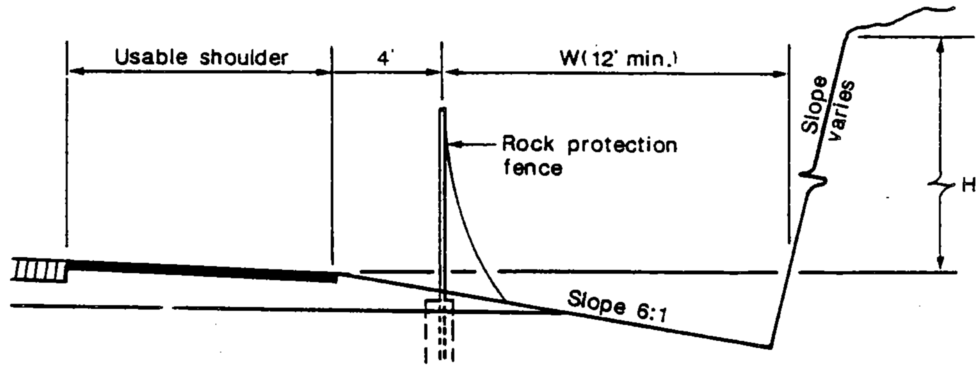
From the preceeding discussion it is evident that most rock slope designs for highways have inherent uncertainties due to deficiencies in both the investigation and analytical processes. In well designed slopes, this uncertainty or design risk is minimized through implementation of selective design and constuction mitigation measures including:

- Variation of slope geometry
- Slope drainage
- Careful excavation procedures
- Construction monitoring
- Remedial stabilization measures

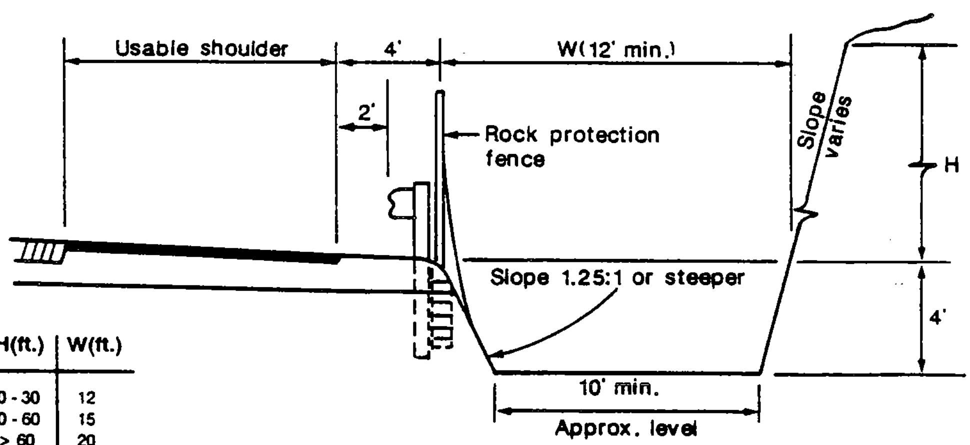
Roadway section in rock cuts – Design A



Stage 1



Stage 2



Rock Sl.	H(ft.)	W(ft.)
Near Vertical	20 - 30	12
	30 - 60	15
	> 60	20
0.25:1 or 0.30:1	20 - 30	12
	30 - 60	15
	60 - 100	20
	> 100	25
0.50:1	20 - 30	12
	30 - 60	15
	60 - 100	20
	> 100	25

Stage 3

Figure 4
Ditch Design

Slope Geometry

Modifications in the geometry of the rock slope can be made to substantially reduce the risk of a rock slope failure. These geometric variables fall into three basic categories.

The first category concerns modification of the horizontal and/or vertical alignment of the highway. This variable is of limited application since under normal design settings the alignment and profile are controlled by design factors other than slope stability requirements.

The second category, and that in which the rock slope designer can have the greatest impact, is the cross-sectional geometry of the rock slope. By simple manipulation of the face inclination in conjunction with site specific knowledge of the structural geology, major improvements can be made in the stability of the slope. This concept contradicts the well established criteria of "template designs" in rock (e.g. 0.25H:1V in granite, 0.5H:1V in shale). For almost all rock slope designs, template designs have no basis in sound rock slope engineering.

A third category of geometric modification is the ditch design. Although the rock slope design process strives to minimize the potential for slope failure, it cannot eliminate it. By providing a ditch width and ditch depth suitable to contain the trajectories of falling rock for the inclination and height of the rock cut, the impact of failure can be minimized. The ditch design criteria in use today was developed primarily by Art Ritchie of the Washington Department of Transportation in the late 1950's (Ritchie, 1963).

Benches on rock slopes, once considered beneficial for rockfall control, are now almost universally regarded as impractical due to construction and maintenance limitations. Due to the lack of accessibility for maintenance equipment, these benches eventually fill with rock debris and actually become hazards in themselves in that they can project rockfall out into the traffic lanes. In a recent rockfall accident in Oregon, a rock originating above a debris filled bench was projected 40 feet horizontally into the path of an oncoming vehicle resulting in the deaths of two children. The preferred alternative to a rock slope bench is to increase the width of the ditch. This location has the advantages of increased storage volume for rockfall debris as well as accessibility for maintenance equipment. The only exception to this is on very high slopes where it is practical to construct an equipment-accessible bench at least 20 feet wide, and preferably greater.

Slope Drainage

Although there are numerous methods to control groundwater in rock slopes, the most economical method is the utilization of horizontal drains drilled into the final slope face at locations of obvious seeps and at regular intervals along the base of the rock slope just above the ditchline (Figure 5). In non-degradable rock these drains can consist simply of uncased drill

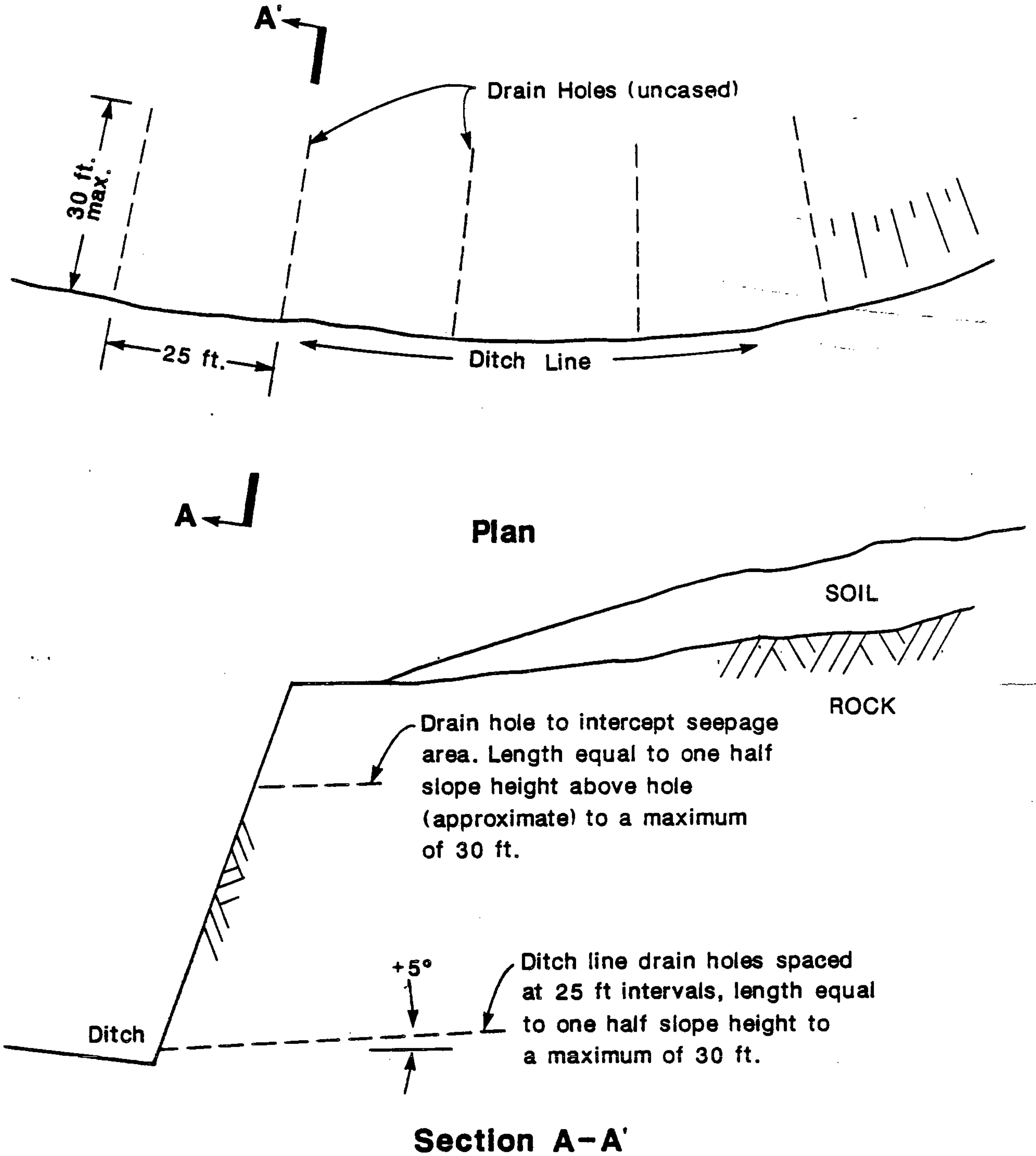


Figure 5
Drain Holes for Rock Slopes

holes with a positive inclination into the slope. If the rock is subject to degradation, some form of perforated pipe is necessary to keep the drainage path open.

Excavation Procedures

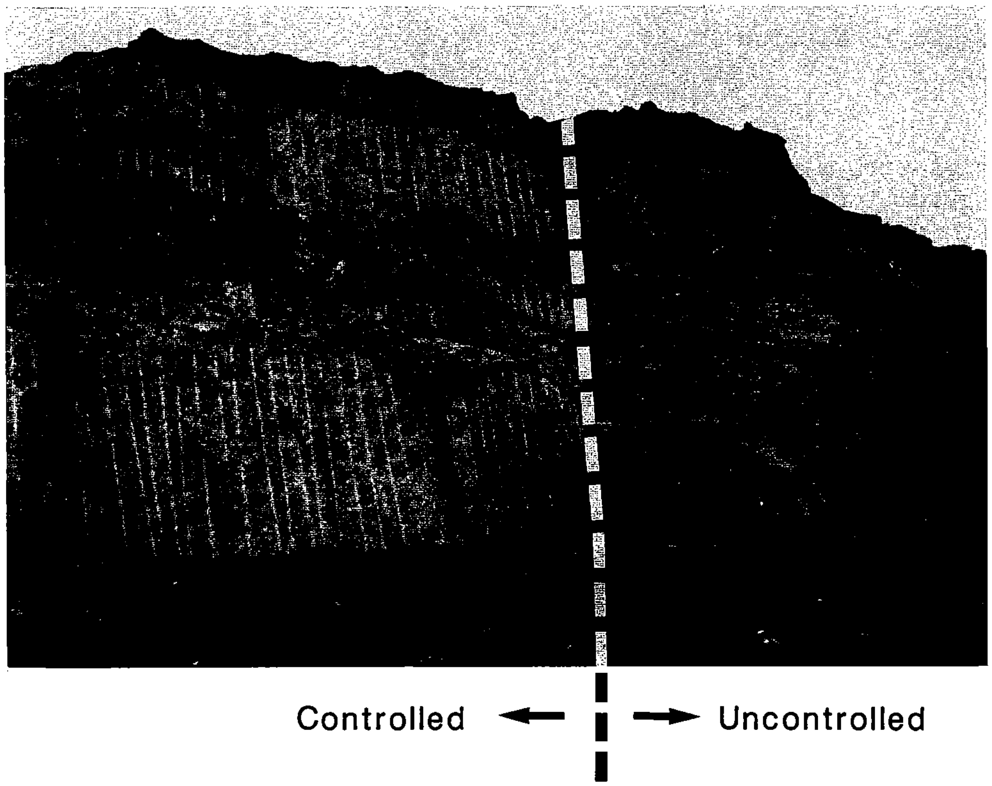
Rock slopes are excavated either by ripping or by drilling and blasting. Either procedure, if not carefully controlled, can result in final slope faces which will produce ongoing maintenance problems due to rockfall.

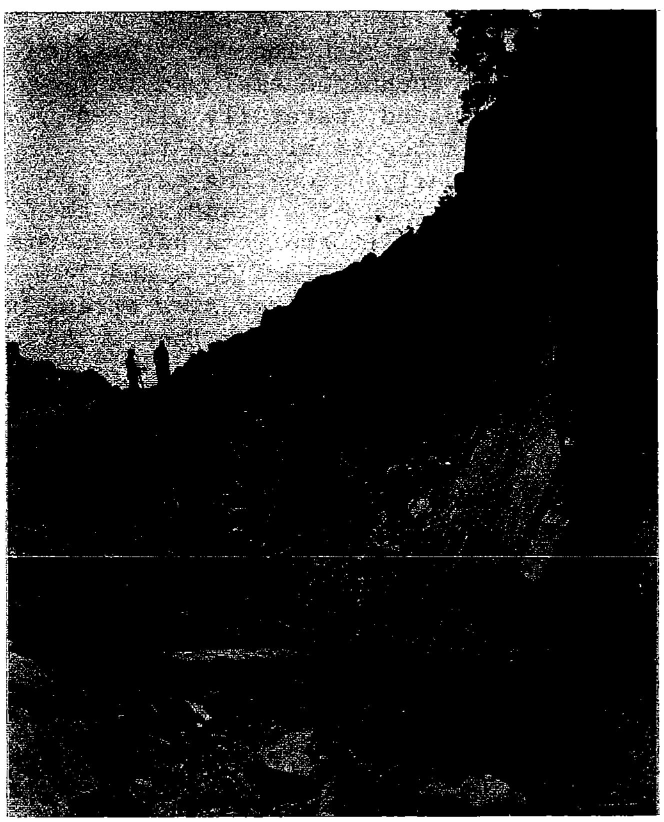
Ripping leads to problems when either the rock properties or the ripping equipment causes the formation of large rock blocks. This commonly occurs where the rock is strong and relatively unfractured but the contractor attempts to rip it with high powered tractors. An irregular final slope face containing partially dislodged, large blocks is the probable result.

When drilling and blasting are employed to excavate the rock, it is essential that the final rock slope not be subjected to significant blast vibrations or gas pressures which weaken the rock mass. Controlled blasting techniques accomplish this through the use of time delays which limit the amount of explosive instantaneously detonated, thereby reducing the vibration levels transmitted through the rock. An important technique employed in controlled blasting is the drilling of a closely spaced, carefully aligned final row of holes inclined at the design angle for the final slope face. This row is loaded very lightly with explosive, usually detonating cord, and fired either before ("pre-split blasting") or after ("cushion blasting") the main production blast. This line drilling tends to split the rock between the final row holes which results in minimal backbreak in the final face and a less disturbed rock mass. This is important to preserve the cohesive strength component along discontinuities and to form a smooth, intact rock face which is less susceptible to degradation and generation of rockfall. Also, by maintaining the integrity of the rock, steeper slopes can be excavated. Depending on rockmass character and particularly for harder rock types, high quality, controlled blasting will usually result in blasthole imprints or "half rounds" left on the face. Figure 6 illustrates a comparison of controlled blasting and conventional blasting within the same rock cut.

Construction Monitoring

The monitoring of major rock slopes by qualified geotechnical engineers or engineering geologists during construction is an effective method of mitigating against design uncertainties. This individual checks that assumptions made for the initial design are verified in the pioneer access construction and in the upper lifts of the cut. He should also detect structural anomalies, such as significant faults, which were not accounted for in the design. It is much easier to implement major design changes during the early excavation sequence than to re-access the top of the cut after it has been completed or after it has failed.





Uncontrolled blasting (circa 1915)

Figure 6
Comparison of Controlled Blasting and Conventional Blasting

Remedial Stabilization Measures

An integral part of construction monitoring is to have a "bag of tricks" available through the contractor for the remedial stabilization of site specific problems. These stabilization measures commonly include shotcrete, tensioned or untensioned steel rockbolts, wire mesh, fences, buttresses etc. Occassionly such measures can be specified in the design of rock cuts but more often they are required as a result of conditions revealed during excavation. For this reason it is advisable that the construction contractor have the equipment and expertise available to implement these techniques on as-required basis.

AESTHETIC ISSUES

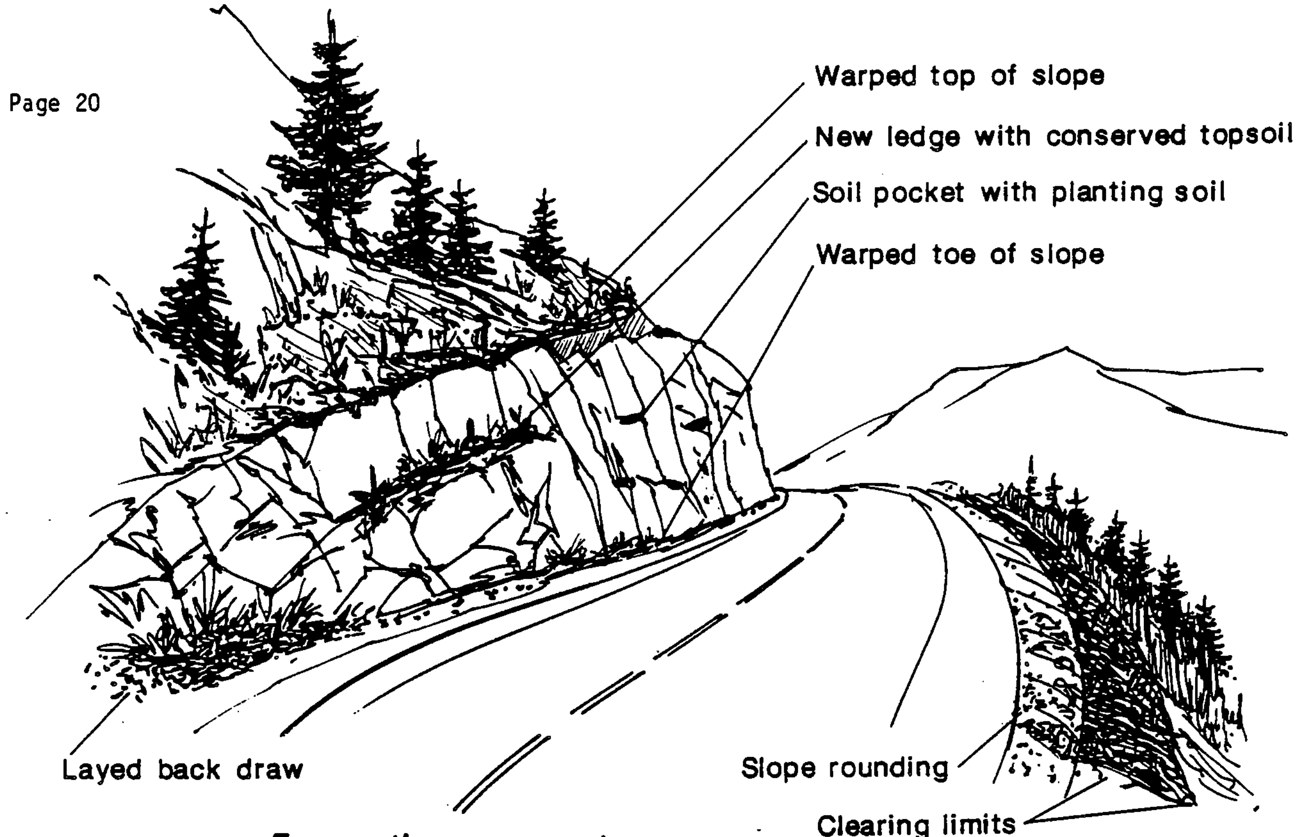
The following discussion deals with those aspects of rock slope design in which aesthetic and engineering objectives are typically in conflict.

Slope Geometry

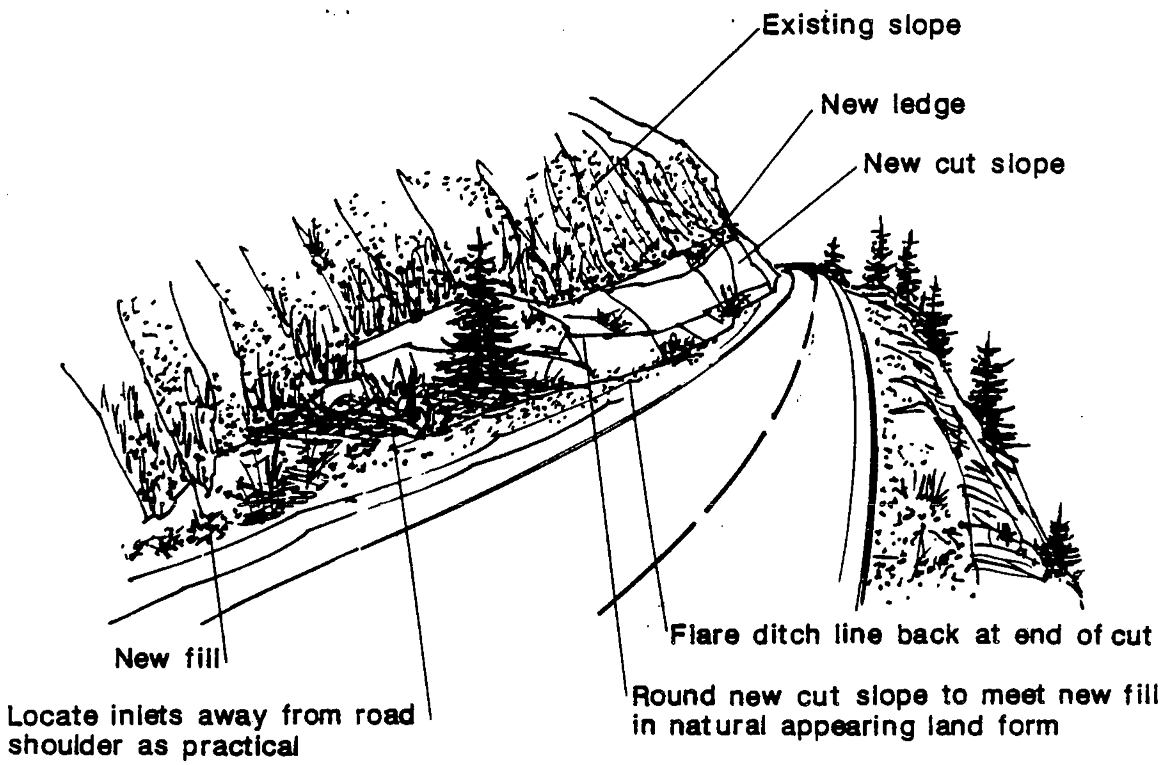
Engineering experience indicates that smooth, uniform, rock faces without benches are optimal for safety from rockfall. A tight slope face minimizes cracks and fissures in which weathering, ice and root wedging can progressively loosen and undermine rock blocks, eventually leading to a rockfall problem. A uniform slope face, free from overhangs, provides less source material for rockfall.

Other desirable aspects of slope geometery, from the engineering viewpoint, include rounded slope crests to transition between intact rock and weathered rock or surficial materials, plan view shapes which are planar or concave and slope inclinations which are the steepest possible, yet compatible with the structural discontinuities of the site-specific rock mass.

Aesthetic guidelines for rock cut slopes attempt to create irregular or "sculptured" faces which are intended to have a natural appearance. The slope face is intended to be warped with non-parallel top and toe cut lines which impose variable inclinations to the cut (Figures 7 and 8). It is sometimes specified that randomly-located ledges and pockets be provided for planting vegetation (Figure 9). These requirements are diametrically opposite to engineering requirements to minimize rockfall potential as discussed above. In addition, they assume that rock can be "cut like butter" to provide randomly located ledges and pockets. Even if they could be physically and economically constructed, there is overwhelming engineering experience with rockfall trajectories which dictates against deliberately building in slope face roughness to accomodate such aesthetic guidelines.



Excavation concepts
(Blasted or excavated)



Transition between cut and fill
or between new cut and existing ground

(Adapted from Reference 6)

Figure 7
Aesthetic Landscape Requirements for Rock Slopes

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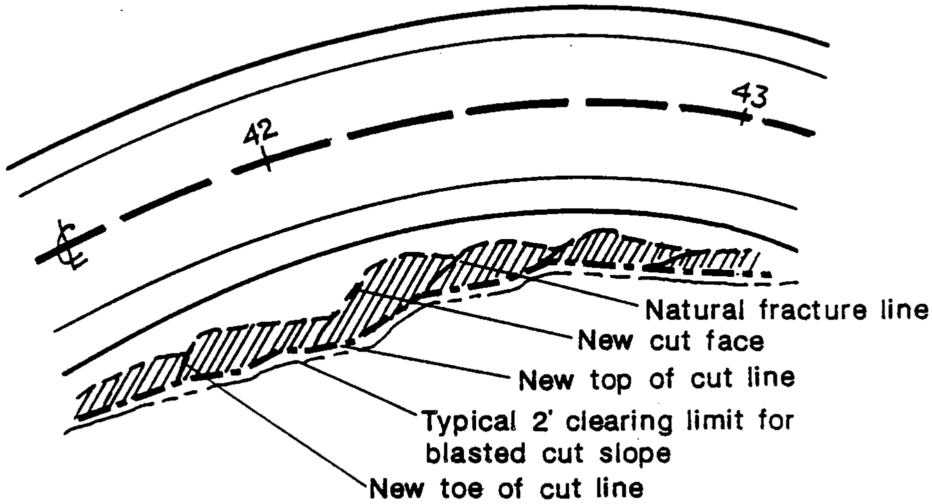
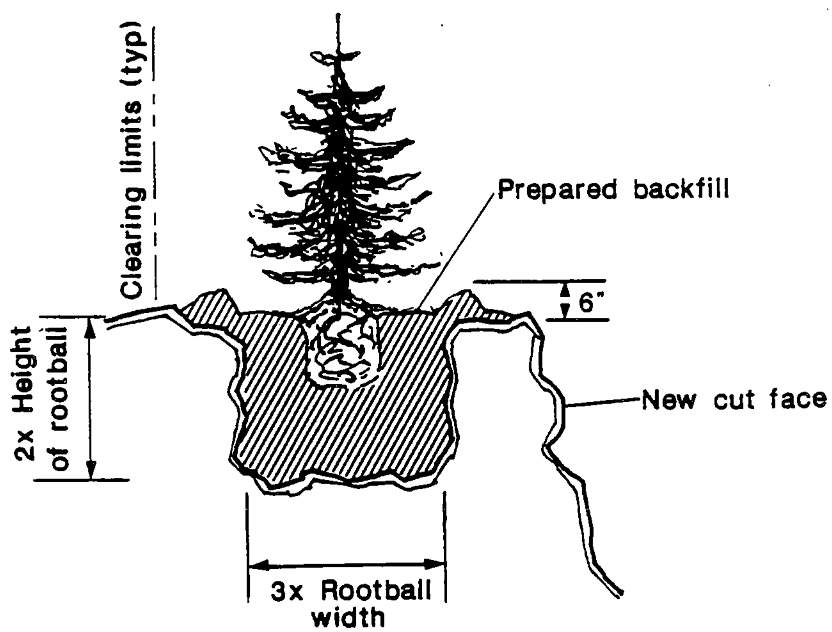
Note:
Warp top & toe of cut slope where indicated in irregular line, not parallel to centerline warp to follow natural fracture lines where possible.



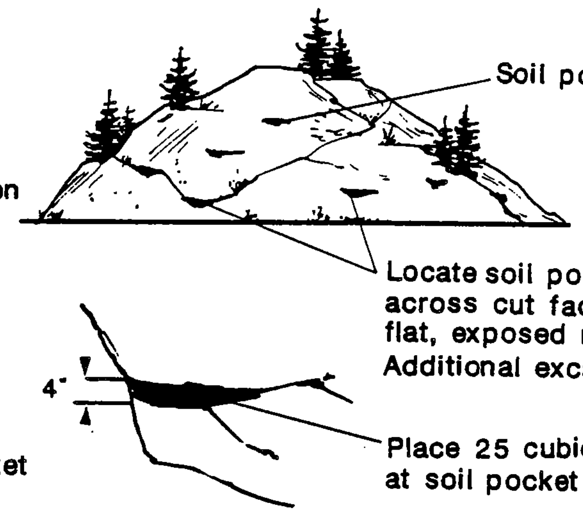
Warping top/toe of cut face (Plan view)



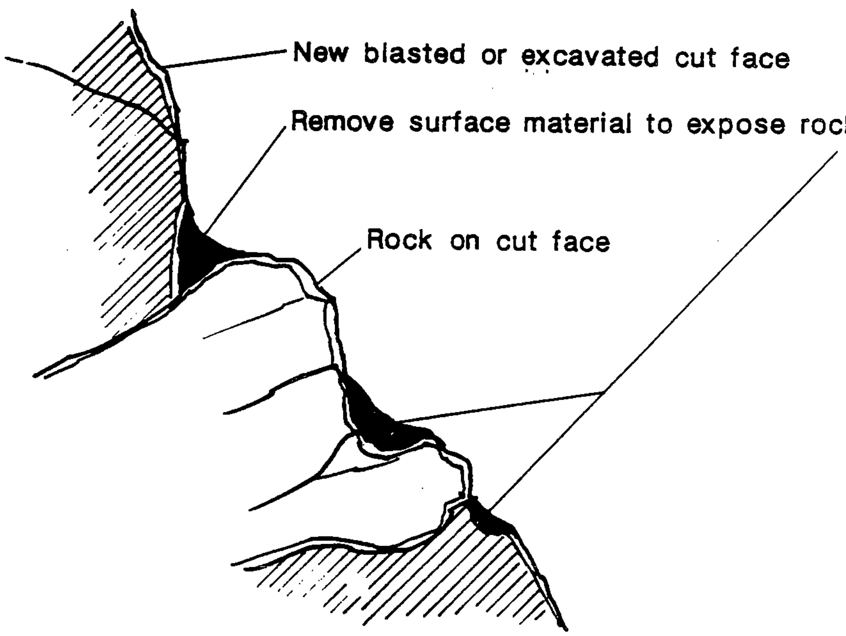
Planting on top of cut slope

(Adapted from Reference 6)

Figure 8
Aesthetic Landscape Details for Rock Cuts



Cut slope soil pocket concept



Highlighting rock outcrop–cut slope

(Adapted from Reference 6)

Figure 9
Aesthetic Landscape Details for Rock Cuts

Controlled Blasting

Controlled blasting refers to all methods which attempt to limit the disturbing effects of the blasting on the rock which will form the permanent rock slope. The associated blast hole imprints are totally objectionable to some landscape architects (Figure 10). To create the "ragged and more natural" cut face required by aesthetic interests, necessitates that uncontrolled blasting be used. This results in not only a ragged face but also in disturbance of the rock some tens of feet into the cut with consequent long term rock fall problems. Much of the rockfall hazard which is present on the highway system today is the direct result of uncontrolled blasting practices prior to the 1960's.

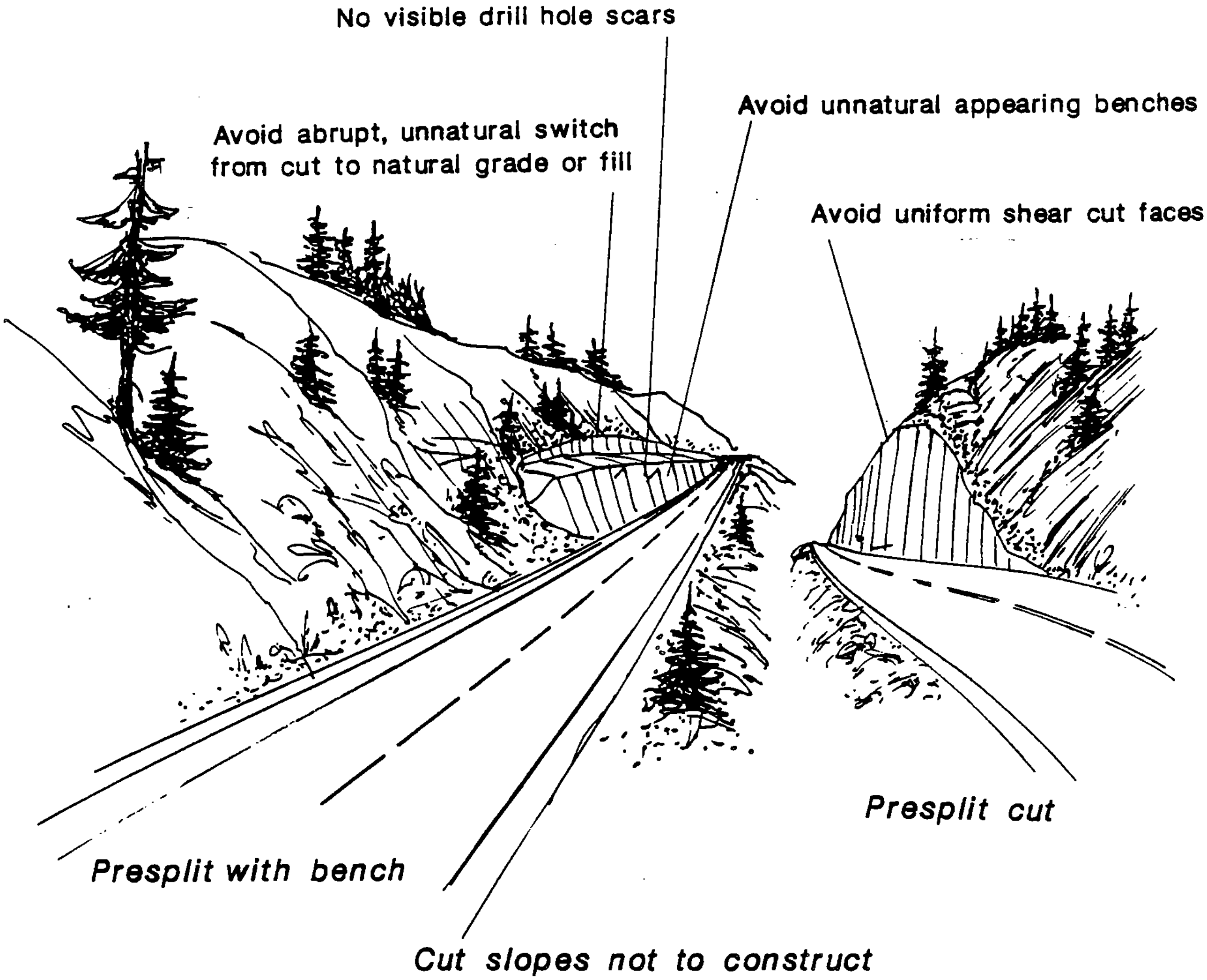
Controlled blasting is considered essential for the excavation of safe rock slopes with minimal costs for scaling, excavation, remedial stabilization and maintenance (Piteau and Peckover, 1978). For rock cuts of limited height with adequate fallout zones and with favorable geologic structure, a few procedures are available to induce limited slope roughness and to minimize blasthole imprints. A slightly wider spacing of holes in the final row can induce some roughness and limit imprints to a certain extent. Additional experimental techniques include control of the direction of detonation within each of the presplit holes and the utilization of special explosives with a higher brissance and higher detonation velocity which can be used to induce slight shattering of the rock immediately around the blast hole which will cause the rock to weather more quickly to remove the hole (H.W. Sheeran, personal communication). <u>Extreme</u> care should be taken to insure that such methods do not compromise the integrity of the rock slope and should only be based on a site specific evaluation and test blasts under the guidance of a blasting expert.

Slope Drainage

Promotion of slope drainage to reduce pore water pressures and minimize ice formation, which can lead to seasonal increases in pressues, is integral to sound cut slope design. Drain holes, often completed with slotted drain pipe, are the most economical method of controlling water pressures in a highway rock cut. Unfortunately, the array of pipes and holes on the slope face can be unsightly and is unacceptable in aesthically sensitive areas. Mitigation can be partially effected by limiting the amount of pipe protruding from the face or by strategically locating the drain pipes in less visible recesses in the face. In non-degrading rock types, less visible, uncased drain holes are preferable.

Shotcrete

Long-term weathering of certain rock types, such as shale, particularly when interbedded with more resistant rock types, can lead to under-cutting and block instability. Shotcrete is an invaluable tool to the design engineer to



(Adapted from Reference 6)

Figure 10
Aesthetic Dislikes for Rock Slopes

prevent degradation through weathering and to thereby preserve the rock mass strength derived from the interlocking of component blocks. This is accomplished with a thin coat, typically 2 to 3 inches, of pneumatically applied concrete, usually referred to as shotcrete. To avoid buildup of water pressures in the rock mass behind the shotcrete, drain pipes through the shotcrete are common.

Aesthetic objections to shotcrete include the unnatural gray concrete color, the mottled appearance associated with leaching and chemical precipitation caused by groundwater, the associated drain pipes, and the texture which is intermediate between that of the rock and a cast-in-place concrete wall.

Many treatments are available to improve the appearance of shotcrete. For slopes excavated to close tolerances, the final shotcrete surface can be hand-trowelled to create a texture indistinguishable from cast-in-place concrete. A more natural texture can be achieved by applying a finish coat of rough exposed aggregate. The color of the shotcrete can be matched to the surrounding rock through the addition of dye, although careful control of mixing is required to avoid color variations. Bleaching of color with time can be minimized with good drainage and with the use of silica-fume additive to the shotcrete mix. As an alternative to dye, the shotcrete can be painted after it is applied. Visible drain pipes through the shotcrete can be eliminated with drainage fabrics installed as vertical strips at regular spacing behind the shotcrete. The fabric drains should be provided with a gravity drain pipe outlet at the base of the cut.

Rock Bolts

Rock bolts are typically utilized on an "as-required" basis during excavation to stabilize local areas on the slope face. This application often precludes a predetermined location for each bolt in the design phase.

The aesthetic objection to rock bolts is the same as that for drain pipes; an unnatural protuberance from the cut face. Where shotcrete is employed, the rock bolts can simply be covered to eliminate visual impact. Alternatively, the bolt can be installed with the top end recessed in a larger diameter hole, tensioned with a temporary extension rod and grouted for the full length. After the grout has set, the extension rod is removed and the recess hole is filled with grout to match the rock color (Figure 11).

Rockfall Catch Ditches

Ditches have been proven effective to control rockfall which cannot be stabilized in-place on the slope. Ditch geometry is integral to its effectiveness as shown in Figure 4. Trapezoidal ditches with steep sideslopes and soft, energy absorbant bases have been found most effective for the retention of severe rockfall. Maintenance requirements dictate that the

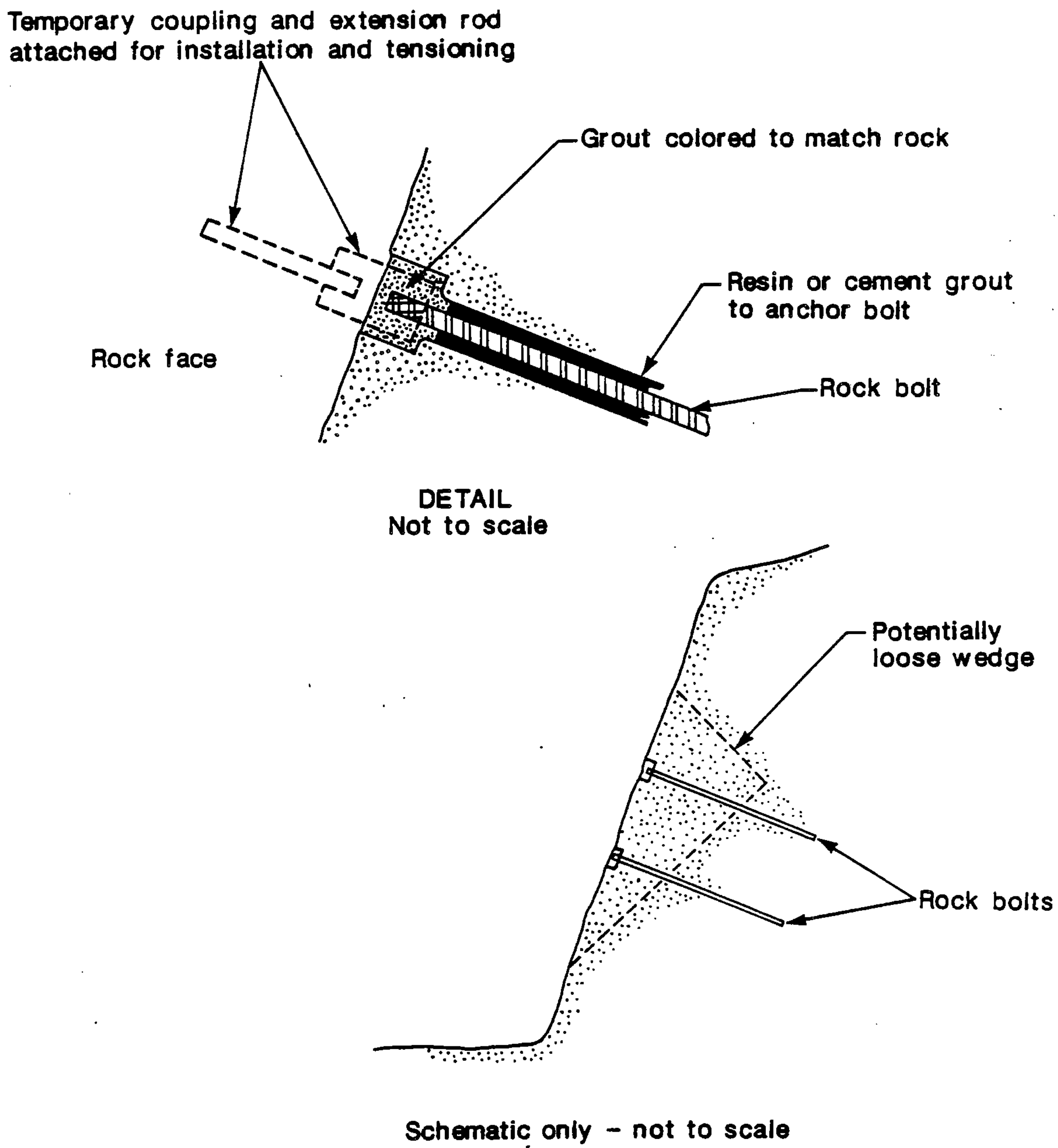


Figure 11
Mitigation of Rock Bolt Visibility

minimum width should accommodate equipment for ditch cleaning (usually about 12 feet). To prevent vehicles from entering the ditch while still retaining the desired rockfall catchment characteristics, the optimum ditch shape is sometimes modified either by increasing the width and flattening the sideslopes or by installing a guard rail. The Washington State Department of Transportation Design Manual (WSDOT, 1984) contains a range of ditch designs to suit the expected amount and size of rockfall (See Stages 1 to 3, Figure 4).

Some aesthetic requirements desire a rounded toe of slope or a warped toe line which results in an uneven ditch width and depth (Figures 7 and 8). These requirements can serve to "kick" falling rock out into the traffic lanes, hamper maintenance efforts and reduce ditch catchment capacity and effectiveness. As shown in Figure 4 the ditch geometry must be designed in accordance with the slope height and inclination of the cut and for this reason there is little flexibility to accomodate aesthetic priorities.

Vegetation

The roots of trees growing on rock cut slopes can lead to rockfall through the long-term wedging action which results from root growth and from the prying action caused by wind loading on large trees (Figure 12). It is common for rock scaling contracts to call for the cutting of all large trees on a rock face up to a distance of 10 feet beyond the slope crest.

Aesthetically motivated slope designs have the opposite objective; often specifying that trees be planted on the slope face in pockets or ledges or along the crest of the slope as shown in Figure 8. This desire can be incorporated to a limited extent if the planting pockets are located near the ditch line so as not to interfere with maintenance activities and at sites where they do not represent a stability concern. Suitable types of vegetation should avoid those which will mature as tall trees with extensive root systems.

Retaining Wall Systems

In weathered or weak rock, retaining systems are often employed to enable steeper cuts to be excavated. An emerging system of in situ reinforcement for permanent support of weak rock and soil incorporates steel pins, shotcrete and drainage pipes installed in lifts as the excavation proceeds. This system, commonly referred to as "soil nailing", offers the potential for significant cost savings over traditional retaining systems, such as cantilever walls or soldier pile-tieback walls, because the support is installed from the top down thereby eliminating the need for temporary cut support.

Aesthetic resistance to this type of wall includes objections to the shotcrete, drains and rock bolts. In addition to the mitigation for these

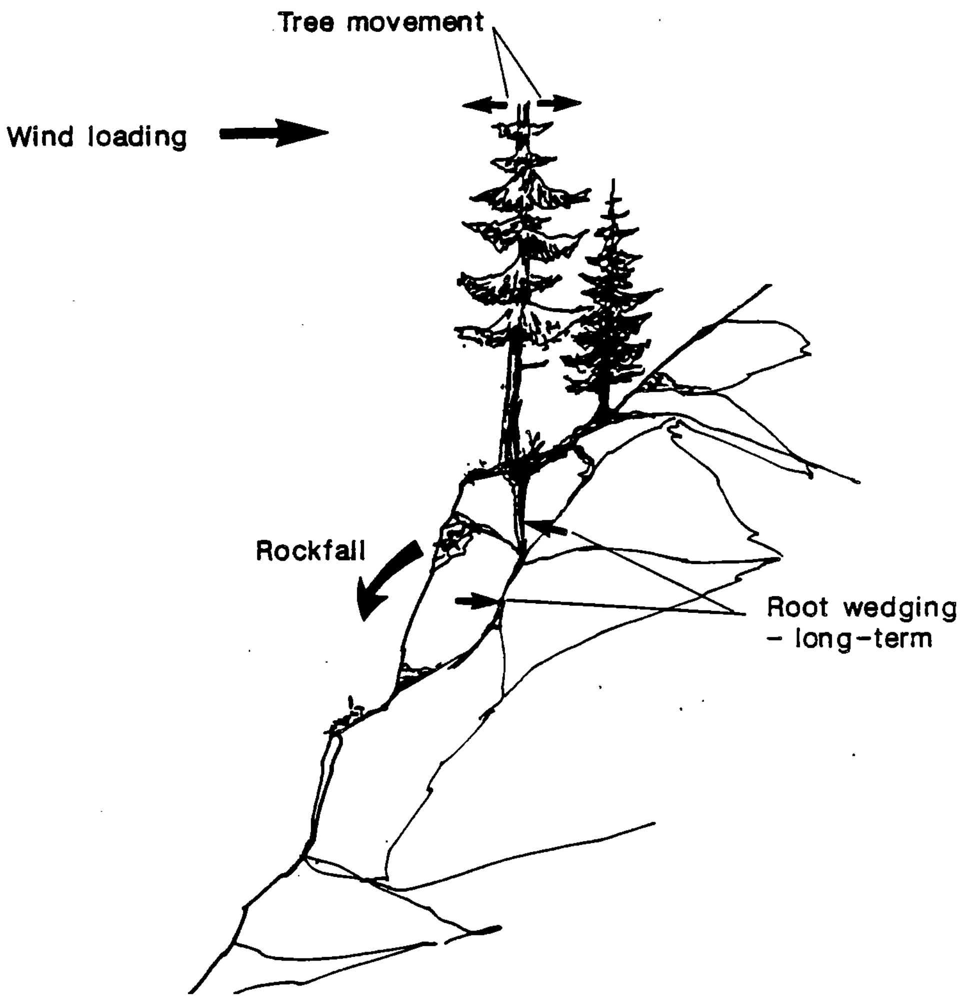


Figure 12
Effects of Vegetation on Rock Slopes

measures previously discussed, a number of facia wall treatments can be utilized to improve the aesthetic appearance. These methods are expensive and can eliminate the cost effectiveness of in situ reinforcement systems.

INTEGRATED DESIGN REQUIREMENTS

From the preceeding discussion, it can be seen that there are divergent issues related to the design and construction of rock slopes. On the one hand, there is a desire on the part of landscape architects to minimize the "monolithic" appearance of the rock cuts, while on the other hand the geotechnical engineer/engineering geologist has the responsiblity to design and construct stable slopes. The design and construction standards of rock slopes today are well established and uniformly accepted within the industry. If these standards are not implemented, the ultimate liability for a rock slope failure will be with the geotechnical engineer/engineering geologist. Slope designs should only incorporate aesthetic requirements after geotechnical approval in the **feasibility, detailed design** and **construction** phases.

Feasibility Stage

To minimize the conflicts between these opposing viewpoints in the later stages of a project design and to avoid having construction contracts that can't be built due to unrealistic aesthetic requirements for rock slopes, early project scoping and communication is essential. This scoping must occur prior to having the aesthetic issues of a project "cast in concrete" (i.e. environmental committments) and should include a feasibility level evaluation of rock slope design issues including a review of:

- The location and height of proposed rock cuts.
- Existing rock exposures (if present) to determine the presence or lack of geologic structure within the rock mass. This review would also entail an evaluation of existing slope failures and their extent.
- Determination of the probable range of groundwater conditions present within the project.

The data derived from this preliminary evaluation process is invaluable in assessing the risks associated with rock slope design and construction for the project. These risks, if communicated early, can prevent conflict at later stages of the design. For instance, if the project consists of only small (less than 15 ft) rock cuts with little or no structural control, the risk and consequnces of slope failure are low and therefore the opportunity to include "aesthetic treatments" to the rock slopes is greater. On the other hand, if the project consists of major rock cuts that will be structurally controlled, the risk and consequences of failure increase dramatically and "aesthetic treatment" of the rock slope may be impossible. The bottom line in these situations is safety of the travelling public and is no longer a visual or aesthetic issue. The early scoping process can identify those projects, or areas within a project, where aesthetic designs could be considered for low risk slopes and also identify the high risk slopes in which aesthetic issues would play a secondary role to rational engineering design and construction.

Detailed Design

Based on this early communication in the feasibility stage between rock slope designer and the landscape architects, critical issues to both parties can be aired and a "design approach" formulated. As the detailed design proceeds and more site specific data is input to the rock slope analyses, there is a need for flexibility in the "design approach" to accomodate the new information. For example, it may be discovered during detailed design that a small rock cut would undercut a large kinematically controlled failure and to stabilize the slope would require an extensive bolting effort which would be aesthetically unpleasing. In this situation, the control to the design is stability and, ultimately, safety, and therefore aesthetic issues must be a secondary criteria.

Another issue for consideration during the detailed design stage is to avoid the contractual inclusion of aesthetic design elements which ultimately cannot be constructed. Design elements such as planting pockets not only are very difficult, if not impossible, to construct, they also detract from the overall long term stability and safety of the slope.

Construction Stage

During construction, aesthetic issues for the project must not be rigid to the exclusion of remedial slope stabilization measures which may be needed as the rock cut is being brought down to grade. To flatly deny the use of shotcrete, rock bolts or other methods of stabilization is very risky since the nature of the slope instability may not be known until it is encountered. Limiting the use of stabilization techniques either compromises the stability and long-term safety of the slope or adds significantly to the construction cost through the use of less cost-effective stabilization alternatives.

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CONSTRUCTION CONSTRAINTS - WETLANDS, RUNOFF, CONTAMINATION

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ABSTRACT

The construction of highways is becoming a more regulated activity as each year passes. The environmental constraints impacting construction can include wetlands, critical habitats, contaminated soils, steep slopes, and stream encroachment.

This paper discusses the impacts of wetlands, naturally acidic soils, high ground water and critical habitats, specific to approximately 3 miles of highway construction and generally as to how to mitigate environmental constraints for successful highway construction.

The 3 mile highway located in southern New Jersey required a fast track design coordinated with multiple environmental permits. The layout of the roadway was shifted to avoid as much environmentally sensitive areas as was possible. However, the design still impacted 40 acres of wetlands in an area with high ground water and natural acidic glauconitic sands. It was necessary to develop a wetland mitigation plan that provided wetland creation and preservation to offset the wetland impacts that the proposed project would cause.

For the creation of the wetland areas, it was necessary to excavate several feet below the existing surface to a depth that intercepts or approaches the water table. In addition to providing adequate moisture conditions, proper substrate conditions were also required depending on the wetland type being created. To determine subsurface conditions, a hydrogeologic investigation was undertaken. The results of this investigation were incorporated into the final wetland creation area designs. From these data, grading elevations and substrate suitability were determined. Grading plans with wetland zone planting areas were developed.

The most critical of the controlling abiotic environmental factors in the formation and maintenance of wetlands is the presence of water. Water must be present in amounts and duration sufficient to form a habitat that supports predominantly hydrophytic vegetation. Several hydrologic regimes exist that support wetland vegetation, and vary from semipermanently flooded areas, to saturated soils with the water table at or very close to the surface for periods long enough to influence the establishment or predominantly wetland vegetation. There was concern expressed over the possibility of exposing acid-forming subsoils that contain sulfides. When exposed to oxygen, the sulfides contained in these subsoils are oxidized, and ultimately form sulfuric acid which can cause the pH to drop below 3.0.

That hydrologic investigation indicated acidic soils and water, and in addition that the soils were too sandy and low in nutrients to support wetland vegetation. A wetlands mitigation plan was designed and implemented to create 30 acres of wetland and preserve 87 acres of wetland. Mitigating measures included neutralization of soils, special plantings and redesign of the drainage system to create appropriate hydrologic conditions.

This paper summarizes the approach to various environmental constraints for highway construction from identification of concerns through mitigation in accordance with applicable regulations. The interaction of vegetation, soils, rock and hydrologic conditions can be managed to provide for environmental protection as well as highway construction.

INTRODUCTION

The construction of highways is becoming a more regulated activity as each year passes. The cause of this is twofold; more concern for natural resources has resulted in promulgation of regulations and heightened awareness of the public; and remaining undeveloped lands and areas around existing roadways often are available for construction because of lack of development caused by environmental constraints of highway proximity. It is now being realized that assessment of environmental constraints should be performed in the study phase and not later in the design phase. The identification of construction constraints prior to design allows for incorporation into the planning stage design parameters which can save money and time.

Federal and state legislation regulates roadway location and construction practices and their impact on the natural environment. Protected environments include habitat for threatened and endangered species; the open waters of streams, lakes, and rivers; coastal areas; soil source aquifers; and wetlands. These, in addition to traditional engineering constraints, such as steep slopes, poor foundation materials and drainage, can greatly influence the final location and design of a roadway. Traditional engineering constraints can also influence the mitigation opportunities available to the roadway designer.

Wetlands use to be defined as swamps, bogs, and during construction wet excavation. Wetlands now are defined as areas periodically or persistently supporting vegetation that is dependent on moist soil or grow in water, having soils which are predominantly undrained and wet over a sufficient period of time to periodically produce conditions which the soils contain no oxygen, and shallow water covers or ground water saturates the areas some time during the growing season and are regulated by Federal and State regulations.

Heightened concerns of contamination is resulting in investigations during the study phase to identify potential areas of concern prior to right of way takings and condemnation procedures. If the property that a highway is going to be built on is part of an industrial facility, an old landfill or even a gasoline station, an investigation prior to design can identify constraints or potential environmental problems. The investigation generally consists of a historical review of properties, Federal and State environmental file reviews and a site reconnaissance to identify if contamination may exist. Attempting to construct a highway in an area of leaking underground tanks will increase cost and timeframes substantially.

High ground water, stream crossings and drainage areas have always been design parameters for highways and mitigating measures were developed through construction. However, through heightened awareness and dwindling natural resources the protection of ground water, stream channels, and drainage areas are becoming more regulated and design parameters are evolving. The approach has become not just how to construct the highway but how in the best interest to the environment can construction proceed.

CASE HISTORY

Throughout the United States there are many highways and roadways which have been designed and then shelved for years. The reasons are often financial but there are many that are on hold because permits are denied, environmental constraints are evident and to get approvals, designs need adjustment.

BACKGROUND

Location and design of a three mile extension of State Highway 18 in Monmouth County, New Jersey began over twenty years ago. Permit approvals were granted only after successful resolution of concerns for impacts to wetlands. In accordance with Council on Environmental Quality regulations (40 CFR 1508.20), proposed mitigation included measures to avoid impacts, measures to minimize impacts, and measures to compensate for impacts by replacing the affected environment.

The four lane roadway is located within 300 feet right-of-way. Opposing travel lanes are separated by a 60 feet grass median. The project area is within the inner coastal plain, an area that was ocean floor approximately one million years ago, and more recently influenced by outwash from continental glaciation. Topography is generally flat with occasional rolling hills. Elevations in the inner coastal plain vary from sea level to 400 feet. Soils are primarily sand, silt, gravel and clay (Robichaud and Buell 1983).

Vegetation communities include the marshes, swamps, and bogs of the lowlands, which are influenced by low relief and high water table; and the mesic uplands, which are influenced by well drained soils. The mesic uplands are overlaid by a belt of soils containing the mineral glauconite, commonly called greensand marl. In places where the mineral is abundant, the soil has a dark green color (Robichaud and Buell 1983). Soil groups with an abundance of glauconite exhibit strong to excessive acidity. When exposed to oxygen, sulfides in glauconitic soils are oxidized and form sulfuric acid. Soil pH may drop as low as 3.0. Soil pH in the project area ranges from approximately 3.5 to 5.5 and is classified by the USDA Soil Conservation Service as excessively acidic. Development of wetland vegetation on these soil is at best difficult and limits opportunities to mitigate for loss of wetlands. Early coordination with regulatory and resource agencies during the planning phase of roadway development allowed for identification of these constraints and development of measure to avoid or remedy them.

THE PROBLEM AND REMEDY

State transportation agencies operate within the direction given by Executive Order 11990, Protection of Wetlands. Federal aid project must comply with Department of Transportation (DOT) Order 5660.1A, Preservation of the Nation's Wetlands which results from this executive order. DOT Order 5660.1A requires that state transportation agencies demonstrate there is no practicable alternative to construction in a wetland and all practicable measures to minimize harm have been included in the project.

Extension of State Highway 18 required the taking of wetlands, making mitigation necessary to gain project approval from regulatory (i.e. US Army Corps of Engineers and US Environmental Protection Agency), resource (i.e US Fish and Wildlife Service), and sponsoring agencies (i.e. Federal Highway Administration).

The hierarchical mitigation policy described by the Council on Environmental Quality prescribed that the first mitigation measure to be used is to avoid impacts to the environment by alternative roadway location. The roadway corridor was realigned, and existing wetlands within the right-of-way not occupied by the roadbed or construction staging areas were preserved. However, it was estimated that 46 acres of wetlands would still be impacted by the project. Placed fill would convert these areas to upland and prevent reestablishment of wetland vegetation.

Measures to minimize impacts were accomplished by the following. Alterations to wetland hydrology within the project area were minimized by preserving natural drainageways with culverts and pipes. Duration and frequency of flooding by surface or ground water is the most important abiotic factor influencing wetland vegetation. Impacts to wetlands were further minimized by reducing the originally proposed roadway median width from 120 to 60 feet, development of a comprehensive soil erosion and sediment control plan, and application of best management practices for road construction. A unique stand of Atlantic white cedar (Chamaecyparis thyoides) adjacent to the project area was protected by the presence of a qualified forester during the early phase of road construction. Approximately 87 acres of wetlands were preserved, both within and outside the right-of-way.

Finally, measure to compensate for impacts by replacing the affected environment were accomplished by creating 40 acres of wetlands in areas that were mesic uplands. The process of creating wetlands is the focus of this paper and is described below.

The initial step in the development of the Route 18 roadway wetlands impact mitigation plan was to document existing conditions. This documentation began with a review of the U.S. Fish and Wildlife Service (FWS) National Wetlands Inventory Maps for the area and interpretation of available aerial photography. Field investigations, including identification of hydrophytic species and hydric soils, were conducted to confirm the regional FWS assessment.

The second step in the development of the Route 18 mitigation plan was to refine the wetlands impacts predicted. Refinement consisted of field verification of the types and exact locations of wetlands within the proposed right-of-way. This field verification effort was based upon identification of hydrophytic indicator species and a survey of areas within the right-of-way where these species dominate.

Based upon the results of the wetlands survey of existing conditions, the total acreage of wetlands within the right-of-way and the total acreage of wetlands to be filled was calculated. The calculated acreage of wetlands to be filled was refined using engineering judgement regarding the proposed limits of roadway construction. The design of Route 18 roadway was intended to minimize indirect adverse impacts to wetlands beyond the limits of construction. Wetlands impact reported in the mitigation plan included only areas within the right-of-way.

In order to assess the impact that filling the wetlands would have on the environment of the study area, a functional assessment of wetland values was performed. The functional value of wetlands within the project area were all high in terms of ground water recharge, flood storm water storage, sediment and nutrient trapping and wildlife habitat. These wetland areas have generally a low to moderate functional ratings for fishery habitat, food chain support and active recreation.

Several wetland evaluation tools have proved effective in documenting functions and values of wetlands. Wetland functions and values include ground water recharge and discharge, flood storage and desynchronization, shoreline anchoring and dissipation of erosive forces, sediment trapping, nutrient retention and removal, food chain support, habitat for fish and wildlife, and active and passive recreation. Regulatory and resource agencies often require that such evaluations be made to assure the quantity and quality of created wetlands adequately replace those functions and values impacted by construction.

The wetland functions and values described above may be evaluated by the Federal Highway Administration Assessment Method (a.k.a. Adamus Method) first described in 1983 and recently revised and renamed by the US Army Corps of Engineers as the Wetland Evaluation Technique (Adamus, Clairain, Smith and Young 1987). The technique measures predictors of physical, chemical, and biological characteristics of a wetland. It describes wetland function and values by evaluating their social significance, effectiveness in performing a function, and opportunity to perform that function. For example, communities prone to flooding would likely rate the social significance of a wetland that moderates flood flows as high. The effectiveness of that wetland to perform that function is dependent on its location within a drainage. Wetlands positioned near the mouth of a stream are more likely to moderate flood flows effectively that wetlands positioned at the headwaters. Finally, the opportunity to perform that function is dependent on frequency of flood events and increased runoff caused by upstream urbanization.

Successful creation of wetland areas required drainage plans which allow the soils to receive sufficient water to support hydric soils and vegetation. The drainage characteristics of the proposed wetlands creation areas is a function of soils and local topography.

SOILS AND VEGETATION

The soils within the project area range from poorly drained to excessively well drained. Keyport and Lakehurst sands are well drained and will generally not allow sufficient water retention for successful wetlands creation. In order to overcome this potential problem, it was necessary to prepare wetlands creations area in these soils groups to increase water retention. Highly organic topsoil and proper drainage was used to increase fertility and water retention sufficiently to support hydrophytic plants.

The Atision and Colemantown soils are classified as hydric or excessively wet. Hydric soils develop under saturated conditions and soil pores in the root zone are permanently or periodically filled with water. These soils are found in several ground water discharge areas within the project corridor. These soils are generally capable of supporting wetlands vegetations without soil preparation provided sufficient nutrients are available. However, alteration of existing contours was of concern in that excessive water accumulation in these areas might occur altering the existing vegetation and posing problems for proper roadway drainage and possible flooding of adjacent properties. In order to reduce the possibility of these problems, wetlands creation areas which were planned for hydric soils was included in the roadway drainage system. A detailed drainage plan for the major wetlands creation areas was developed to handle excess storm water. In several locations, the excessively well drained wetlands creation areas and the poorly drained hydric soils group were adjacent to one another. At these locations, connections were made to allow the created wetlands areas to function as a single hydrologic unit. The overall goal of the planting in the wetland creation areas was to establish proper conditions which would allow characteristic wetlands vegetation and hydric soils conditions to establish and perpetuate themselves. The success of this creation strategy depended upon proper site preparation and selection of suitable planting material.

Prior to final design of planting or site preparation work, each of the individual creation parcels was inspected by a qualified plant ecologist to determine the exact needs of the site. Based upon this inspection, grading and planting plans were developed. The initial phase of the planting scheme involved removal of existing vegetation, excavation and grading. These activities were be coordinated to the maximum extent possible with roadway construction activities.

Soil preparation involved final grading and placement of organic soils and topsoils. A portion of the organic soil necessary for preparation of the wetlands creation sites was available from within the project limits. The chemical properties of the existing on site soils presented additional constraints for wetlands vegetation. The soils groups within the project area exhibited strongly to excessively acidic soil reaction characteristics (SCS, 1978). The natural soil reaction pH of several of the soil groups in the project area ranged from approximately pH 3.5 to 5.5. These pH levels were low enough to classify the soils as excessively acid according to USDA - Soil Conservation Services (SCS, 1978). Techniques for minimizing the adverse impacts resulting from construction activities in highly acidic soils were developed.

Once grading and soils preparation was complete, planting proceeded. In general only, indigenous species were utilized and planting densities approximated those in existing wetlands. Wherever possible, clumps of existing trees and shrubs were salvaged and transplanted from within the limits of construction. However, it was necessary to supplement available planting material with nursery stock and also necessary to attempt to stage the planting to allow some on site propagation and replanting of desirable species.

The planting scheme involved four zones - Upland Buffer; Forested Wetland: Scrub Shrub and Flooded Field. The material to be planted in each zone were as follows:

Upland Buffer

This area was intended to provide a transition between uplands, roadway shoulders and wetlands creation areas. Roadway planting followed standard specifications using native grass/wildflower mix including panic grass, redtop grass, little blue stem and reed canary grass. In lower and moist portions of the buffer zone, sedges, blackberries, high bush blueberry and swamp rose were be introduced along with scattered transitional shrubs such as spice bush, coast pepperbush and aider. The upland buffer zone is of limited value to wildlife due to proximity to the roadway and low spatial diversity. The planting scheme attempted to compensate for this by introducing a variety of plants with wildlife food value.

Forested Wetland

Limited areas of forested wetland were planned at five locations. These forested wetlands provided varied habitats with vertical spatial diversity in order to accommodate local wildlife species displaced by the roadway. Full development of these areas will require upwards of 10 years and the goal of the initial planting is to establish proper conditions for such communities. Plantings in the forested wetlands zones included Red Maple, Cottonwood and Gum. These species will eventually make up the forest overstory; most were commercially available and have value as wildlife food. These trees were planted in clumps throughout the forested wetland zone. Planting densities depended upon local conditions and the vegetation of adjacent existing areas. Ground cover of ferns, natural wildflowers and wetlands grasses will be allowed to establish themselves throughout the zone.

Scrub Shrub

The largest percentage of the creation areas was planted with wetlands shrub species. These areas represent an intermediate level of plant succession and offer the advantages of relatively high species diversity, are quickly established and tolerate a wide range of soil saturation. One drawback to this creation group is that many of the necessary plant species were not commercially available. Planting in the scrub shrub zone also included swamp dogwood, swamp azalea, swamp rose, coast pepperbush and rushes. Low growing herbs such as orchids and milkworts as well as flowers like Turk's cap lily and ferns were established or introduced into this area.

Flooded Field

The flooded field zone represented a vegetation community similar to the scrub shrub group but included rooted herbaceous hydrophytes capable of growth in periodic flooded areas and conditions of long term soil saturation. This vegetation group represents a mix of emergent wetlands and scrub-shrub wetlands found within the study area. Scrub shrub zone plants were introduced to the edges of the flooded field areas and the demarcation between these two zones is not rigid. However in the center of the flooded field, large stands of burreed-spikerushes were established. The planting scheme concentrated on plants species with value as wildlife food.

In order to evaluate the success of the wetlands mitigation plan, a preliminary assessment of the value of the wetlands creation areas was performed. Environmental analysis of the State Highway 18 extension project made use of NWI maps, SCS soil surveys, and field study. The Federal Highway Administration Assessment Method was applied and allowed for evaluation of both filled wetlands and the created wetland proposed as mitigation. Wetlands within the project area were rated high for ground water recharge, flood storage and desynchronization, sediment trapping, nutrient retention, and wildlife habitat. The functions and values of fishery habitat, food chain support, and active recreation were rated low to moderate.

Created wetlands were designed to replace the functions and values described above to replicate the native vegetation where possible. Soil properties and ground water elevations at proposed locations of created wetlands were intensively monitored by borings and wells to better understand physical and chemical constraints to mitigation. Created wetlands were constructed by excavating below existing grade to an elevation that approached or intercepted the high ground water table. Subsoils within these sites were analyzed and found in some cases to be unable to support wetland vegetation. This because of the predominance of sands with little organic matter and low concentrations of nutrients; and the presence of glauconite. An overlay of both mineral and organic soil provided a physical barrier to glauconite subsoils, improved water retention, provided some nutrients, and offered a seed source for development of wetland vegetation.

The FHWA technique recognizes that perfect replacement of functional values is seldom feasible and exact quantification of projected wetlands values is not possible. The results of functional assessment of the Route 18 wetlands creation areas reflect these limitations.

A permit to conduct this activity was obtained from the Army Corps of Engineers by the New Jersey Department of Transportation. To obtain this permit, it was necessary to develop a wetland mitigation plan that provided wetland creation and preservation to offset the wetland impacts that the proposed project would cause. The mitigation plan included a discussion as to the feasibility or lack thereof of hydraulically connecting mitigation areas restorative measures to be taken if predicted ground water elevations are inaccurate and do not support the created wetland areas, documentation regarding how the 87 acres of preserved wetlands are to be protected, provisions for interconnected borrow ponds to enhance the habitat value for amphibians and reptiles, and provide irregularly shaped ponds 3 to 5 feet in depth covering between 20% and 40% of the available land areas surrounded by low-elevation non-continuous berms, and with shallow radiating collection channels.

Overview of Final Wetland Mitigation Plan

For the creation of the wetland areas, it was necessary to excavate several feet below the existing surface to a depth that intercepts or approaches the water table. In addition to providing adequate moisture conditions, proper substrate conditions were required depending on the wetland type being created. To determine subsurface conditions, a hydrogeologic investigation was undertaken. The results of this investigation were incorporated into the final wetland creation area designs. From these data, grading elevations and substrate suitability were determined. Grading plans with wetland zone planting areas were developed.

The key to a successful wetland mitigation plan rests in creating the proper habitat that will support the desired hydric vegetation. To achieve this primary objective, data on the controlling environmental factors were identified and incorporated in the creation of these wetland habitats.

The most critical of the controlling abiotic environmental factors in the formation and maintenance of wetlands is the presence of water. Water must be present in amounts and duration sufficient to form a habitat that supports predominantly hydrophytic vegetation. Several hydrologic regimes exist that support wetland vegetation, and vary from semipermanently flooded areas, to saturated soils with the water table at or very close to the surface for periods long enough to influence the establishment of predominantly wetland vegetation.

Another abiotic factor associated with wetland conditions is the formation of hydric soils. Hydric soils are separated into two major categories based on their constituents: (1) Organic soils and (2) Mineral soils. Organic soils contain a high percentage (>20%) of organic matter throughout the soil profile and are generally formed in water-logged conditions where aerobic decomposition of organic matter is impeded by anaerobic conditions associated with prolonged flooding. The accumulation of organic matter, particularly plant material, is thus encouraged under these conditions. Ponds, shallow lake shores, and wet depressions are examples of areas in which organic soils are typically formed.

Hydric mineral soils form in situations where the hydrologic regime consists of temporarily flooded conditions and saturated soils. Small amounts of organic matter can accumulate under these conditions, but not enough to classify the soil as organic. Hydric mineral soils commonly occur where ground water is close to the surface, in low permeability soils that form perched water conditions and have the tendency to hold water (clays), and areas with low lying topographic positions.

HYDROGEOLOGIC INVESTIGATION

A hydrogeologic investigation was undertaken to study and identify subsurface conditions in order to create habitat conditions with hydrologic and substrate characteristics necessary to support the desired wetland creation types.

There had been concern expressed over the possibility of exposing acid-forming subsoils that contain sulfides. When exposed to oxygen, the sulfides contained in these subsoils are oxidized, and ultimately form sulfuric acid which can cause the pH to drop below 3.0. To clarify this condition an additional 13 soil samples were analyzed for potential acidity. The test oxidizes the sulfides present in a sample into sulfates. Then the sulfate composition is determined.

Ground water monitoring wells were installed to obtain the following data:

1) Depth to ground water. This data was used to determine the finished grades of the wetland creation areas. Ground water level readings were being taken about twice a month to accurately determine ground water elevations and to determine seasonal water table variations.

2) Ground water pH monitoring. Concern had been exhibited over the possibility of releasing acid ground water into the surface water systems and exposing acid producing geologic deposits. Therefore, pH readings were taken along with ground water level readings to obtain the data base needed to access the possibility of this occurrence.

In general the upper several feet of material consists of tan to brown sands or gray silty clays. These upper deposits are underlain with beds of silty clays. These upper deposits are underlain with beds of sands and clays which consist of varying mixtures of quartz and glauconite. In most cases the mitigation areas are underlain with a subsoil that consists of glauconite sands, commonly called greensand marl. The soil has a dark green color and exhibit strong to excessive acidity.

Chemical analysis was performed on subsoil samples that were at the approximate elevations of the anticipated wetland creation grades. Subsoil chemical conditions varied considerably for most of the parameters tested. The pH values ranged from 3.5 to 5.0. Twenty-nine out of 36 samples had pH values from 3.5 to 4.0.

Percentage base saturation was determined for potassium, magnesium, and calcium and ranged as follows: potassium, 0.6% to 5.7%; magnesium, 1.2% to 27.9%; and calcium, 1.6% to 67.0%. The data showed that a few steps should be taken to prepare a substrate that will adequately support plant growth. First, with regard to texture, the existing subsoils were generally too sandy. Second, the pH was less than or equal to 5 at most locations and below 4.0 at several locations. Third, organic matter was absent. Finally, nutrient availability was low due to low pH.

To remedy the above short comings, two measures were taken. First, the standards for establishing vegetation on soils with a pH below 4.0 or containing iron sulfide, dictate that the acid soils are covered with a minimum of 12 inches of soil having a pH of 5.0 or greater before seedbed preparation. This prevents acid soils from lowering the surface water pH, and provides a viable substrate for plants to grow. Testing for pH and potential acidity was conducted during construction, to determine if conditions warranting over excavation and placement of the soil lift exist. The Soil Conservation Service has determined that it is less expensive and more effective to provide this soil "lift" than it is to treat the acid soils with lime.

The second strategy was to cover the mitigation areas with 8 inches of topsoil. If available, an organic topsoil (wetland topsoil) derived from the wet excavation of impacted wetlands was used in those portions of the wetland creation areas that are designed to be semipermanently or permanently flooded. Organic soils develop naturally under these similar conditions.

The remaining portions of the wetland creation areas was covered with a mineral topsoil. Palustrine scrub-shrub and forested wetlands typically inhabit mineral hydric soils. If sufficient organic soil is not available, the mineral topsoil will also be suitable in those areas described above.

Ground water depths range from about one foot to over fifteen feet below the existing ground surface in most of the wetland creation areas. In wetland creation areas situated in existing uplands it was necessary to excavate down below the existing surface to the water table. In wetland creation areas situated adjacent to existing wetlands it was necessary to excavate the wetland creation areas down to the elevations of the adjacent existing wetlands.

Wetland areas which were hydraulically designed to have specific maximum water level (surface elevation) that will maintain specific wetland vegetation habitats required careful study of the fluctuating water table. The maximum design water levels were determined by the high ground water level of the spring season. When the water levels reach the maximum desired level, water will overflow into adjacent drainage system. Some highway drainage crossings were removed or modified to facilitate the wetland creation design.

The upper boundary wetland limits was based on the following:

1) Once the wetland areas develop, the ground water is expected to slightly conform to the slopes of the wetland creation areas, therefore causing the water table to be several inches higher than in the open water areas.

2) The soil several inches above the water table will be saturated by capillary action.

3) Runoff and ground water movement down the slope will tend to make the bottom of the slope wet.

As required by the permit, the feasibility of hydraulically connecting four areas was explored. In general, the four areas each contain a pond. The ponds are designed to have the same maximum water elevation. The elevation was selected because this was the highest ground water level measured for the four areas, and should control the elevations of the other ponds when connected with equalizer pipes. Equalizer pipes interconnect the ponds with the same invert elevation. The equalizer pipes are set one foot from the bottom of the pond to avoid clogging by siltation. In addition sediment traps were dug at the pipe openings. These pipes are submerged below the pond surfaces to maintain underwater corridors between the four ponds.

A typical scenario of the hydraulics of these four areas, will serve to explain how they are designed to function. Ground water and surface water will feed into the areas during a storm. As the water accumulates to the maximum elevation it begins to overflow through the outlets to the other areas, the primary outlet and equalizer pipes are sized to accommodate a 100-year storm, but if rainfall is intense enough the water level may temporarily exceed the overflow. This will cause the areas to flood. When the rain stops, the water will equalize. Ground water inflow will help maintain this level.

SUMMARY

The awareness of environmental constraints from the planning stage through construction allows for timely and cost effective projects that are not halted due to regulatory compliance.

The Rt. 18 project was halted for years but once attention was given to the controlling environmental conditions the project was final designed and constructed in less than three years.

Both construction constraints and environmental requirements limit options for both roadway location, design, and methods of construction. Constraints, such as naturally occurring acidic soils, also limit mitigation opportunities or require use of special measures. However, not all construction constraints need be looked upon as liabilities. Requirements to disposal of roadway runoff and provisions for natural drainage ways may provide mitigation opportunities by offering sources of water for creation of wetlands.

This paper has described the ability of the roadway designer to take advantage of roadway runoff and to deal with natural contaminant (i.e. glauconite) as part of a wetland mitigation strategy. This strategy includes:

- early coordination with regulatory and resource agencies,
- generation of field data to accurately describe constraints and their magnitude,
- use of assessment and evaluation tools acceptable to the regulatory community, and
- development of a comprehensive mitigation design.

Our experience suggests such a strategy can expedite project approvals and better assure completion of a project within a reasonable time.

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URBAN ENGINEERING AND THE NEW TECHNOLOGIES

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ABSTRACT

In the field of urban construction, engineers are being increasingly forced to resolve difficult problems in situ, and in a timely, cost effective manner. Such problems have been addressed out of necessity in densely populated areas in other continents for many years, and as a result, there is available a wide range of effective and sophisticated techniques from European and Japanese sources. The paper describes salient details of some of these "New Technologies", as applied to minimize the environmental impact of major urban projects involving tunnelling, retaining walls and direct structural support. As the trend towards redevelopment of our major cities gathers momentum, these techniques should have a major impact on all aspects of project performance.

1. BACKGROUND

The contracting skills of an engineering community develop in response to the times and the environment. Many of our largest civil engineering firms mastered the practicalities and controls of massive earth-moving schemes during the heydays of dam and highway construction. More recently, structural engineers have responded to the challenges of oil exploitation: the colossal platforms, both on- and off-shore, in the world's less hospitable regions are fitting testimony. At different times, the innovative talents of bridge engineers, railroad builders and hydraulic specialists, for example, have all been in particular demand.

One of the major themes in world construction today is infrastructure development, redevelopment and upgrading. There has been especially intense activity in transport and sewage projects in both developed and developing countries: Cairo's current resewerage scheme, for example, involving many miles of bored tunnels, cut and cover excavations and huge pump stations, is one of the major engineering projects in the world. In addition, many old and dilapidated "inner city" areas are being aggressively redeveloped for commercial, residential or recreational purposes. Examples in this category extend from London's Docklands to Pittsburgh's Golden Triangle, and from Baltimore's waterfront to St. Louis' railway station.

Irrespective of purpose or geography, there is a common set of factors facing engineers engaged in such projects.

- Construction must be carried out in heavily urbanized areas with existing above and below ground structures and services.

- Construction is confined to specific sites within these areas, and so must accommodate the particular geological conditions. Few major cities are founded directly on solid rock, and most have extensive artificial fills overlying glacial, alluvial or marine deposits in which the groundwater level is within the depths touched by construction.

- Construction is witnessed by, and directly impacts, the inhabitants of these areas.

As a consequence, urban construction must resolve the problems of restricted access, unfavorable ground conditions and environmental compatibility. There is no room - literally and metaphorically - for the "walk away" solution: in situ solutions must be found since relocation to an easier area is not usually a viable option. In addition, there is increasing pressure, nationally, that such solutions must be achieved at minimum cost. As Nicholson (1987) wrote, "For a long time, America used to have surplus money to 'throw' at problems.... this has not been true for a number of years." Furthermore, the competition between the growing number of highly competent specialist contractors is intense, and so cost considerations are crucial from their viewpoint also.

These problems have already been addressed for many years in several countries of the Old World and the Far East for many reasons ranging from the need to repair war damages to the necessity to provide proper facilities for growing populations in geographically restricted areas. These countries have fostered the development of many original geotechnical construction techniques, popularly referred to in America as the "European Technologies." And, of course, such techniques have been allowed to mature in contractual and legal environments that encourage calculated risk while not punitively penalizing imperfections.

Engineers in the United States frequently debate the reasons that they appear to be importers of new technologies. and seem to have little innovative impact on foreign practice. O'Rourke (1987) and Nicholson (1987) both emphasize the restrictive impact of the unimaginative contractual framework and heavily litigious atmosphere in America, factors that are unlikely to change either favorably or quickly. There is also the fundamental question of necessity and, in this respect, the exploitation of new technologies is being forced on American engineers as they face contemporary challenges in several fields, as well as in urban engineering. Waste containment, dam rehabilitation and liquefaction control are equally demanding attention at the present time.

The debate on the reasons for the status of American engineering as a net importer of new technologies need not be restated. Rather, the situation can be accepted, and American engineers should, therefore, adopt the viewpoint that they are in a very privileged position. They can afford to be very selective and highly critical of these techniques with respect to satisfying current particular requirements - without having to endure the risk and expense of the development process.

This paper introduces a number of newer developments in geotechnical construction which have been fostered by the demands of urban engineering. Six topics are described briefly, for applications in soft ground tunnelling, retaining walls and structural support. The list is not comprehensive, being limited to the direct experience of the author and his company and by the space available. The following techniques are introduced:

A. Soft Ground Tunneling

(i) Premilling
(ii) Grouting

B. Retaining Walls

(i) Soil nailing
(ii) Post grouted anchorages
(iii) Hydromill diaphragm wall excavation

C. Structural Support

(i) Minipiling

The range of techniques and applications is great, but they do share the common goals of minimizing environmental impact, and optimizing construction time and safety.

2. NEW TECHNIQUES IN SOFT GROUND TUNNELLING

2.1. Premilling

Papers describing the successful development of "predecoupage mecanique" (Premill) as a specialized tunnelling technique in rock and soils have appeared sporadically in the French technical press over the last few years, and a review was presented recently by Bruce and Gallavresi (1988).

In essence, the system comprises a track-mounted frame, of shape corresponding to the tunnel extrados (Figure 1). Mounted on the frame and projecting out in front is a large band saw type milling machine about 3m long. This can be moved around the frame to cut a slot about 120-200mm wide into the ground around the volume to be excavated. In competent rock this slot is left open, to optimize the subsequent blasting parameters and performance. In soft ground the slot is filled immediately with high strength, fast setting concrete, so forming an insitu arch to minimize decompression effects during subsequent excavation.

The system was evolved in response to the need to absolutely minimize construction related effects in urban areas involving large diameter tunnels close to the surface under old and delicate structure. Under such conditions even the excellent performance afforded by the standard New Austrian Tunneling Method was not acceptable. The early examples for railroad and Metro construction in France have been followed by similar contracts for example in Belgium, Spain, and Italy.

In competent rock formations of up to 250 bar (3500 psi) compressive strength, the premill is used only to provide a continuous slot around the volume to be excavated - usually with a drill and blast method. Premilling provides the following major benefits:

- less explosives (and blast holes) are required, rendering the entire blasting operation safer, faster and environmentally more acceptable.

- fissuring or decompression occurs in the surrounding rock mass, thus preserving its virgin properties, and so reducing the demand for subsequent reinforcement, e.g., with bolts.

- there is no overbreak and, therefore, there are associated cost savings in time, effort and materials.

- the smooth profile makes the placing and performance of arches more efficient.

- less contact or consolidation grouting is needed behind the final tunnel lining.

- following blasting, there is a greatly reduced danger from rock falls due to chimneys of fractured ground developing above the excavation.

- the magnitude of vibrations transmitted upwards towards nearby surface structures is greatly attenuated.

Developments of the technique continue, for example, in special diamond tools, high pressure water jetting, and increased cutting power, to permit its use in harder rock formations, faster, and with increased safety.

In <u>soft ground</u> as noted above, the major difference is that the cut slot is filled with a special concrete mix as early as possible. The advantages are as identified above for the rock premill, although the prime target is the elimination of surface settlements induced by the tunnelling.

Each cover, up to 3.5m long, depending on the soil, is inclined slightly outwards and overlaps the preceding one by 300 to 500mm. The cone is cut in discrete segments so that the concrete can be placed in each segment, as early as possible and without having to wait for the whole arch profile to be first completed. Cutting times for a typical 3m long segment may be as low as one minute.

The concrete may be placed by dry or wet shotcrete methods. A typical mix reported by Bougard et al (1979) comprises, per cubic meter of mix:

Cement	450Kg	Coarse gravel	650Kg
Sand	560Kg	Accelerator (Sigunite)	27Kg
Fine gravel	650Kg	Water, as appropriate, typically w/c = 0.25 - 0.30	

This gives a strength of up to 100 bar (1400 psi) at eight hours. Spraying the mix into the premilled slot ensures that none of the fine aggregate is lost, as is the case in conventional NATM applications of shotcrete on open faces. The concrete in place is, therefore, of superior quality, further enhancing the performance of the system.

In comparison with the NATM, there are certain similarities, notably the overall concept of the support, and the common construction elements such as shotcrete, bolts, and arches. However, the major dissimilarity is that with premilling the primary lining is placed up to 3m ahead of the face before excavation, whereas in NATM the lining follows 1 or 2m behind the excavated face. This greatly impacts the generation and scale of tunnel deformations, and so the effect on overlying structures. Goer (1982) described a monitored case history of the relative performance of the two methods in the same material - Argenteuil marl (Figure 2). Typical properties of this material were listed as:

Density = 2
Effective ϕ = 20°
Effective cohesion = 0.5 - 1.0 bar
Undrained cohesion = 1.5 bar
Deformation modulus = 500 bar

Three times less settlement was achieved in the tunnel protected by premilling.

Most of the earlier applications have been carried out with the conservative "divided section" profile. However, excellent results with the "full section" profile (i.e., cutting a 270° arc and excavation in one pass) in a shallow circular collector tunnel 3.50m diameter (Departement de la Seine-Saint-Denis, France), in very difficult ground, encouraged its use in Lot 7 of the Lille Metro, Belgium. As evident in Figure 3 the performance of the full section profile was superior, with surface settlements no more than 1mm. Prefabricated base slabs were connected structurally to the premill cover by shotcrete, and the steel ribs then placed, bearing on the slabs. This system also proved faster than the divided section approach.

2.2. Grouting

Ground treatment by grouting is hardly a new technique: Charles Berigny repaired a harbor sea lock in Dieppe, France, in 1802 using basic cement grouting techniques, whilst the first major US application dates from 1910 and shaft sinking for the Catskill Aqueduct, New York. Applications in dam foundation sealing and strengthening were for many years the principal American market. However, dam grouting in this country stultified as a direct consequence of restrictive construction practices and unimaginative contractual procedures. As a result, our industry could not cope with the geotechnical challenges issued by dam construction on the less favorable sites left in the last twenty or thirty years. Consequently, major projects were completed wherein the standard of the grouting executed was, simply, poor and the effectiveness highly questionable. Such imperfections soon became common knowledge - although the causes remained uninvestigated: the word soon emerged that grouting "doesn't work."

Today major Federal dams are being repaired against seepage around or under the structure by using concrete diaphragm walls - often constructed by the hydromill excavator, described below. This approach can guarantee an efficient cutoff, but at significant financial premium over grouting, properly executed.

This poor opinion of grouting as an engineering tool has permeated the thinking of those involved in tunneling and deep foundations too, to the extent that grouting is still regarded by many as a last resort - to be attempted when everything else has failed. Although understandable, this attitude is patently unfair and wholly unjustified, in the light of major strides made in the last decade on the execution and control of grouting works.

In rock grouting, significant advances have been made in techniques (e.g. the MPSP system of treating difficult rock masses - Bruce and Gallavresi, 1988) and in materials, (e.g. the use of microfine grouts to penetrate fine fissures Karol, 1985).

Soil grouting is in an even more dynamic situation, benefitting rapidly from technological advances made by chemists, physicists, instrumentation engineers, and geotechnicians. Many of these developments have been associated with tunneling in urban areas, principally for subway or sewer projects. The aim has been to provide better aids to speed progress, improve safety and minimize associated settlements. This aim has been so efficiently achieved in Europe and the Far East that grouting is there incorporated routinely, abinitio, as an integral part of the process.

Bruce and Boley (1987) summarized four categories of soft ground grouting (Figure 4), and for most purposes only the following three have validity for work in urban areas, in U. S. practice.

o Compaction grouting is a specialized "uniquely American" process that has been used since the early 1950s (Baker et al, 1983). Very stiff soil-cement mortar is injected at high pressures (up to 35 bar (500 psi)) at discrete locations to compress and increase the density of soft, loose or disturbed soil. Unlike the case of hydrofracture grouting, the grout forms a very dense and coherent bulb that does not extend far from the point of injection. Near-surface injections result in the lifting of the ground surface (the technique of slab jacking as described, for example, by Bruce and Joyce, 1983) and, indeed, the earlier applications were used exclusively for levelling slabs and light buildings on shallow foundations (Warner, 1982). Prior to the Bolton Hill Tunnel project, compaction grouting had been used in the Baltimore subway project to correct settlement problems caused by subway tunnel construction - but only after the tunnel had been completed and settlement of overlying buildings had occurred (Baker et al, 1983). However, the Bolton Hill project marked a fundamental change, in that compaction grouting was conducted during the excavation of the tunnel, at locations just above the crown. In this way, major surface settlements were prevented from devleoping at the source. Although compaction grouting has practical and technical limitations, its popularity is growing, mainly as the result of the well-researched (and publicized) Bolton Hill project. However, its application should be most carefully reviewed when dealing with tall structures or buildings that can tolerate only the smallest differential movements. Under such conditions, it is imperative to attack the cause of the settlements at the source, and prevent them from migrating away from the excavation. Permeation or replacement grouting is then necessary.

The techniques involved in permeation grouting are the oldest and best researched. The aim of the method is to introduce grout into soil pores without any essential change in the original soil volume and structure. The properties of the soil, and principally the geometry of the pores, are clearly the major determinants of the method of grouting and the materials that may be used (see Figure 5). Excellent reviews of the subject are provided by the FHWA (1976), Cambefort (1977), Karol (1983), and Littlejohn (1983). Other permeation grouting methods, principally from Japan, are described in an earlier study (Bruce 1984).

° Replacement grouting is the youngest major category of ground treatment. According to Miki and Nakanishi (1984), the basic concept was propounded in Japan in 1965, but it is generally agreed that it is only within the last 10 years that the various derivatives of jet grouting have approached their full economic and operational potential. Its development was fostered by the need to thoroughly treat soils from gravels to clays to random fills in areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements. As indicated in Figure 5 jet grouting can be executed in soils with a wide range of permeabilities. Indeed, any limitations with regard to its applicability are imposed by other soil parameters (e.g., the shear strength of cohesive soils or the density of granular deposits).

The ASCE Geotechnical Engineering Division Committee on Grouting (1980) defined jet grouting as a "technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn." Figure 6 depicts one particular type in which the soil is jetted by an upper nozzle ejecting water at up to 600 bar (8400 psi) inside an envelope of compressed air at up to 12 bar (170 psi). The debris is displaced out of the oversized hole by the simultaneous injection of cement-based grout through a lower nozzle (up to 70 to 80 bar). Other simpler variants utilize only grout jetting alone to simultaneously erode and inject, giving much more of a "mix in place" action.

Most jet grouting is conducted to provide circular columns, but panels or membranes can be cut in the ground by omitting rotation during the withdrawal of the tool: the nozzles then act monodirectionally.

In permeation grouting major new trends are evident in

(1) Methods - e.g. powerful diesel hydraulic drilling rigs capable of drilling quickly in restricted conditions through difficult ground to depths of over 60m if necessary.

(2) Materials - new families of stable, microfine cement-based grouts, and high-strength low viscosity chemical grouts which do not creep or synerise. (Tornaghi et al 1988).

(3) Instrumentation and Control - Throughout the grouting industry, the use of computer-aided devices as monitors and controls over grouting operations in the field is increasing. This growth is reflected in several of the papers presented at the "Issues in Dam Grouting." session of the ASCE Convention, Denver, 1985. The most effective of these instrumentation systems, as far as injections are concerned, is similar to the electronic PAGURO system of centralized monitoring and grouting control that has been

developed in Italy. This system displays in real time numerically and graphically the full injection characteristics of each pump (the setting of which remains under manual control). It thereafter gives a printout summary of each sleeve injected (including volume, maximum and average pressures, flow rates and time). Such data then provide the basis for the technical review of the grouting conducted (e.g., grout take analyses) and the quantities of work executed, for payment purposes. Clearly, the investment in such sophisticated equipment is economically justifiable only in projects of appreciable scale and/or complexity such as the Milan subway (Fairweather, 1987).

Most recently, however, a major break-through has been made in Italy in the exploitation of instrumentation for soil investigation and grout parameter design. The sensors of the PAPERO system for drilling investigation continuously record the drill penetration rate, rotational speed, thrust, torque and flush pressure encountered in drilling a certain exploratory hole. These data are combined to give a single unified factor - specific energy. Thereafter, the computer relates this factor to ground type, and prints out a geological log with boundaries at 100mm intervals (see Figure 7). This geological log permits optimization of the subsequent drilling and grouting parameters as well as furnishing invaluable information to the tunnelling contractor in that potentially dangerous conditions (e.g., sand runs) can be closely predicted. The accuracy of the geological log has proved exceptional given the conditions of the Milan subway project (mixed gravels, sands and silts to over 25 m in depth) and groups of three investigatory holes have been routinely drilled at about 6m intervals from the pilot tunnel along much of its length.

The key to the accuracy is obviously the ability of the computer to relate specific energy with ground type. This accuracy has been achieved by conducting statistical analyses of the specific energies recorded at discrete depth intervals, in correlation with visual observations (from core samples) of the ground type. In this way, the influence of depth on in situ ground properties as well as other factors such as the hydrological regimes and borehole inclinations are accommodated, which is not the case in other, less successful systems of drilling parameter analyses.

Similar advances are being made in jet grouting to the extent that major Metro tunnels have been constructed through soft marine clays in Singapore (Mongilardi and Tornaghi, 1986) and railway embankments have been founded on highly compressible peats, stabilized by grouting (De Paoli et al 1988).

In this country, perhaps about 50 projects involving jet grouting have been completed to date by a handful of specialists. If some of these projects have not had entirely satisfactory outcomes, this reflects the decision to employ the technique for the wrong application, and occasionally it reflects the inexperience of the operators and engineers involved.

Nevertheless, it can only be concluded that the significant advances made in grouting over the past ten years offer U. S. engineers an extremely versatile, controllable and effective tool in minimizing environmental impact in urban construction.

3. NEW TECHNIQUES IN RETAINING WALLS

3.1. Soil Nailing

Soil nailing is one of the family of insitu soil reinforcing techniques summarized by Bruce and Jewell (1986-1987). It comprises steel reinforcing elements grouted into horizontal or sub-horizontal holes drilled into the cut face of the excavation as it proceeds downwards in stages. The inserts improve the shearing resistance of the soil by being forced to act in tension. They clearly differ from the nature and mode of action of the other members of the family (Figure 8), namely reticulated micropiles and large diameter soil dowels.

The history of development in the three principal countries of origin, namely France, Germany and the United States, is fascinating and encompasses almost two decades of often erratic progress. Suffice it to note that today soil nailing is one of the fastest growing geotechnical construction techniques within North America, and has enjoyed a boom since the execution of the foundation excavation for the PPG Building in Pittsburgh, Pennsylvania, in 1982 (Nicholson and Boley, 1985).

This upsurge in interest is reflected in the publishing of the National Cooperative Highway Research Program (NCHRP) Report 290 in 1987 on all types of insitu reinforcement and the attention paid to it at the ASCE Atlantic City Ten-year Update Symposium, also in 1987. On the experimental side, FHWA research contracts were let in 1985 for additional fundamental laboratory and field tests, leading to the issue of a formal design manual in 1989.

The standard sequence of construction is illustrated in Figure 9 - basically it is a cycle of excavate-spray with shotcrete - nail, and so on. Detailed case histories are provided by Louis (1987) and Bruce (1988) amongst others. Special attention must be paid to drainage of the face and soil mass during and after construction, and to corrosion protection of the inserts in permanent applications.

Design is complex and there seems to be no "best" method, although the kinematical limit analysis proposed by Juran and Beech (1984) seems consistent with observed structural performance.

There is a wealth of experimental and construction data on the performance of soil nailed excavations in urban settings. Most significantly, Juran and Elias (1987) concluded that post-construction observations in non-plastic soils have shown that after the end of construction, ground movement and facing displacement (maximum at the top) do not typically exceed 0.3% of the total excavation depth and one rapidly stabilized. This puts soil nailing very firmly in Pecks (1969) Category I of excavation performance.

Soil nails differ from ground anchorages in several aspects:

a) Nails are fully bonded to the soil over their entire length.
b) Nails are not prestressed.
c) Nails are relatively closely spaced e.g. 1 per 1.5 to 2.5 square meters of face, to generate the soil mass-insert interaction.

d) High loads are not transmitted to the head of the nail at the cut face. Therefore, there is no need for elaborate bearing plate arrangements.
e) Nails are shorter (typically 50-100% of excavation depth depending on the soil) and so need only relatively light and mobile drilling equipment capable of drilling up to 150mm diameter holes. However, if overall stability calculations indicate a problem to be deep seated then prestressed anchorages will most probably be required either as the only retaining element or compositely with the nailed structure.

It is important to note well the benefits and limitations. In the former category, there are

- Economic advantages: where nailing is possible technically, it is typical to find that the cost savings for excavations on the order of 10m deep are 10 to 30 percent, relative to the use of an anchored diaphragm or Berlin wall. These projected savings are supported by the reported savings of 30 percent on a soil nailed excavation in Portland, Oregon, (ENR) 1976.

- Construction equipment: drilling rigs for reinforcement installation and guns for shotcrete application are mobile, quiet and relatively small in size. Their use is highly advantageous in urban environments, where noise, vibration or access may pose limits on the type of equipment that can be used.

- Construction flexibility: soil nailing can proceed rapidly and the excavation can be shaped easily. It is a flexible technique, readily accommodating variations in soil conditions and work programs as excavation progresses.

- Performance: field measurements indicate that the overall movements required to mobilize the reinforcement forces are surprisingly small. Furthermore, nailing is applied at the earliest possible time after excavation, and in intimate contact with the cut soil surface, thus minimizing any disturbance to the ground and the possibility of damage being caused to adjacent structures. This "early support" also allows the natural undisturbed properties of the soil to be exploited to advantage. As demonstrated by Gassler and Gudehus (1981), nailed structures can withstand both static and dynamic surcharge loading without excessive settlements if properly designed.

Major practical limitations are:

- Soil nail construction requires the formation of cuts generally 1 to 2m high in the soil. These cuts must then be capable of standing up unsupported for at least a few hours, prior to shotcreting and nailing. The soil must, therefore, have some natural degree of "cohesion" or cementing. Otherwise, a pretreatment such as grouting may be necessary to stabilize the ground immediately behind the face.

- A dewatered face in the excavation is desirable for soil nailing. If the ground water tries to percolate through the face, the unreinforced soil will slump locally on initial excavation making it impossible to establish a satisfactory shotcrete skin.

- Excavations in soft clay are also unsuited to stabilization by soil nailing. The low bond resistance possible in soft clay would require a very high density of in situ reinforcement of considerable length to ensure adequate levels of stability, while creep will also affect performance. Bored or jet grouted piles, or diaphragm walls with anchorages are more suited to these conditions.

3.2. Post-grouted Anchorages

Prestressed ground anchorages have been employed throughout the world since 1934. They were introduced into the States during the 1960s and were used initially only as temporary excavation support, although many of the applications were in major urban excavations in very variable soil (Figure 10). Such ground conditions may give rise to very low grout/ground bond values or significant creep amounts in service. In addition, erratic anchor behavior can occur, due to subtle local variations in the soil.

To increase bond capacity (and, therefore, reduce anchor length and cost), to reduce creep, and to regularize anchor performance in such soils, post-grouted, or regroutable, anchorages have been developed in Western Europe. Their relationship to conventional anchorage types is best illustrated in the British Code of Practice BS 8081 which defines four types (Figure 11).

Type A anchorages: consist of tremie (gravity displacement), packer or cartridge grouted straight shaft boreholes, which may be temporarily lined or unlined depending on hole stability. This type is most commonly employed in rock and very stiff to hard cohesive deposits. Resistance to withdrawal is dependent on side shear at the ground/grout interface.

Type B anchorages: involve low pressure (typically grout injection pressure $p_i < 10$ bar (140 psi) grouted boreholes, where the diameter of the fixed anchor is increased with minimal disturbance as the grout permeates through the pores or natural fractures of the ground. This type is most commonly employed in weak fissured rocks and coarse granular alluvium, but the method is also popular in fine grained cohesionless soils. Here cement-based grouts cannot permeate the small pores but under pressure the grout compacts the soil locally to increase the effective diameter and enhance the shearing resistance. Resistance to withdrawal is dependent primarily on side shear in practice, but an end bearing component may be included when calculating the ultimate capacity.

Type C anchorages: feature boreholes grouted to high pressure (typically $p_i > 20$ bar (280 psi)), via a lining tube and packer (i.e. sleeved pipe system). The bond zone is enlarged by hydrofracturing of the ground mass to give a grout root or fissure system beyond the drilled diameter of the borehole after initial stiffening of primary grout placed as for Type B anchorages (Figure 12). A relatively small quantity of secondary grout is needed. Continuous flow or a sudden drop on initial injection pressure might indicate hydrofracture after which only relatively limited pressures can be achieved.

Post-grouted anchorages of this type are commonly applied in fine cohesionless soils, whilst increasing success has also been achieved in stiff cohesive deposits. Design is based on the assumption of uniform shear along the fixed anchor.

Type D anchorages: consist of tremie grouted boreholes in which a series of enlargements, either bells or underreams, have previously been formed. This type is employed most commonly in firm to hard cohesive deposits. Resistance to withdrawal is dependent on side shear and end bearing, although, for single or widely spaced underreams, the ground restraint may be mobilized primarily by end bearing. Such anchorages are becoming less popular, having been superseded by the post-grouted types.

For the design of post-grouted anchorages in cohesionless soils, calculations are based on design curves created from field experience in a range of soils rather than relying on a theoretical or empirical equation using the mechanical properties of a particular soil. In alluvium, for example, test results (Jorge 1969) have indicated in boreholes of 100 to 150mm diameter, ultimate load holding capacities 9 to 13 tons/m of fixed anchor at a grouting pressure of 10 bar (140 psi) and 19 to 24 tons/m at a pressure of 25 bar (350 psi).

In more recent years, design curves for post-grouted anchorages have been extended through proving tests in Germany (Ostermeyer, 1974). For sandy gravels and gravelly sands, it has been found that the ultimate load increases with density and uniformity coefficient. The results of a large number of fundamental tests are shown in Figure 13, which can be used as a design guide for borehole diameters of 80 to 160mm. Skin friction increases with increasing consistency and decreasing plasticity. The technique of post-grouting is also shown to generally increase the skin friction of very stiff clays by some 25% to 50%, although considerably greater improvements are claimed for stiff clay of medium to high plasticity. Ostermeyer also found that there was a steady increase in skin friction as the post-grouting pressure was increased up to 30 bar (420 psi).

3.3. Hydromill Diaphragm Wall Excavation

Diaphragm walling (slurry trenching) is another technique which can hardly be described as new to these shores. For example, Saxena (1974) described the construction of the massive wall built for the World Trade Center foundation excavation in New York in 1968/69. There are a number of reputable specialist companies operating throughout the country, although most of the activity has so far been in the Northern and Eastern States.

The principle of the technique is well-known: a trench, typically 600-1000mm wide and 2.5 to 3m long, is excavated vertically by grab, to the required depth. The trench is maintained filled with bentonite slurry to keep it open during excavation. After excavation is complete a steel reinforcing cage is usually placed in the panel and the bentonite slurry then displaced out of the trench by concrete tremied into the trench from the base up. In this way, insitu reinforced concrete panels are formed in the ground. During construction of the wall, alternate panels are excavated and cast (Primaries). When concrete has reached a certain strength, the intermediate (Secondary) panels are formed contiguously, thus forming a continuous wall in the ground.

Removal of the soil on one side of the wall follows, leaving the diaphragm exposed, to act as a retaining wall or as the support of a deep excavation. Most commonly prestressed anchors are installed through the wall to provide shoring.

Excavation is carried out by a clamshell bucket, either rope suspended or operated from a KELLY bar. Diaphragm walling in this way has always experienced problems with:

- The presence of major boulders.
- The difficulty of "toeing" into rock.
- Making joints between adjacent panels watertight.
- The handling and disposal of bentonite slurry and waste material - especially in city sites.
- Restricted depth capacity (say 50m).

In addition, the drive for increased productivity and the need to have a technological 'edge' on the competition are always constant spurs to developments.

Less than 15 years ago, the hydrofraise excavating machine began to be developed in France. (Figure 14). Called the hydrofraise by the French specialists Soletanche, it comprises a steel frame also serving as a guide, on which are mounted two cutting wheels, hydraulically powered, and rotating in opposite directions. The wheels have tungsten carbide teeth. A third down-the-hole motor operates a reverse circulation mud pump located above the wheels which carries the excavated debris, in the bentonite slurry, to the surface. The "dirty" mud is cleaned there and returned out the top of the trench. The guide frame is attached to the crane operated cable from which it is suspended by a hydraulic feed cylinder which can be controlled a) to give a constant rate of advance or b) to maintain a constant weight on the cutters (16-20 tons maximum).

The cutters can readily chew into the concrete of adjacent primary panels thus ensuring excellent joint properties. They can penetrate all kinds of soil and rock with compressive strengths of up to 1000 bar (14,000 psi). The absence of vibration and shocks, plus the self-contained debris and slurry handling system makes it ideal for urban sites. Overbreak is also less than for conventional systems (less than 10%) and verticality can be controlled and corrected to less than 0.2% if required.

Standard panel sizes are shown in Figure 15. About 20% of the 600,000 square meters excavated by hydrofraise throughout the world has been conducted in the US, with the greatest proportion being on three major Federal dams. On one - Navajo Dam, New Mexico - a world record depth of almost 120m was reached, including up to 60m in bedrock (Fairweather, 1987). Other excellent and typical case histories include shafts for the Channel Tunnel, France (Evers and Hovart 1988) and deep excavations for a nuclear power station in England (Anon, 1988) and for Baltimore-Harbor Place Building (Soletanche 1987).

Production rates can be very high and quoted figures run from 120 square meters/shift in tough alluvium and hard limestone in Paris (Soletanche, 1980), to 40 square meters/hour in dense sands and clay in England (Anon, 1988).

The hydromill is an expensive machine to buy and operate but has clearly several major advantages over conventional diaphragm walling methods. Its speed reduces unit costs, and any cost premium still remaining can often be offset against the environmental bonuses it offers.

4. NEW TECHNIQUES IN STRUCTURAL SUPPORT

4.1. Minipiling

It is now almost 40 years since the technique of minipiling was first applied in Italy (Koreck 1978, Weltman 1987, ASCE 1987). Following the lapse of the early patents, there has been a tremendous growth in the market volume particularly in the cities and industrial centers of W. Europe and S. E. Asia. In this country the start was much later but the expansion has been equally dramatic in the last five years or so, as rebuilding and redevelopment of our older cities picks up momentum. Indeed it is the author's observation that there is probably a greater intensity of minipile activity in Boston, Massachusetts, and New York City than in any other cities in the world. (Bruce 1988)

Minipiles are cast in situ bored piles rarely more than 300mm in diameter and 30m deep. A fundamental feature is their ability to be constructed by equipment of the type used for anchoring and grouting works, as opposed to that needed for conventional bored or driven piles.

Minipiles can be constructed to considerable depths through all types of soil, rock and obstructions, and in virtually any direction. They have a high slenderness ratio and so transfer load almost wholly by shaft friction, eliminating any requirement for underreaming at the base to enhance end bearing. All feature substantial steel reinforcing elements and so can sustain axial loading in both senses. The reinforcement can also be designed to resist bending stresses safely and with minimal displacement.

The construction steps (Figure 16) are characterized by equipment ensuring minimum vibration, ground disturbance and noise, and capable of operating efficiently in awkward and restricted access and working conditions. Thus, although their nature may result in them being lineally more expensive than conventional driven or large diameter piles, they may be the only guaranteed solution given a particular set of ground, site, program and performance conditions.

Regarding their service behavior, minipiles exhibit relatively high carrying capacity (for their diameter) and very small settlements. Piles installed wholly in soils can be constructed to provide safe working loads approaching 100 tons, whereas recent work in Boston (Johnson and Schoenwolf, 1987) and New Jersey, (Bruce 1988) shows that when founded in rock, safe working loads two or three times that figure can be sustained.

Load holding capacity can be improved substantially by the post-grouting techniques described earlier as used for anchors. Settlements to structures being underpinned can be almost eliminated by preloading the piles to working load - by prestressing - so that no further pile movement occurs when the structural load is finally applied.

Another very significant feature involves the consideration of interpile spacings with respect to pile group performance. For example, the British Code of Practice (CP2004, 1972) states that for "friction piles, the spacing center to center should not be less than the perimeter of the pile." On the other hand, test information (Plumelle, 1984) shows that closely spaced minipiles, especially when inclined, interfere positively, as illustrated in Figure 17. Undoubtedly there is a soil structure reaction, as in insitu reinforcing, which is being exploited with excellent results in tunnel related applications (Figure 18).

Overall, therefore, minipiles are an excellent option for upgrading or replacing existing foundation to sustain increased structural loadings, or to help them resist additional settlements arising from adjacent new constructions such as tunnels or deep excavations. They are also finding increasing application as support for new foundations bearing on very difficult geologic profiles which would render conventional piles or caissons exceptionally difficult or very costly to install.

5. FINAL REMARKS

These brief introductions to various ground engineering techniques which have been developed primarily for construction in urban areas highlight that we have considerable power at our fingertips. These techniques cannot be described merely as "having potential" - their potential has been realized and exploited in the countries of origin to the extent that in this country we now have access to tried and proven systems of major relevance.

It can only be hoped that practitioners in this country will be more willing to employ "new technologies" than they have generally been in the past. It is recognized, however, that such innovation must be pushed through in the face of our litigious atmosphere which certainly gives no encouragement in this respect.

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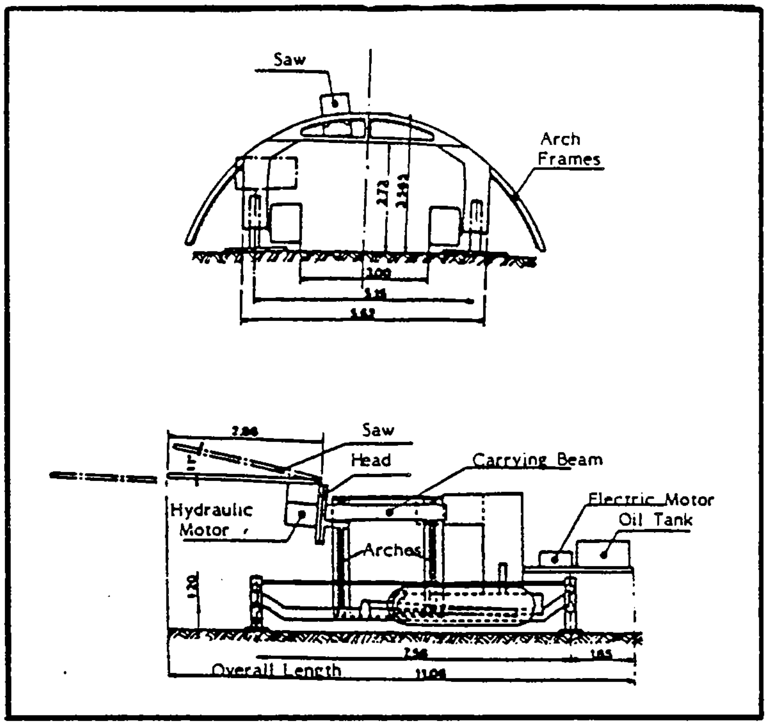


Figure 1. General layout of typical premill machine (Hydraulic arm for shotcreting not shown).

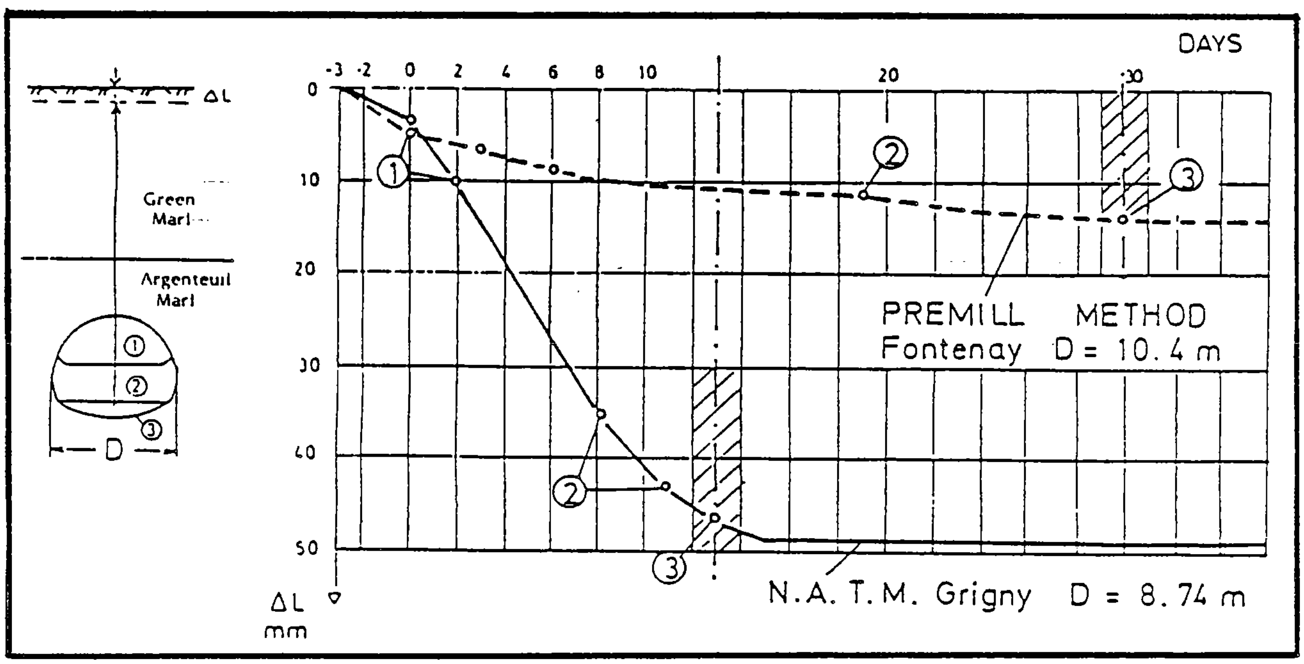


Figure 2. Comparison of settlements generated with time in identical geological conditions, by Premill and NATM. (LeGoer, 1982)

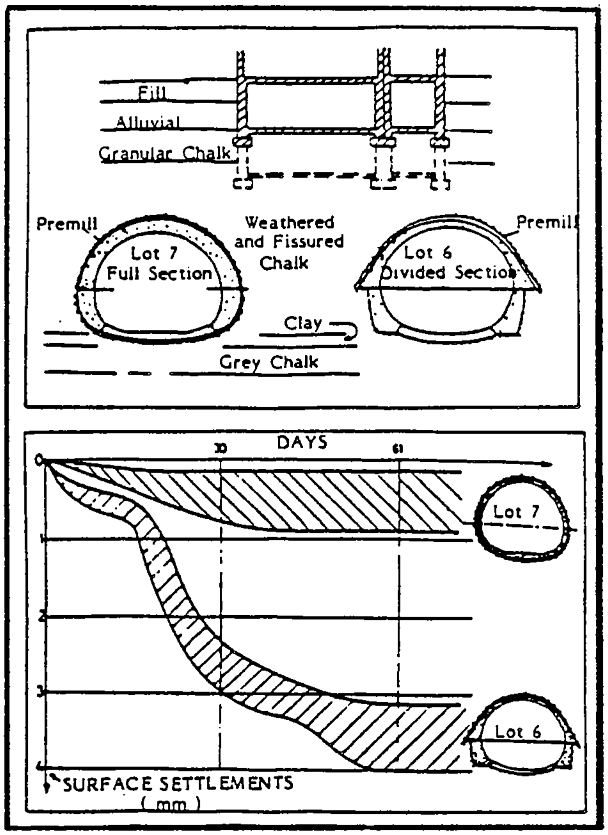


Figure 3. Comparison of settlements generated by Premilling in Divided Section and Full Section, Lille, Belgium (LeGoer, 1982)

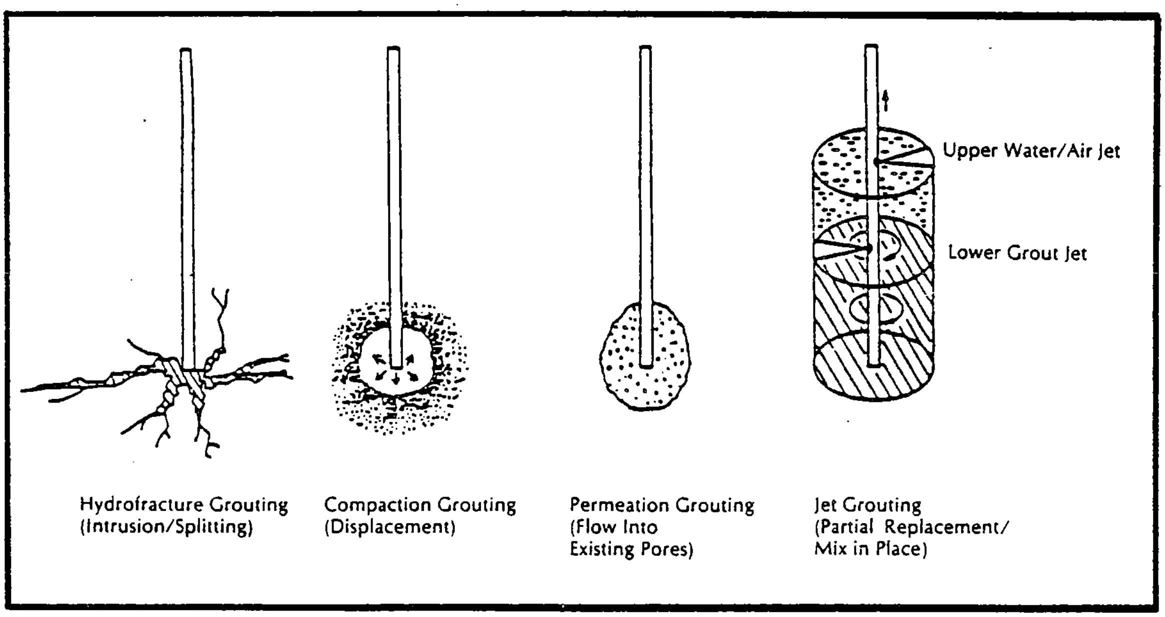


Figure 4. Basic categories of soil grouting.

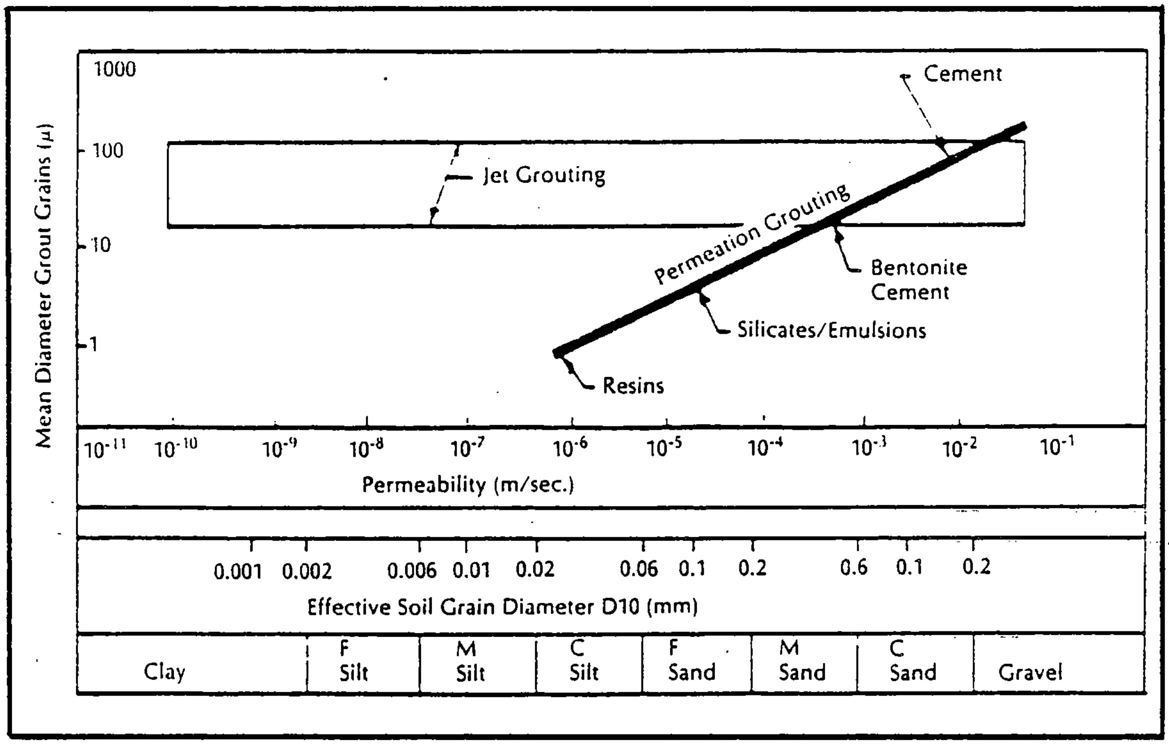


Figure 5. Groutability of soils in relation to grout and soil properties (After Coomber, 1985)

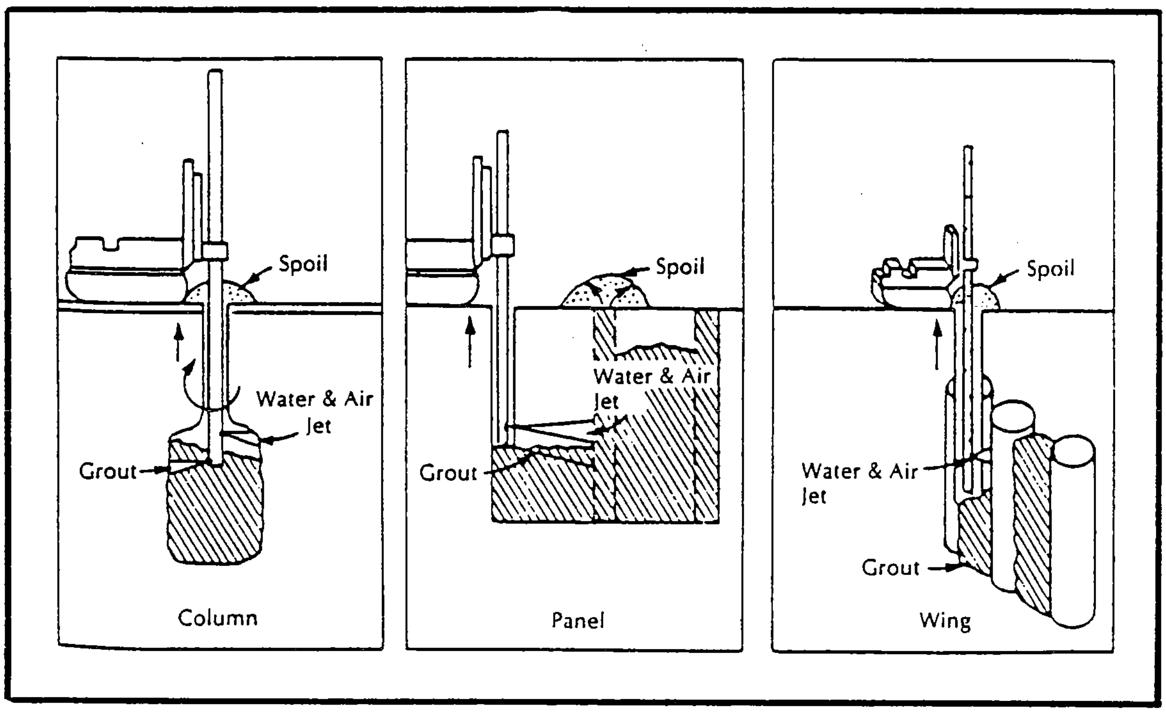


Figure 6. Jet grouting options using the three fluid system (i.e. air, water and grout) (After Coomber, 1985)

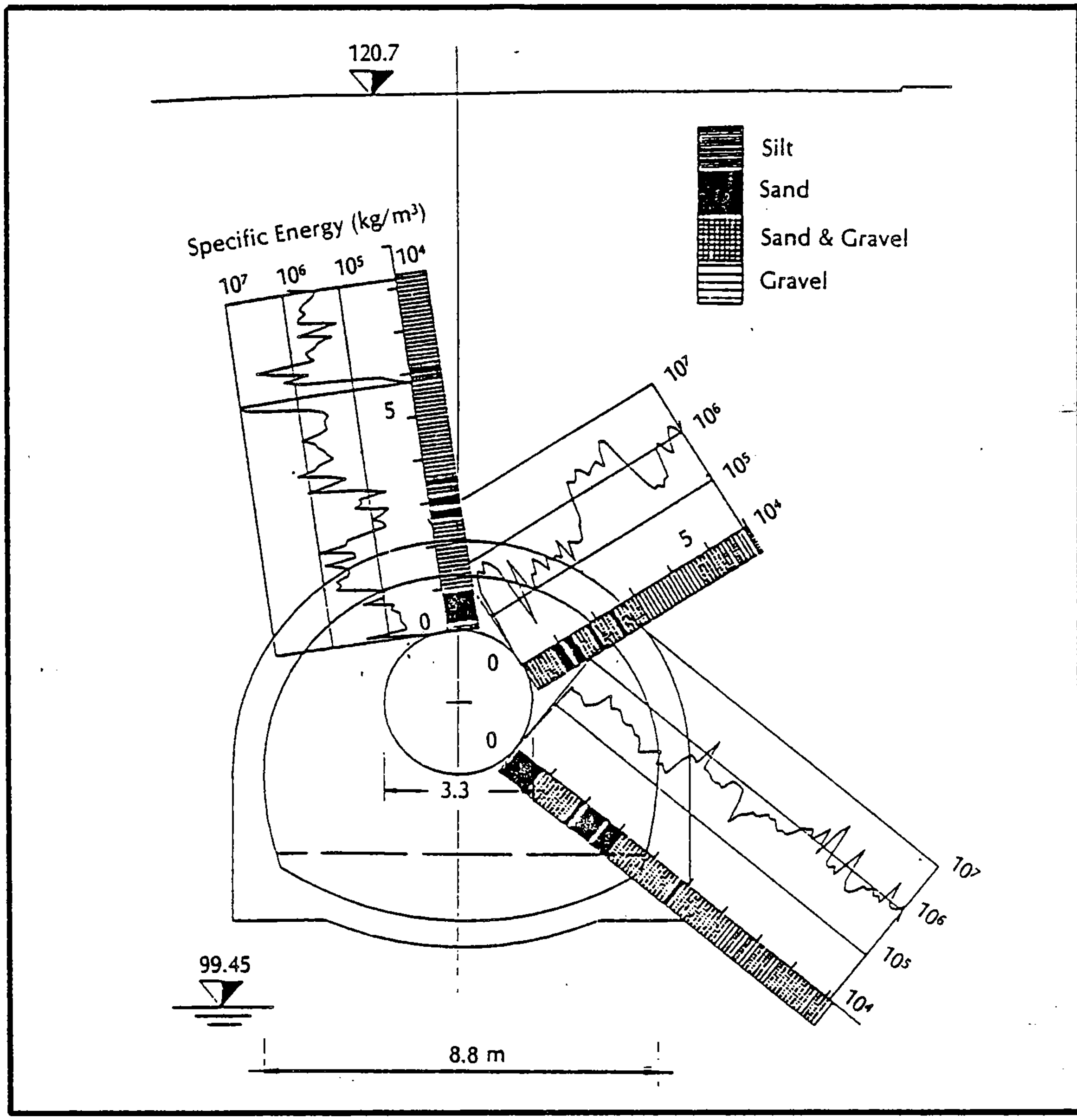


Figure 7. Soil profiles derived from the evaluation of electronically recorded drilling parameters (PAPERO) in terms of specific energy (Milan Metro, Line 3)

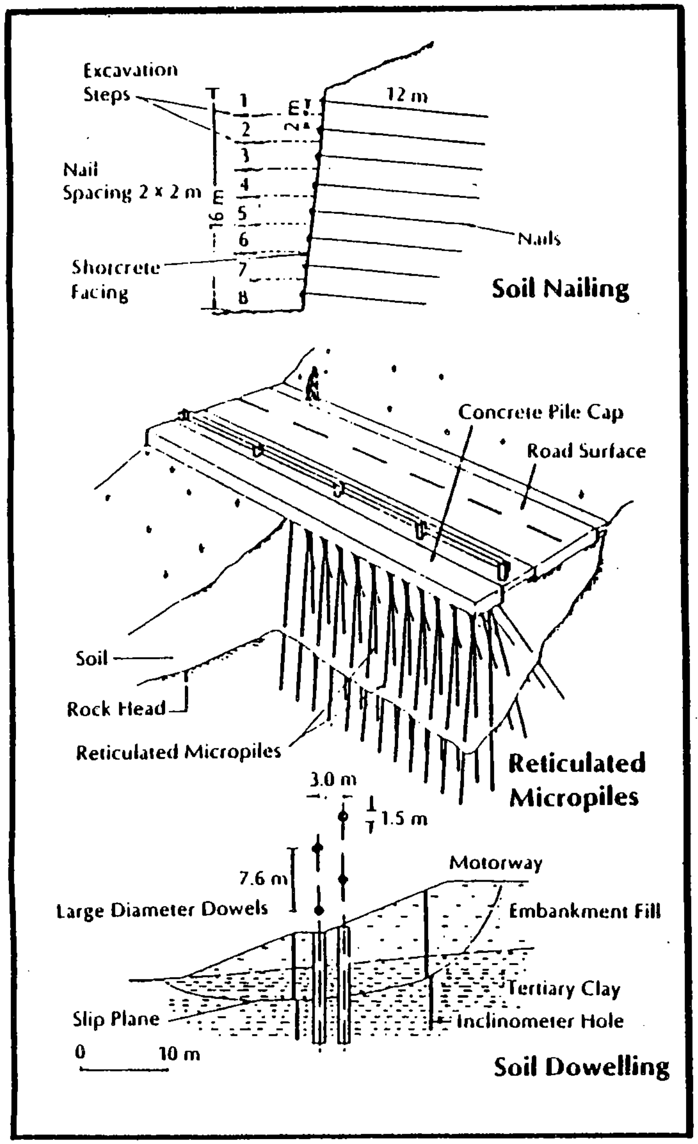


Figure 8. The family of insitu soil reinforcing techniques (Bruce and Jewell, 1986)

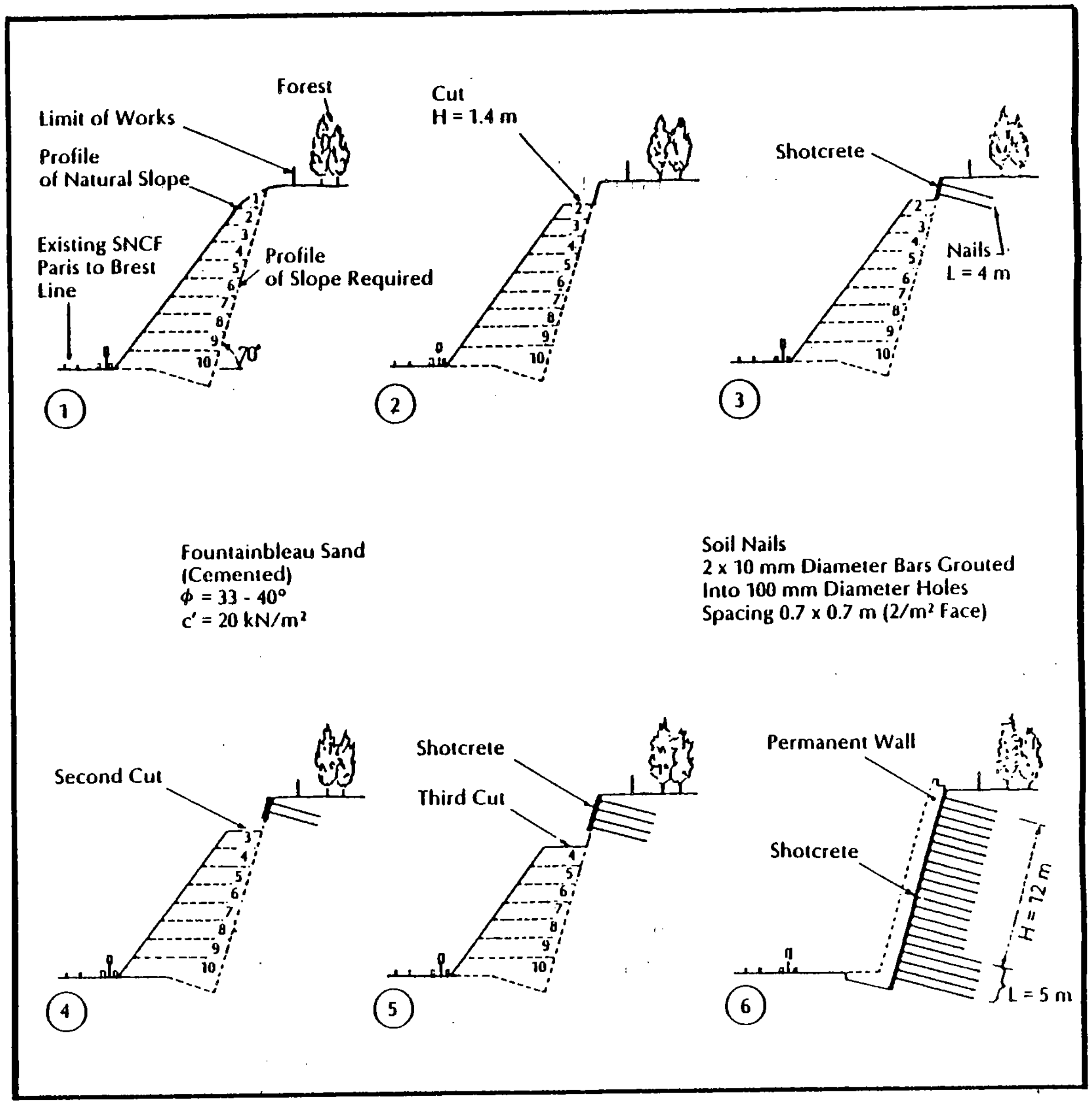


Figure 9. Standard sequence of soil nail construction, as used at Versailles, France.

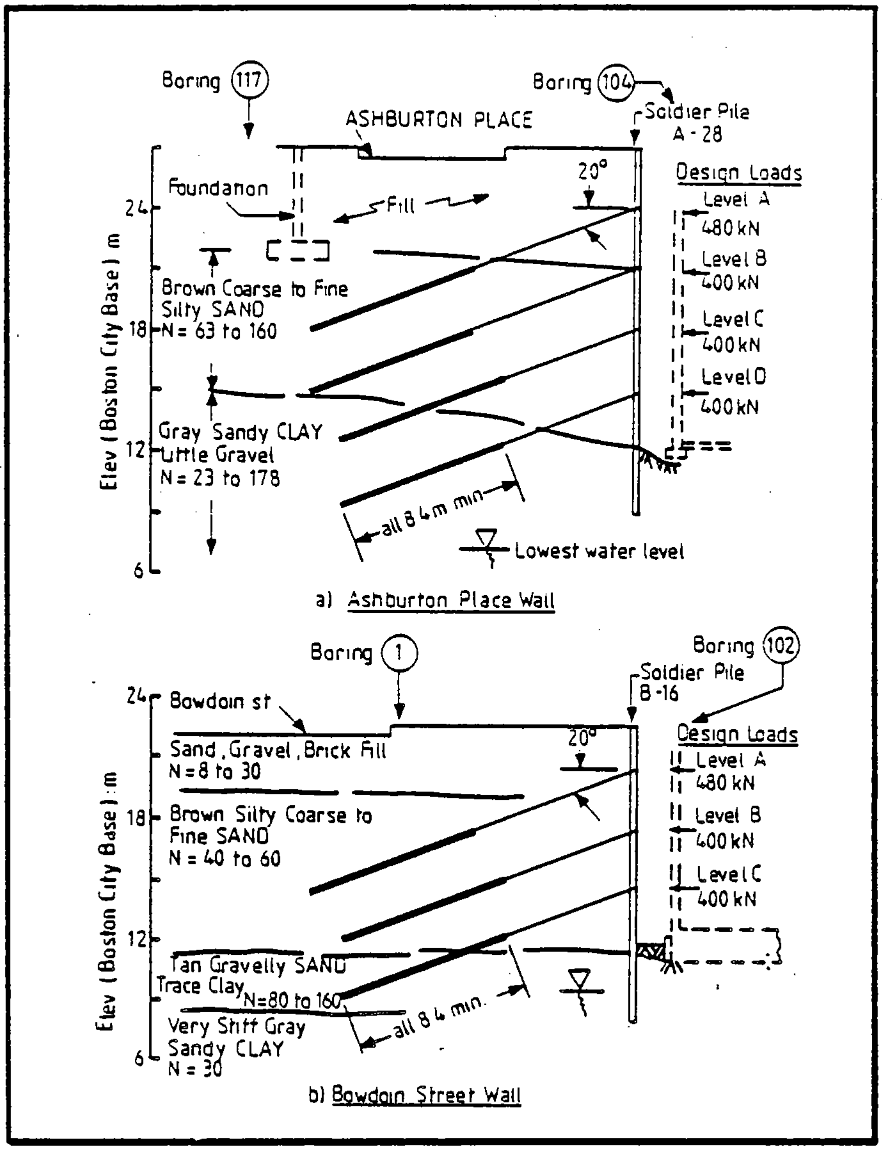


Figure 10. Typical anchored retaining wall, Boston (Oosterbaan and Gifford, 1972)

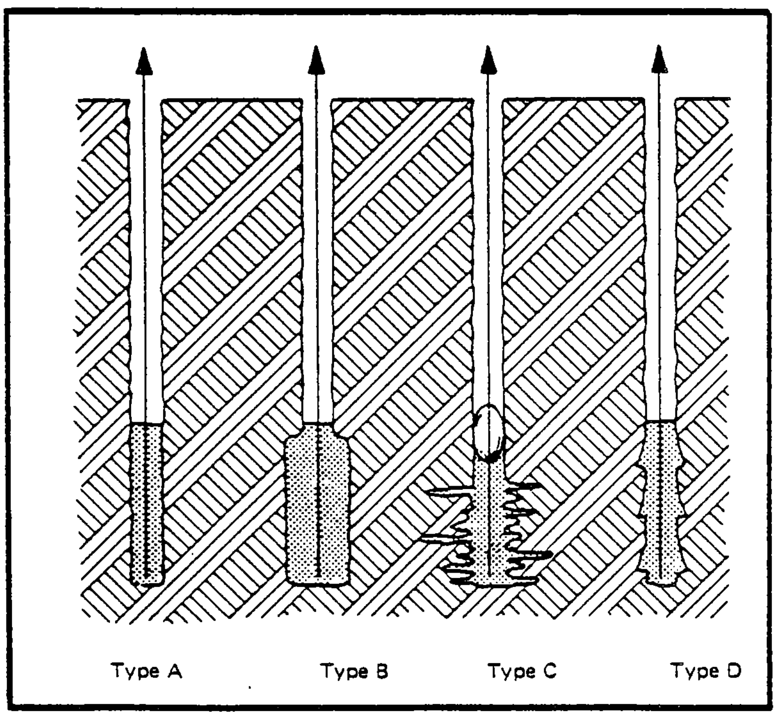


Figure 11. Main types of cement grouted anchorages (B.S.C.P. 8081. 1988)

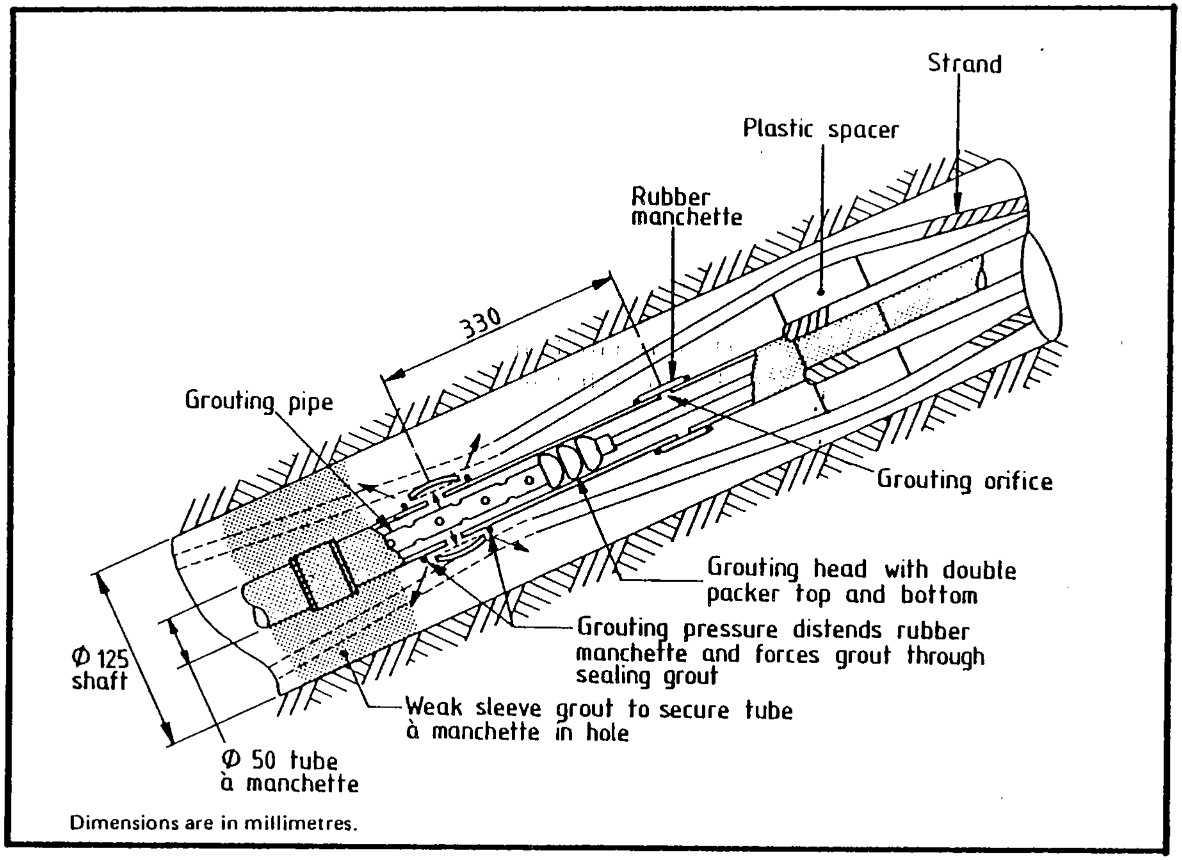


Figure 12. Detail of sleeved tube used in post-grouted anchorages (B.S.C.P. 8081, 1988)

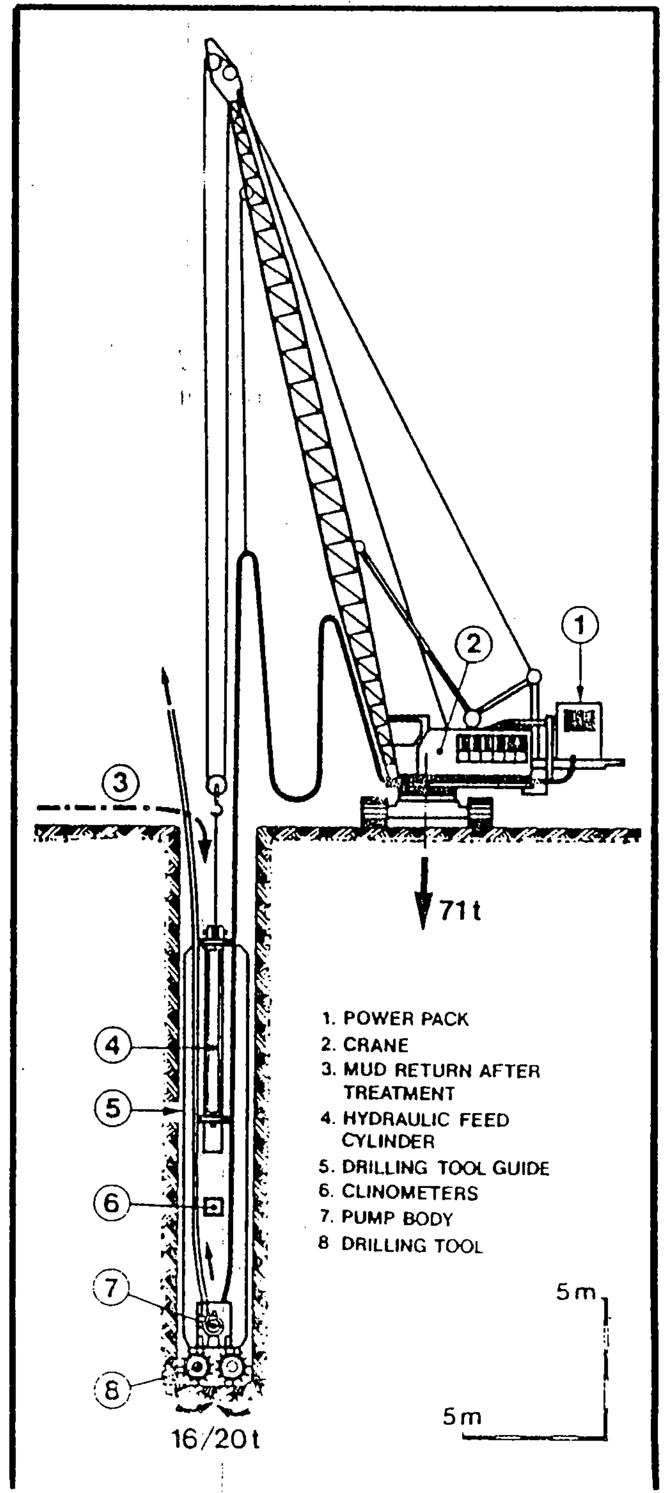
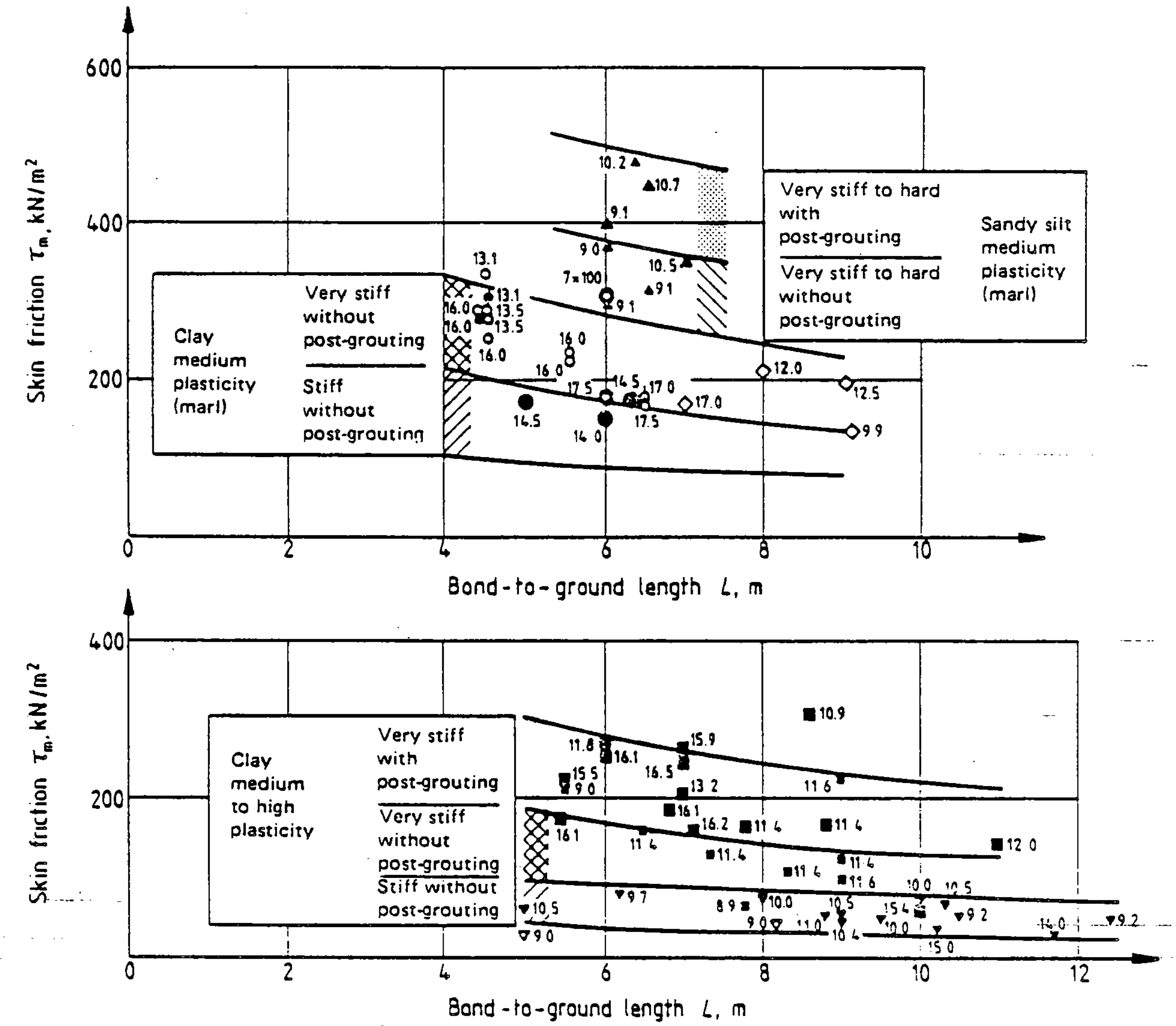


Figure 14. Principle of operation of the hydrofraise (hydromill) excavator. (Evers and Hovart, 1988)



Failure load was reached	Failure load was not reached	Post-grouting	Type of soil	L_L %	P_L %	I_c %
▲	△	Without	Silt, very sandy (marl) medium plasticity	~45	~22	~1.25
▲	△	With				
●	○	Without	Clay (marl) medium plasticity	32 to 45	14 to 25	1.03 to 1.14
●	○	With				
●	○	Without		36 to 45	14 to 17	1.3 to 1.5
●	○	With				
	◇	Without	Silt medium plasticity	23 to 28	5 to 11	0.7 to 0.85
■		Without	Clay medium to high plasticity	48 to 58	23 to 35	1.1 to 1.2
■		With				
▼		Without		45 to 59	16 to 32	0.8 to 1.0

Figure 13. Skin friction in cohesive soils for various fixed anchor lengths, with and without post-grouting (Ostermayer, 1974)

A	2.36	2.36	2.36	2.36	2.36
B	2.40	2.40	2.40	2.40	2.40
C	0.59	0.76	0.96	1.16	1.46
D	0.63	0.80	1.00	1.20	1.50
E	15.00	15.00	15.00	15.00	15.00

METRES

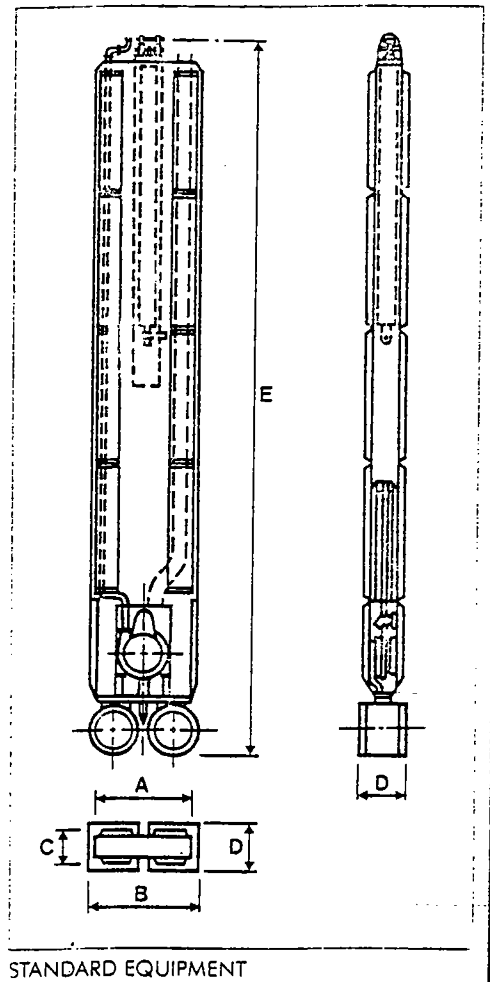


Figure 15. Standard dimensions for hydrofraise (hydromill) excavation. (Soletanche, 1987)

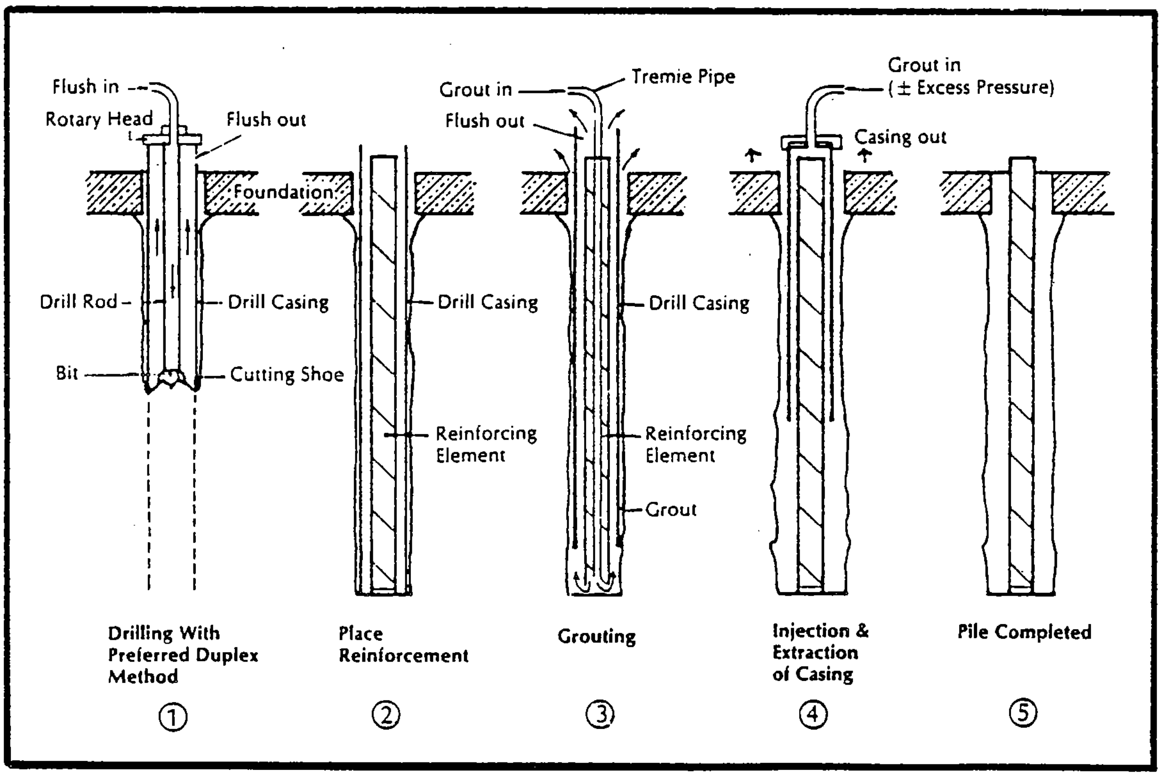


Figure 16. Standard sequence of minipile construction (After Koreck, 1978).

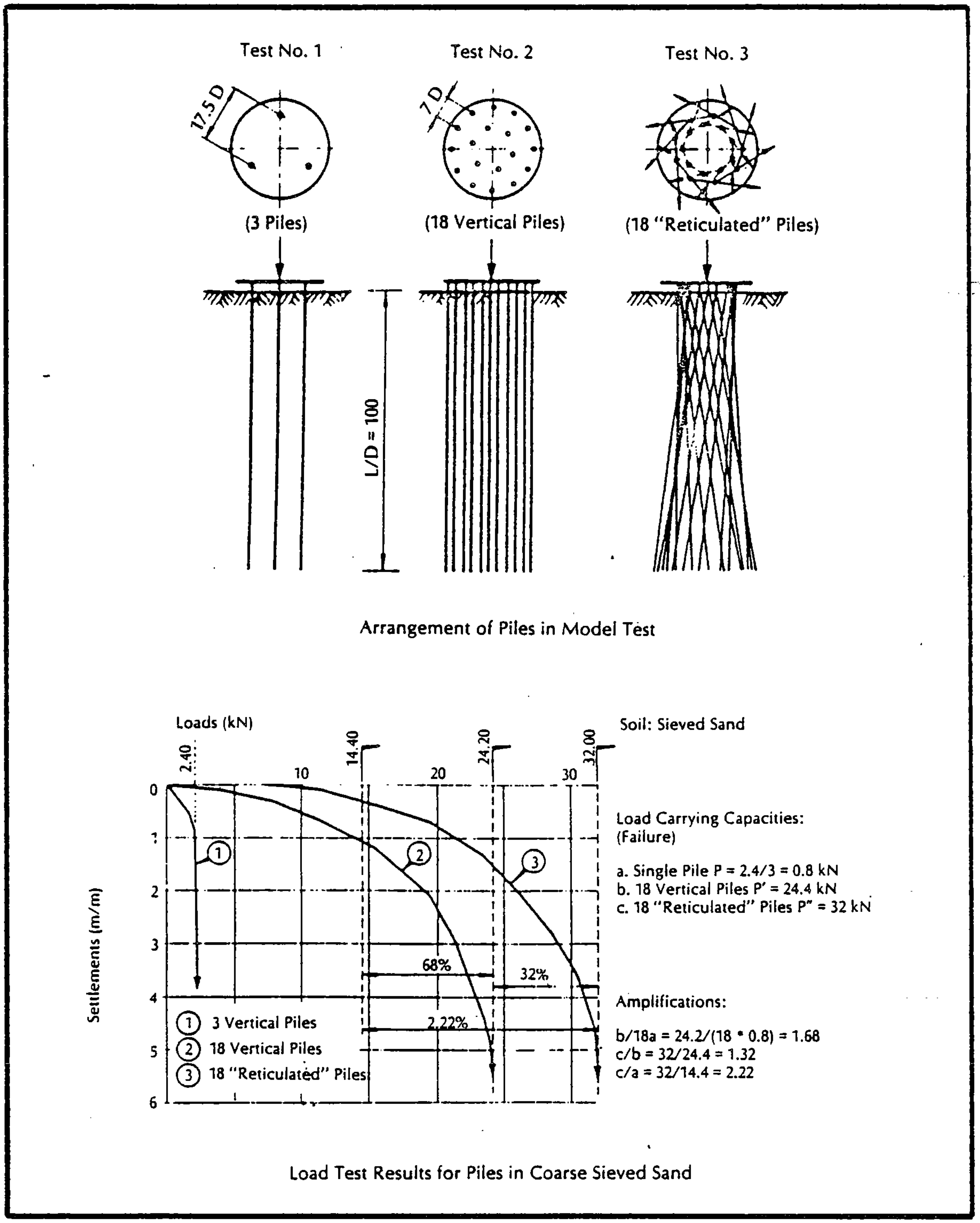


Figure 17. Model test data for different minipile arrangements in coarse sieved sand (Lizzi, 1978).

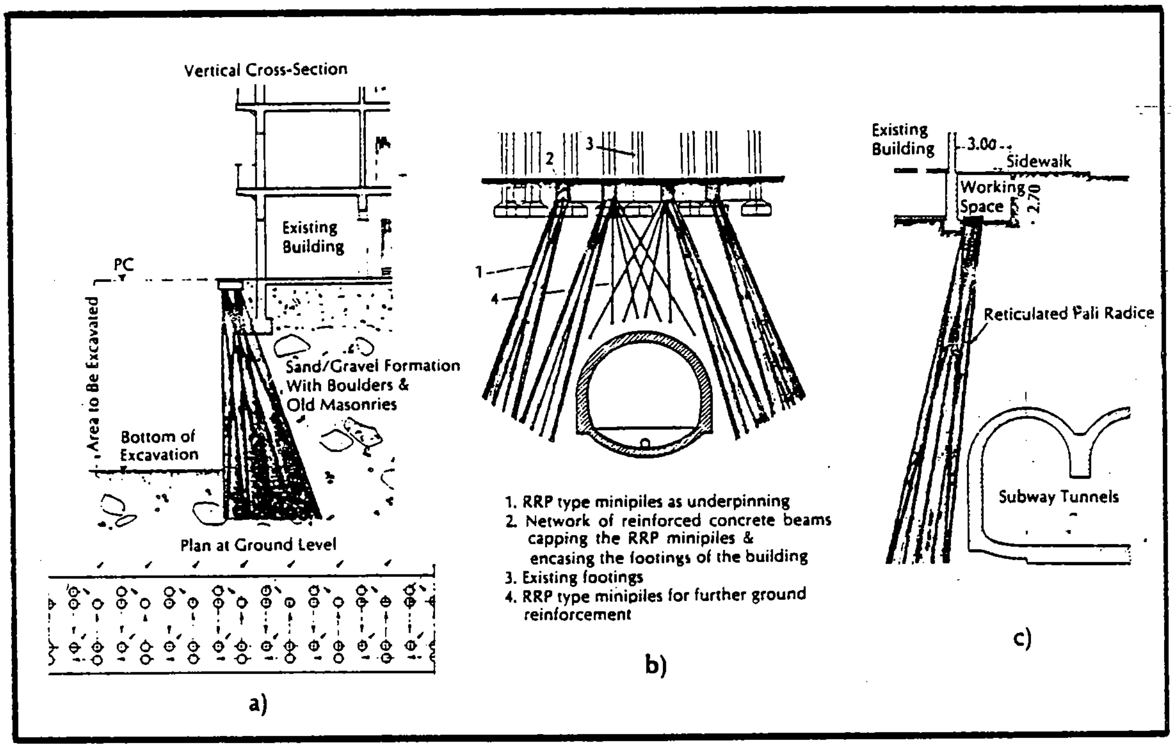


Figure 18. Applications of reticulated micropiles as used for in situ reinforcement: a) for cut and cover excavation; and, b) and c) around bored tunnels. (After Lizzi, 1982)

PROJECT EVALUATION

OF

FILL SLOPE REPAIR USING SOIL BIOENGINEERING SYSTEMS

NC 126, BURKE - MCDOWELL COUNTIES

NORTH CAROLINA

State Project: 8.1850901 (R-2029)
F.A. Project: SR 1991 (6)

Report prepared by
Geotechnical Unit, NC DOT and
Soil Bioengineering Corporation
Marietta, Georgia

ABSTRACT

Soil bioengineering is the practice of applying living vegetation, primarily cut woody plant material, as the major structural component in land stabilization. The live plants or cuttings may be used in conjunction with inanimate structural members such as wood, stone or synthetic fabrics. Soil bioengineering systems function immediately upon installation to reinforce the soil and act as barriers to surface erosion. As the plants develop a root system, the overall shear strength of the unit is increased. Excess moisture in the soil is removed by transpiration. With continued growth, the plants improve the environment for the natural invasion of the surrounding plant community, and provide a renewable source of plants for future local soil bioengineering projects.

In March, 1984, the North Carolina Division of Highways' Geotechnical Unit contracted with Soil Bioengineering Corporation to develop a Preliminary Reconnaissance Report for soil bioengineering solutions at ten sites typical of the shallow landslide and erosion problems that are widespread along North Carolina highways. In December 1985, it was decided to establish a demonstration project to repair a fill slope on highway NC 126 in McDowell County using soil bioengineering technology. The Federal Highway Administration (FHWA) showed considerable interest in this technique and encouraged the Division of Highways to pursue this repair work.

The demonstration site selected was a 870 foot long fill slope with a maximum height of 60 feet, that was built in 1980 - 1981 on a slope ratio varying from 1.5 : 1 to 2 : 1 (H : V). Soil Bioengineering Corporation was contracted to develop the construction plans and specifications, and provide on-site consultation and instruction during the construction phase. Several different soil bioengineering systems were designed into the project to satisfy the slope conditions. Systems employed in the design were: cut brushlayers, fill brushlayers, live fascines, live stakes and live cribwall.

Project construction began on October 27, 1987, with N.C. Division of Highways' Maintenance Unit providing the labor. The soil bioengineering technique is a labor intensive operation requiring the use of hand tools in both the harvesting and installation processes. Installation must be accomplished during the dormant season. Although several plant species were used, black willow (Salix nigra) was the best suited to the site environment. Planting was completed on April 2, 1987. Growth in the spring and summer of 1987 was spectacular. Plant shoots from new growth exceeded 8 feet in length with stems as much as 1 1/4 inches in diameter. The entire slope is presently very well vegetated, stable and performing as expected.

Introduction

North Carolina has been plagued with numerous shallow landslides, slumps, sloughing and erosion on both cut and fill slopes throughout the state. In March 1984, the Division of Highways' Geotechnical Unit contracted with Soil Bioengineering Corporation to develop a Preliminary Reconnaissance Report for soil bioengineering solutions at ten sites in North Carolina typical of the shallow slide and erosion problems. Soil bioengineering is the practice of applying living vegetation singularly or in conjunction with inanimate structural members such as wood, stone or synthetic materials, to stabilize earth structures. The soil bioengineering systems function immediately as soil reinforcing units and as barriers to surface erosion. In time, roots and shoots develop, forming an additionally strengthened earth reinforcing matrix within the soil mantle itself, and a vegetative cover. The roots or fibrous inclusions can greatly increase the shear strength of soils and the resistance to sliding. The systems grow stronger with age, they encourage the natural invasion of a diverse and stable plant community and cause the land to become as self-supporting as possible.

In December 1985, after reviewing the Consultant's Preliminary Report, it was decided to establish a demonstration project to repair a fill slide on NC 126, using soil bioengineering technology. The F.H.W.A showed considerable interest in this technique and encouraged the D.O.H. to pursue this repair work. In May 1985, an F.A. Rural Secondary project was programmed to proceed with the repair work.

Project Site

The demonstration site was an approximately eight hundred and seventy (870) foot long fill slope on NC 126. Generally, the east-west running slope has a southern exposure. The maximum height was approximately sixty (60) feet, with a one hundred (100) to one hundred twenty (120) foot face. This fill section was constructed in 1980-1981 with the slope varying from 1.5:1 to 2:1. The slope began to slide and erode soon after construction and was repaired several times. At the time a decision was made to repair the slope, sliding and erosion had progressed to the point of undermining the guard rail posts at several locations and encroaching on the pavement structure.

Plan Development

Soil Bioengineering Corporation of Marietta, Georgia was contracted to develop preliminary construction plans, procedures and specifications for the project. The Design Contract was to be accomplished in two (2) phases. Phase I was the "Design Report" submitted April 29, 1986, which defined and evaluated the site from a soil bioengineering systems viewpoint and compared the cost to an appropriate conventional repair system. The report describes and illustrates each soil bioengineering system and its intended function to stabilize the slope. The document also provided estimated quantities and cost for the proposed repair work.

The "Construction Document", Phase II, submitted June 1986, included a standard set of construction plans. These served as the guide for project construction. The document included: scope of work, definitions of terms, descriptions of various soil bioengineering systems, plant harvesting, installation procedures and detailed drawings of the soil bioengineering systems. The site was divided into three major areas with particular soil bioengineering systems to be installed in each area. The following is a brief description of the areas and the soil bioengineering systems installed:

Area 1 (Station 18+50 to 20+50)

The slope at the western end is 2:1 with a maximum height of approximately forty (40) feet, and is moderately stable except for a circular slump. This area also had a twelve (12) to twenty-four (24) inch scarp, in the lower portion of the slope face between Station 19+00 and 19+60.

Soil bioengineering systems installed included live staking, live cribwall, cut brushlayers and live fascines.

Area 2 (Station 20+50 to 24+00)

The slope is 1.5:1 with a maximum height of approximately sixty (60) feet. It was very heavily eroded with signs of previous sliding.

Soil bioengineering systems installed were fill brushlayers, reinforced brushlayers, live staking and rooted plants.

<u>Area 3</u> (Station 24+00 to 27+20)

The slope at the eastern extremity is 1.5:1 to 2:1 with a maximum height of twenty-five (25) feet. This area had cracks in the upper slope that paralleled the roadway, approximately three (3) to five (5) feet outside the guardrail and one (1) foot deep. The toe of the slope is fortified with natural existing tree vegetation.

Soil bioengineering systems installed were cut brushlayer, live fascine and live staking.

Phase III of the project was the construction phase. Soil Bioengineering Corporation provided the on-site consultation and instruction. All work was to be accomplished using state forces from Division 13 Maintenance and Landscape Units.

Pre-Construction

A pre-construction meeting was held on October 20, 1986 at the NC DOT, 13th Division Office in Asheville. Present at the meeting were supervisory personnel from the 13th Division, District, Landscape Unit, Geotechnical Unit, and Soil Bioengineering Corporation. Preliminary work included constructing a haul road to the toe of the embankment, installing erosion control units, improving drainage and removing loose saturated soil from the toe of the slope. This work began on October 22, 1986, and was to be performed six (6) days per week with the projected project completion date of January 16, 1987. A teaching slide presentation was made to all supervisory personnel. The intent of this presentation was to describe in detail the various soil bioengineering systems and preparation techniques for the individual installations.

Construction

Actual soil bioengineering production work began on October 27, 1986. The construction period was often hampered by rains. Harvesting of the cut plant material began on Monday, October 27, and by Thursday a sufficient quantity of cuttings was available for placement of the first fill brushlayer terraces in Area 2.

The project was scheduled to be completed by January 16, 1987. Working a six (6) day work week, less holidays, provided sixty-six (66) working days for completion of the project. Generally limited work was accomplished on Saturdays. The time estimate was based on using thirty (30) employees. The completion date of January 16, 1987 was not met. Due to poor weather conditions, only thirty-eight (38) working days were used by that date. Planting was finally completed on April 2, 1987, which was seventy-five (75) days from the first day of planting, Thursday, October 30, 1986. Work after April 2 consisted of conventional operations, such as paving the shoulder, replacing the guard rail and curbing installation. These activities required four (4) working days.

The soil bioengineering technique is a labor intensive operation that requires the use of hand tools in both the harvesting and installation process. Production was slowed several times because the Maintenance Unit had to perform its normal functions, such as snow and ice removal, guard rail repair, clearing fallen trees or removing slide debris from roadways. The project would have been completed more efficiently if all personnel working on the project had not also had other work responsibilities.

The average number of daily workers, including on-site supervisory personnel, was approximately twenty-four (24), but it varied between twelve (12) and thirty-five (35) for most of the construction period.

Harvesting-related activities required approximately half of the man-hours consumed. These activities included locating harvesting sites, securing permission to harvest, sometimes improving access to the harvesting site, the actual harvesting of the plant stems and transportation to the construction site. Harvesting sites were a considerable distance from the project and as sites became depleted, new sites were located.

Installation related activities were also quite labor intensive. While a small John Deere bulldozer was used to prepare the fill brushlayer terrace lifts in Area 2, terraces for cut brushlayers and live fascine trenches were dug with shovels.

Typically, plant material delivered to the site consisted of bundles containing long live stems or brush. The average truck load contained forty (40) bundles. Using these estimates, the quantities of plants installed in each area are given below. Additional cuttings were needed to correct brushlayers in Area 1 and 3 that were incorrectly installed.

	Length		Bundles
Area 1			
Cut Brushlayers	600 Lin. Ft.	0.7 bundles/Lin. Ft.	420
Live Fascines	570 Lin. Ft.	0.2 bundles/Lin. Ft.	115
Live Stakes	1,500 stakes estimated		
Live Cribwall	40' long	8 layers - stepped	80
Area 2			
Fill Brushlayers	5,483 Lin. Ft.	1.36 bundles/Lin. Ft.	7,450
Rooted Plants	3,000		
Live Stakes	2,500 stakes estimated		
Area 3			
Cut Brushlayers	1,315 Lin. Ft.	0.7 bundles/Lin. Ft.	920
Live Fascine	1,514 Lin. Ft.	0.2 bundles/Lin. Ft.	300
Live Stakes	3,000 stakes estimated		

In addition to the above live stems or brush, quantities of lime, fertilizer, grass seed and jute mesh were used.

Selected fill brushlayers were reinforced, using a six and one-half (6.5) foot wide layer of Tensar Geogrid on the terrace before placement of the plant cuttings. The large openings in the Tensar should allow for root development through the openings and into the soil. This added additional reinforcement to the soil in the fill slope.

Site Analysis - 6 Months

The site looks very good overall with the living cut plants predominantly of willow. It was difficult to determine the different species on the site, due to the dense growth of vegetation, which made it difficult to walk on the slope and evaluate. The cut brushlayers in Area 2 are growing very well and have the best appearance overall. Growth is uniform and dense. Alder, dogwood and privet were also observed at this time.

All species of rooted plants, especially the black locust, are growing very well with some plants now over eight (8) feet tall. Most of the grasses, except for the clover, appear to have died out.

In Area 1, near the top of the slope, there is a noticeable yellowing of the leaves on several black willow plants and scattered others; less than a dozen total have died. This is probably the result of below normal rainfall this summer, which caused some drying in the upper part of the slope in Area 1. Plant embedment length in this section is short compared to the fill brushlayers in Area 2 that are performing so well.

Plants in Area 3 are also growing well. No dead or dying plants were observed and, while growth is not as pronounced as in Area 2, it is better than in Area 1, which was poorly installed.

Some black willow growth from live stakes now exceeds eight (8) feet in height and one and one-quarter (1.25) inches in diameter at the base.

System Evaluation

All of the soil bioengineering systems employed could be used to control erosion and to stabilize heavily eroding slopes. An analysis to determine the proper system to employ in a given situation is most critical. Equally critical is the proper installation of the system. Even the simplest system, live staking, can easily be installed incorrectly. For example, live stakes must be tamped to the proper depth with the growing tip facing upward.

Live Staking System

This is the simplest and least expensive system to install and should be employed on a slope before erosion problems start. It should be part of a routine slope maintenance program.

Brushlayering System

This includes both brushlayers cut into a slope and used in a conventional fill situation where additional soil must be added to the slope face. This system works very well as porous filter units to control surface erosion from heavy rains during and after construction. This was observed during construction when four and one-half (4.5) inches of rain fell in a three (3) day period. The brushlayers prevented erosion of the loosely compacted fill material placed in the fill brushlayer system in Area 2. Areas of the slope where systems had not been installed experienced large soil flows and earth failures.

Installation of the brushlayering system requires considerably more careful planning than does live staking. The application is quite different and more complicated but offers immediate soil reinforcement and surface protection.

Live Fascine System

This system serves as a pole drain immediately after installation to control and direct surface runoff. It is useful to prevent erosion at specific locations. The root system which develops from the live fascine serves to strengthen the slope by placing roots or fibrous inclusions in the soil mantle. These act as reinforcements in arching and buttressing units. Rooted live fascines also work well to stop headcutting up the face of a slope.

Live Cribwall System

This is a very site specific system that requires considerable planning. Installation is somewhat more complicated than with the other systems. Under certain conditions, this system could replace a conventional cribwall. It is useful in areas where space is limited and where immediate structural stability is required. The living root, placed in the interior fill of the wall, consolidates the soil particles. The parts of the living system are also intended to root in the soil mantle behind the unit. This enables the live cribwall to become part of the land.

Conclusions and Recommendations

Based on the past year's growth of the soil bioengineering systems and the apparent stability of the embankment, the demonstration project described herein can be considered a successful slope repair.

The soil bioengineering systems installed on this project each served to reinforce the soil mantle and provide permanent stabilization of the fill slope.

Soil bioengineering provides an excellent method of repair for many shallow landslide and erosional slope problems. It can be especially useful in areas where access to heavy equipment is difficult. Many slopes can be stabilized before this becomes necessary. Large projects such as the NC 126 repair would require detailed plans and specifications, cross sections, quantities and contract special provisions. They should therefore be contracted to a general contractor. Using soil bioengineering technology in new projects could prevent future failures. Small projects can be handled by properly trained maintenance forces and should become a routine maintenance procedure.

There are a number of factors that should be considered regarding a soil bioengineering land stabilization project:

1. A soil bioengineering project is very labor intensive. There is considerable handwork, requiring use of shovels, mauls, chainsaws and picks. This may be an asset on sites where equipment cannot access. Large fill projects, however, do require conventional equipment.

2. The live plant material should only be installed during the dormant season, usually November through April. This is the season of snow and rain which can cause a loss of project time. It is also a time when labor is most plentiful and therefore the least expensive.

3. A readily available source of biotechnically capable plant material is especially important. The plant material should be easily accessible and available in large quantities. As projects are completed, the locations can become harvesting sites for new projects. If planned in advance, the sites can be cut the year before, to increase productivity.

4. Personnel working on installation of a soil bioengineering system must be given instruction and must be supervised very closely during the installation. They should be made to feel confident about the systems' success.

5. It is imperative to have a project designed and planned by a trained, experienced soil bioengineer.

6. The soil bioengineering systems blend naturally into the landscape and do not intrude visually, which make them highly environmentally compatible.

7. The effectiveness of a soil bioengineering project actually improves with time. Once the vegetation is established, it becomes self-repairing through constant regeneration. This leads to low maintenance requirements.

8. A properly designed, well planned and constructed soil bioengineering project has shown in many instances to be more cost-effective than conventional approaches.

9. Soil bioengineering systems produce products and offer capabilities which generally are not available in conventional engineering solutions.

Photo 1. Close-up view of site prior to soil bioengineering construction; summer 1986

Photo 2. Overview of the slope, summer 1986

Photo 3. Area 1: slump failure - live cribwall construction; January 1987

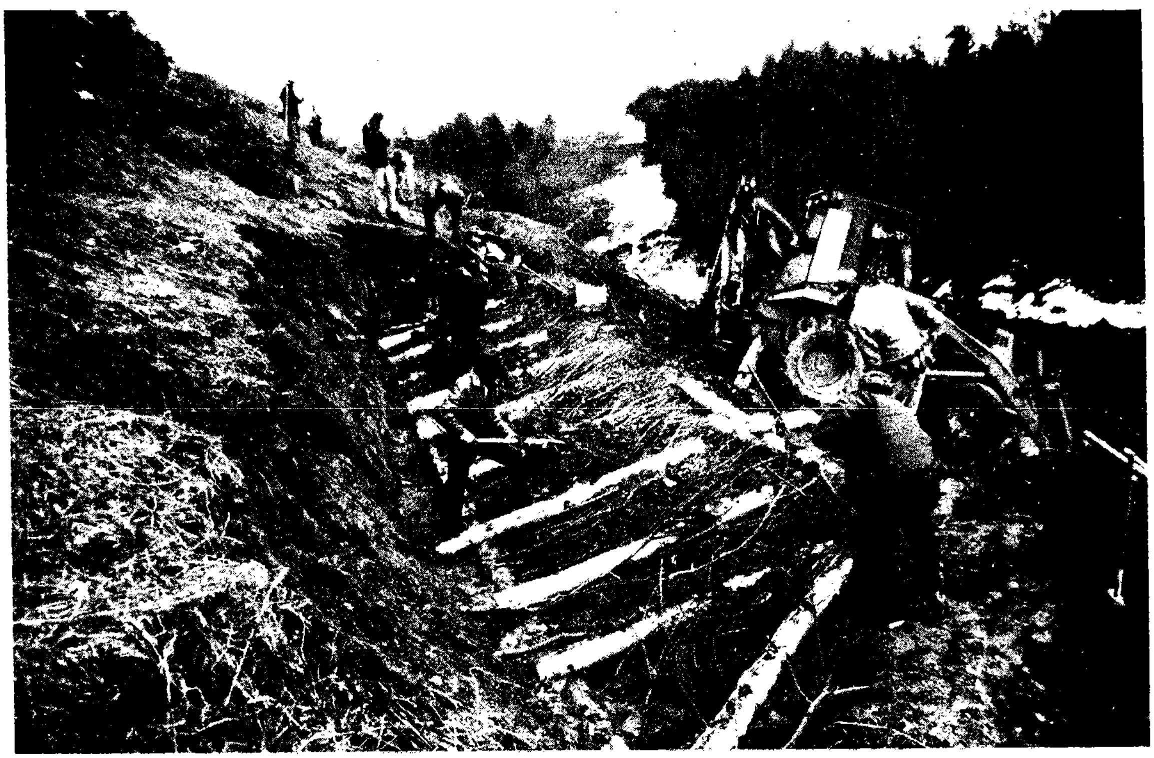


Photo 4. Close-up of live cribwall construction

Photo 5. Repaired live cribwall construction; July 1987

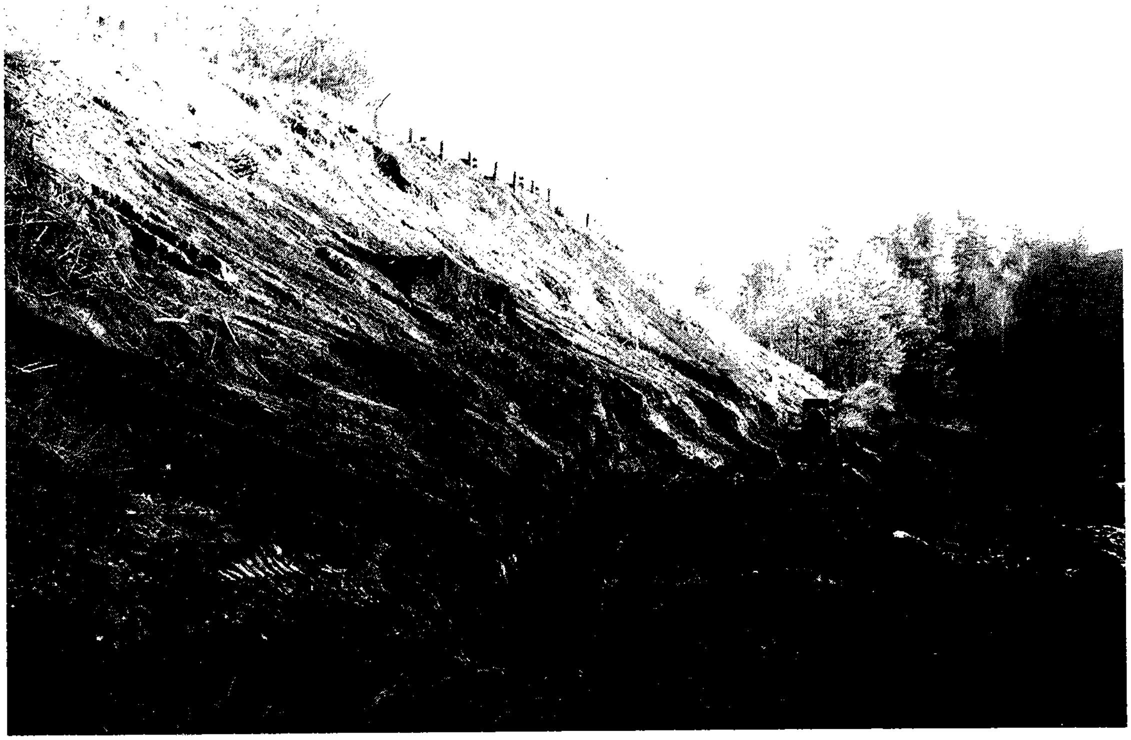


Photo 6. Area 1: Prior to cut brushlayer installation; January 1987

Photo 7. Area 1: July 1987, four months after construction - a stable soil bioengineering site

Photo 8. Area 2: February 1987, during fill brushlayer construction

Photo 9. Area 2: March 1987, completed soil bio-engineering construction

Photo 10. Area 2: May 1987, demonstrating two (2) months of stabilizing growth

Photo 11. Area 2: June 1988, one (1) year after construction of soil bioengineering systems for stabilization

STABILITY INVESTIGATION OF MT. CARMEL TUNNEL BY PHYSICAL AND FINITE ELEMENT MODELS

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ABSTRACT

Mt. Carmel Tunnel in Zion National Park, Springdale, Utah, has experienced structural problems since the initial bore was completed in 1930. Most of the distress can be related to the tunnel's location just inside of and parallel to a 600-ft-high cliff of Navajo sandstone. The mechanics of failure in the tunnel were studied by means of physical models using Navajo sandstone and by finite element analysis.

Two large sandstone block models were tested under vertical compression to simulate the state of stress in the vicinity of the Mt. Carmel Tunnel. The arch-shaped tunnel opening was located one tunnel diameter from the boundary of the sandstone block, thereby simulating the close proximity of the Mt. Carmel Tunnel to the cliff face. Cracking initiated near the corners of the tunnel where the walls intersect the flat invert, and in the crown and at the springline. Cracks extended two tunnel diameters above and below the tunnel before failure. The high-angle cracks isolated a tall, thin slab of rock between the tunnel and the free (cliff) face of the block. Ultimate failure occurred when this slab buckled or separated from the rock mass.

The finite element model employed a parabolic Mohr failure criterion to characterize both tensile and shear failure. It predicted the crack pattern observed in the experimental models, however the sequence of fracturing was somewhat different. The finite element model also showed good agreement with the tunnel closure history of the experimental models.

INTRODUCTION

Mt. Carmel Tunnel is a two-lane highway tunnel on State Highway 9 in Zion National Park. It is located just inside of and parallel to a 600-ft-high cliff of Navajo Sandstone (Fig. 1). Six openings or galleries to allow natural ventilation extend through the cliff wall along the tunnel. The tunnel has experienced structural problems, localized at the galleries, since the initial bore was completed in 1930.

Stability of the Mt. Carmel Tunnel has been the subject of several previous investigations (Buchele 1976, Hamilton 1977, 1978, Robinson 1980, 1985, and Wolf 1977). The distress in the tunnel is mostly a result of unfavorable joint orientation and continuity, along with natural cliff degradation (Robinson 1980). However, several investigators (Hamilton 1977, Kendorski 1978) have suggested that stress concentrations generated by the tunnel may also foster distress along sections of the main bore; especially in regions where close proximity of the tunnel to the cliff face produces high stress concentrations. This study uses physical and numerical models to investigate failure mechanisms and stresses generated by loading of an arch-shaped opening located adjacent to a free face. Numerous investigators have conducted experimental studies of tunnels (Bonsall et al. 1982, Gay 1976, Heuer and Hendron 1971, Lajtai and Lajtai 1975, Kaiser et al. 1985, Krauland 1970, and Sharma 1976), however the authors are not aware of previous rock model tests of an opening located close to a free face.

GEOLOGY

The Mt. Carmel Tunnel lies within the early Jurassic Navajo Formation, a very fine to medium grained sandstone of aeolian and shallow marine origin. Most of the tunnel is in the basal section of the Navajo which is characterized by thin planar bedding and very fine grain size (Robinson 1980).

Joints are largely responsible for the columnar blocks forming the cliffs containing the Mt. Carmel Tunnel (Fig. 2). The major joint set in the Park, and particularly in the vicinity of the tunnel, is nearly-vertical and strikes N10W to N20W. This set defines the east and west sides of the massive vertical columns (Fig. 2) present along the tunnel alignment (Robinson 1980). This major joint set includes discrete joints, joint clusters, and fracture zones. The discrete joints are tight and planar with 1/8 to 1/4-inch undulations, and 5 to 150 ft spacing. The joint clusters consist of two or more joints and often as many as 10 joints, spaced 3 inches to 3 feet apart. The fracture zones consist of two or more discrete joints spaced several inches to several feet apart, with the intervening rock broken into 1 to 12 inch fragments. The joints of this group tend to be continuous over large distances.

A minor set of regional systematic joints strikes N65E to N80E and has a near-vertical dip. This joint set contains shorter and more closely spaced joints than the major joint set and exerts less influence on topography. The minor joint set generally intersects the tunnel at angles of 15-25°. Most of the severe crown overbreaks experienced during tunnel excavation occurred along this joint set and adjoining cross-bedding planes (Robinson 1980). Joints of the minor set have a wider range of orientations than those of the major set. They are generally approximately parallel to the cliff face and are mimicked by exfoliation joints. Field relationships show the minor joint set terminating against the major joint set, indicating that the minor set postdates the major set (Robinson 1980).

Generally, exfoliation fractures in Zion National Park are oriented subparallel to the major and minor joint sets, although a number of exfoliation fractures near the tunnel dip approximately 60 degrees toward the valley. The continuity of exfoliation fractures at the site is highly variable, ranging from a few feet to several hundred feet.

DISTRESS IN TUNNEL

75% of the tunnel is currently supported with 3-6 ft wide reinforced concrete rings spaced 3-5 ft. The intervening rock is covered with gunite. The remaining 25% of the tunnel, which primarily includes the gallery areas, is supported by a continuous reinforced concrete liner (Robinson 1980).

Indications of minor rock and liner distress along the tunnel include the presence of exfoliation joints in the sidewall, spalling, hairline cracks, and the formation of precipitates. The exfoliation joints probably formed shortly after the original excavation. The spalled sections have been less than 10 square feet and generally occur where the gunite is too thin for long-term support. The hairline cracks generally follow reinforcing bars or edges of I-beams. With the exception of cracking between 50+09 and 50+15, none of these cracks appear to indicate severe distress in the liner; but may indicate minor adjustments to stresses in the cliff face (Robinson 1980).

PHYSICAL MODEL TESTING

Sandstone block models were cut with a rock saw from sandstone samples obtained in Zion National Park. The blocks were machined on a surface grinder to ensure parallelism of the loaded surfaces. A simulated arch-shaped opening was cut 29 mm away from the right lateral boundary of the block using a diamond bit core barrel. The circular profile was extended into an arch shape, using a tungsten carbide hacksaw blade to excavate the bottom corners. A square box file was used to define the corners of the opening and a 1-inch-wide flat file was used to level the tunnel floor. The height and width of the opening in all models was 31mm, with the springline located 15mm above the invert.

The loading conditions applied to the models simulate the stress state existing at the Mt. Carmel Tunnel, however true similitude is not achieved because of differences in loading rate, moisture content, and size effect. Loading on the tunnel is caused mainly by vertical overburden pressure. Horizontal stresses can be neglected because the tunnel is parallel to and close to a cliff face. The three-dimensional model representing a section of tunnel was loaded in uniaxial compression perpendicular to bedding. To minimize end restraint effects, the model was compressed between aluminum end pieces with the same section as that of the model. The ratio of Poisson's ratio to elastic modulus for aluminum is close to that for the sandstone.

Locations of strain rosettes used to monitor changes in strain around the opening are shown in Fig. 3. A calibrated Instron extensometer was used to measure convergence of the model opening. A multichannel computer-controlled data acquisition system was used for data collection.

The opening was located in the right side of the model as viewed from the front face. The right side of the model, where the opening is closest to the edge of the block (cliff), is called the free face. All descriptions of cracking are based on a front view of the model. The strain and crack orientations on the back face are described as though viewed from the front face to maintain a consistent frame of reference.

Stresses were calculated from measured strains at several stages of loading. The stresses adjacent to the sidewalls of the tunnel were calculated using the elastic parameters obtained by compression testing of cores. However, the crown and invert areas experienced a mixed stress state with both tensile and compressive components. Elastic modulus is generally much lower in tension than compression and based on data compiled by Haimson and Tharp (1973) for sandstones, tensile Young's modulus E_t was assumed to be 1/20 of the modulus in compression E_c. The tensile Poisson's ratio was calculated using the relationship:

$$\nu_c / E_c = \nu_t / E_t$$

which is required by the theory of anisotropic elasticity.

MODEL ONE

Crack patterns have been mapped on both the front (non-strain gaged) and back (strain gaged) faces (Fig. 4). It was not possible to correlate crack location and sequence on the front face of the model with that on the back face because load-crack data were not recorded during testing on the strain gaged face where most cracks were obscured by gages, leads and other instrumentation. For that reason cracks observed on the front and back faces are discussed separately. In general cracks had similarly located counterparts on both faces with the exception of cracks 4g, 6g, and 7g which were seen only on the back face.

Cracking on Front Face of Model 1

The loading rate was approximately 400 psi/min for testing Model 1. The first fracture was seen at an applied stress of 2812 psi (Fig. 4). The crack (#1) did not intersect the opening when it was observed but rather propagated 67 mm in a predominantly vertical downward direction from a point located 6 mm below and to the right of the right corner of the opening.

The next cracking was observed at an applied stress of 3594 psi. Crack #1 propagated by an additional increment and a vertical crack (#2) approximately 5 mm in length developed in the crown. At the same loading level a vertical crack (#3) 2.5 mm in length developed at the invert.

At an applied stress of 4375 psi, crack #1 propagated an additional 5 mm. Crack #2 in the crown increased in length to 33.5 mm while crack #3 in the invert increased in length to 11 mm. As the load continued to rise above 4375 psi a whitish discoloration originated from the left corner of the opening and extended obliquely for a distance of 62 mm toward the base of the model. This discoloration probably represents a shear zone, but upon reaching an applied stress of 5469 psi the linear zone became an open crack (#4). With the opening of crack #4, tensile crack #3 in the invert closed.

Failure, which was defined by a sudden drop in load carrying capacity, occurred at 6600 psi. It was marked by the development of additional cracks (#1a,#1b) intersecting crack #1. Crack #1 propagated back toward the opening, intersecting the sidewall just below the springline. The sidewall in this region had spalled extensively. A crushed zone also developed at the point of initiation of crack #4 at the left corner of the opening.

Cracking on Back Face of Model 1

Strain measurements and crack locations were used to constrain the approximate load at which cracks occurred and the sequence of cracking on the back face of Model 1. The first crack (#1g) occurred in the invert at approximately 1400 psi. The tensile major principal strain was 2200 microstrain (291 psi) and the compressive minor principal strain was -500 microstrain (-1324 psi). Cracking at this load in the invert is consistent with the 300 psi tensile strength of the sandstone in the plane of the strata (Robinson 1980). As load continued to increase, a crack (#2g) propagated from the crown at approximately 2200 psi. The major and minor principal strains in the crown area at this load were 2075 microstrain (286 psi) and -650 microstrain (-1869 psi) respectively. Again, the major principal stress was close to the tensile strength of the sandstone.

The crown and invert strain rosettes recorded maximum tensile principal strains at 3600 psi and 3800 psi respectively. Continued opening of the crown (#2g) and invert (#1g) cracks was confirmed by the increasing tensile strains. The maximum tensile principal strains in the crown and invert were 3000 microstrain (488 psi) and 11400 microstrain (1897 psi) respectively. These apparent stresses are both in excess of the tensile strength of the sandstone and indicate that measured strain was probably partially accomodated by cracking.

Decreased tensile strain in the invert occurred as the load exceeded 3600 psi. This initial decrease in tensile strain is marked by a drop of 3.07 microstrain/psi of load. This decrease suggests that tensile strain may have shifted to crack #3g, which propagated nearly vertically downward from the right corner of the opening. A nearly contemporaneous decrease in tensile strain was recorded in the crown area at a load of 3800 psi. The initial decrease in tensile strain in the crown was not as great as that occurring in the invert; decreasing only 0.7 microstrain/psi of load.

At 5200 psi a major crack, #4g, propagated vertically upward from the right side of the crown. Associated with the propagation of crack #4g was a sharp decrease in tensile strain in the crown. The strain decreased at 8.5 microstrain/psi of load for a 1000 psi load increment, before decreasing more gradually. As cracks (#4g, 6g, 7g) developed on both sides of the crack in the crown (#2g) the stress free zone became more extensive until eventually the crown strain rosette became isolated from loading.

Propagation of crack #4g strongly influenced the strain field and its initiation was recorded by a number of strain rosettes. For example, there was a sharp change in the direction of the principal angle in the vicinity of the right top rosette, the minor principal compressive strain recorded by the right bottom strain rosette stabilized and remained nearly constant to failure, and there was also a noticeable increase in closure of the tunnel opening (Fig. 6) at 5200 psi.

The nearly vertical fractures (#3g and #4g) above and below the right sidewall of the opening detached a slab of rock about 30 mm wide at the right boundary of the model causing it to behave as a column under axial load. At a load of 4500 psi the right rosette located on this slab recorded a tensile major principal strain which remained tensile until a load of 5200 psi at which load it again became compressive. This tensile strain may have represented a slight outward buckling of the column.

Strain data indicate that crack #5g propagated vertically downward from the left corner of the opening at a load close to 5400 psi, which is similar to the load at which the corresponding crack on the front face (#4) propagated. Strain measurements do not provide conclusive evidence as to the exact load at which cracks #6g and 7g developed.

Cracking and Spalling Inside the Model Tunnel

The most extensive sidewall slabbing occurred along the left sidewall. High vertical (-3200 microstrain, -12,402 psi) and horizontal (-400 microstrain, -4624 psi) compressive strains recorded by the left rosette correlate with this extensive slabbing. The high vertical compressive stress adjacent to the left sidewall (-12,404 psi) was in excess of the unconfined compressive strength (7000 psi) of the sandstone. The existence of a confined state of stress at the strain rosette location was the probable reason that this high vertical compressive stress did not result in greater failure.

Sidewall slabbing was less extensive along the sidewall adjacent to the free face, where cracking occurred, and where lower minor and major principal compressive strains of -1850 microstrain (-7042 psi) and -100 microstrain (-2181 psi), respectively, were recorded.

MODEL TWO

Model 2 reached a maximum stress of 5143 psi at a loading rate of about 300 psi/min. The failure pattern was distinctly different from that observed in Model 1 (Fig. 5) and may be related to strength inhomogeneity associated with thin beds of weak, coarse grained sandstone. Failure mechanisms in Model 2 preclude separate discussions of cracking on the two model faces. Cracks other than throughgoing crack #2 were hairline cracks playing only a minor role in failure. The first fracture was a 7.2 mm slightly curved, nearly vertical crack located adjacent to the left sidewall of the opening. This crack is not labelled in Fig. 5 because the zone in which it occurred spalled under increased load.

A spalled zone in the right wall of the opening terminated just above the springline upon reaching a coarser grained layer. Shear fracturing in this coarse grained layer resulted in a crushed zone that extended horizontally for a distance of approximately 5.9 mm. This shear zone may be similar to the normal shear fractures described by Nesetova and Lajtai (1973). Normal shear fractures initiate at points of maximum compressive stress concentration near boundaries and propagate along a path that is normal to the direction of loading.

Minor spalling of the left sidewall occurrred in conjunction with the development of the crushed zone. With increased load, the high compressive stresses in the crushed zone were eventually relieved by the opening of a 40.5 mm long, slightly curved, near-vertical crack (#2) which propagated from the crushed zone.

The opening of the throughgoing vertical crack (#2) at approximately 4400 psi is documented by strain data in the crown area. The major principal tensile strain in the crown decreased from 2150 microstrain

(208 psi) just prior to propagation of crack #2, to 1700 microstrain (159 psi) after crack #2 opened. The minor principal compressive strain increased from -300 microstrain (548 psi) prior to fracture, to -450 microstrain (860 psi) after crack #2 opened. The strain data suggest that release of tensile stress in the crown and increased compressive loading of the arch were caused by propagation of crack #2.

At 4400 psi the invert gage recorded a maximum tensile principal strain of 1400 microstrain (128 psi) and a maximum compressive principal strain of -500 microstrain (-968 psi). Magnitudes of both strains then decreased nearly contemporaneously with ongoing shearing of the right sidewall and propagation of crack #2. It is unlikely that crack #2 influenced behavior in the invert. It is more probable that the decrease in strain above 4400 psi resulted from crushing of the left corner of the opening. This crushing changed the effective load carrying capacity of the rock around the opening, which effectively increased in width, decreasing stress below the invert.

Immediately prior to failure, crack #2 extended upward as a series of en echelon cracks with a geometry suggesting downward movement of the crown with respect to the sidewall. As the en echelon cracks continued to propagate, the lower left corner of the opening experienced extensive crushing, resulting in a series of thin, slightly curved vertical slabs moving toward the opening. Associated with development of the en echelon cracks and crushing of the left corner was continued relaxation of tensile strains in both the crown and invert and an increase in compressive strains adjacent to both the left and right sidewalls of the opening. Unlike Model 1, the highest compressive strains occurred between the opening and the free face. The major and minor compressive principal stresses at the right strain rosette at a load of 5100 psi were -1232 psi and -9128 psi respectively.

At higher loads the en echelon cracks joined to form a continuous extension of crack #2 that separated a column of rock above the opening from the model block. At this time, collapse of the model occurred, with intense crushing propagating horizontally outward from the right sidewall.

EXPERIMENTAL CONVERGENCE MEASUREMENTS - MODELS 1 AND 2

Tunnel closure measurements show that Model 1 exhibited less closure than Model 2 with increasing load (Fig. 6). Closure of the openings was nearly linearly proportional to load over most of the loading range, with the exception of a few small jumps in displacement that may be attributed to the opening of fractures.

FINITE ELEMENT MODEL

A plane stress finite element model (FEM) employing 349 constant strain, transversely-isotropic elements was used to simulate deformation and failure of the physical model. A uniformly distributed vertical load was applied at the top of the model to duplicate the loading of the physical models.

Young's modulus and Poisson's ratio were measured on small diameter cores oriented parallel or perpendicular to bedding. The values used were from the same rock used for model 1. For loading parallel to bedding $E = 2.0 \times 10^6$ psi and $\nu = 0.23$; for loading perpendicular to bedding $E' = 3.2 \times 10^6$ psi and $\nu' = 0.27$. Shear modulus $G' = 1.0 \times 10^6$ psi was computed by an empirical equation proposed by Amadei (1983). A parabolic Mohr failure criterion (Fu, 1982) was used to characterize both tensile and shear failure, but only tensile failure occurred in the models. After tensile failure of an element, the elastic parameters associated with the direction parallel to the maximum tensile stress are reduced to $E = 1.0$, $\nu = 0.0$, and $G' = 1.0$.

Finite Element Crack Simulation

The finite element model predicts the major cracks seen in experimental Model 1, however the sequence of fracture is slightly different (Fig. 7). The finite element model also predicted cracking at lower applied stress than obtained experimentally and loss of load-carrying capacity occurred at only 55 to 75% of the collapse load for the tunnels in the physical models.

The finite element model initially generated nearly simultaneous tensile failures in the crown and invert (Fig. 7). With increasing load, tensile failure progressed vertically from the invert. At a load of 1987 psi a zone of failed elements started extending from the bottom of the crack in the invert back toward the right corner of the opening. At the same load the crack in the crown had experienced further propagation. At 3000 psi an additional zone of failure initiated approximately one opening diameter below the centerline of the opening and propagated back toward the left corner of the opening. The finite element model shows fairly good correlation with the early closure history of experimental Model 1 (Fig. 6).

COMPARISON OF THE MODELS

The finite element program predicted fracturing at lower applied stresses than observed in the physical model. One possible reason for this is that for most rocks the slope of the stress-strain curve in tension is lower than in compression (Haimson and Tharp 1973). This may explain some of the discrepancy between applied load and fracture sequences observed in the FEM and physical models. If the finite element analysis had used a reduced Young's modulus in tension, tensile stresses would have been reduced causing fracture to occur at higher applied stress and resulting in behavior more similar to that observed in the physical models.

One marked similarity of the two physical models and the finite element model was the development of tensile fractures dipping toward the centerline of the tunnel from both corners of the invert. This cracking effectively isolates and destresses the zone below the invert.

Similarities exist between the physical model, the FEM and the elastic analysis by Mindlin (1948) for stress around a circular hole near a free face. Mindlin has shown that the free-face stress is low at the point nearest the circular hole, but increases with vertical distance away from this point, before gradually decreasing to equal the remotely applied stress.

The same behavior is seen in Fig. 8, a plot of vertical stress magnitude along the free face calculated by FEM. It is observed that the stress is low, approximately equal to the applied stress, at the point nearest the opening, and increases to 1.2 times the applied stress at a vertical distance of 1.5 times the height of the opening. This stress distribution is reflected in both physical models (Fig. 4 and 5), where fracturing near the free face (cliff) is more pronounced above and below the level of the tunnel than directly opposite the tunnel. It is also reflected in the strain measurements of Model 1 where higher vertical stresses existed at the locations of the right bottom and right top strain rosettes than at the right rosette directly opposite the opening (Fig. 3).

Mindlin's study shows that the zones of tensile and compressive tangential stress change as an opening is moved closer to a free face (cliff). The compressive zone between the opening and free face is reduced while the tensile zones in the crown and invert are increased in size. Sections of the crown and invert close to the free face (cliff) experience tensile tangential stress that would be compressive in an opening located a greater distance from the free edge. Mindlin also showed a very large increase in the compressive tangential stress between the free face (cliff) and opening as the opening is located closer to the free face (cliff). Model 2 reflects the presence of a high stress concentration near the free face (shear zone development). The larger tensile zone in the crown and invert may be responsible for the extensive vertical cracking extending from the springline in Models 1 and 2.

An important feature observed in both physical models was a fracture extending upward from the springline on the side of the tunnel near the free (cliff) face. This fracture was not observed in the FEM, but based on the timing of its appearance in Model 1, it is possible that it would have formed at higher load. Significantly, a nearly vertical crack is observed at the springline in the Gallery 3 area of Mt. Carmel Tunnel (Robinson 1980). Measurements of offset indicate that the crown is moving down relative to the sidewall. This is consistent with the above interpretation that the rock slab between the tunnel and cliff face is lightly stressed except at the tunnel sidewall, and that it may behave as a rigid monolith while failure occurs above and below.

IMPLICATIONS FOR THE MT. CARMEL TUNNEL

Behavior of the physical and finite element models corresponds in several predictable ways to behavior observed in the Mt. Carmel Tunnel. Before the Mt. Carmel Tunnel was lined, block fallouts from the roof occurred as a result of pre-existing joints and tensile stress in the crown. Carlson gages in the column and crown at station 50 + 19 also indicated the expected compression in the sidewall and tension in the crown.

The physical models indicate that distress in a tunnel near a free face may take complex forms, with stresses falling as well as rising and fracture displacements decreasing as well as increasing. Even the displacement mode may change, with early shear being replaced by later tension or vice versa.

Although the physical and numerical models imply that loading of the Mt. Carmel Tunnel is an order of magnitude below the ultimate strength, they also define the mechanics of failure in this geometry. In particular, it is suggested that failure would be preceeded by isolation of an intact rock mass between the tunnel and the free face. The existence of high-angle natural joints parallel to the tunnel might promote this development. This rock mass may extend vertically for one or several tunnel diameters above and below the centerline of the tunnel. It would remain sound directly opposite the tunnel, while experiencing cracking and loss of integrity above and below. This has interesting implications both for monitoring strategies and reinforcement design.

ACKNOWLEDGEMENTS

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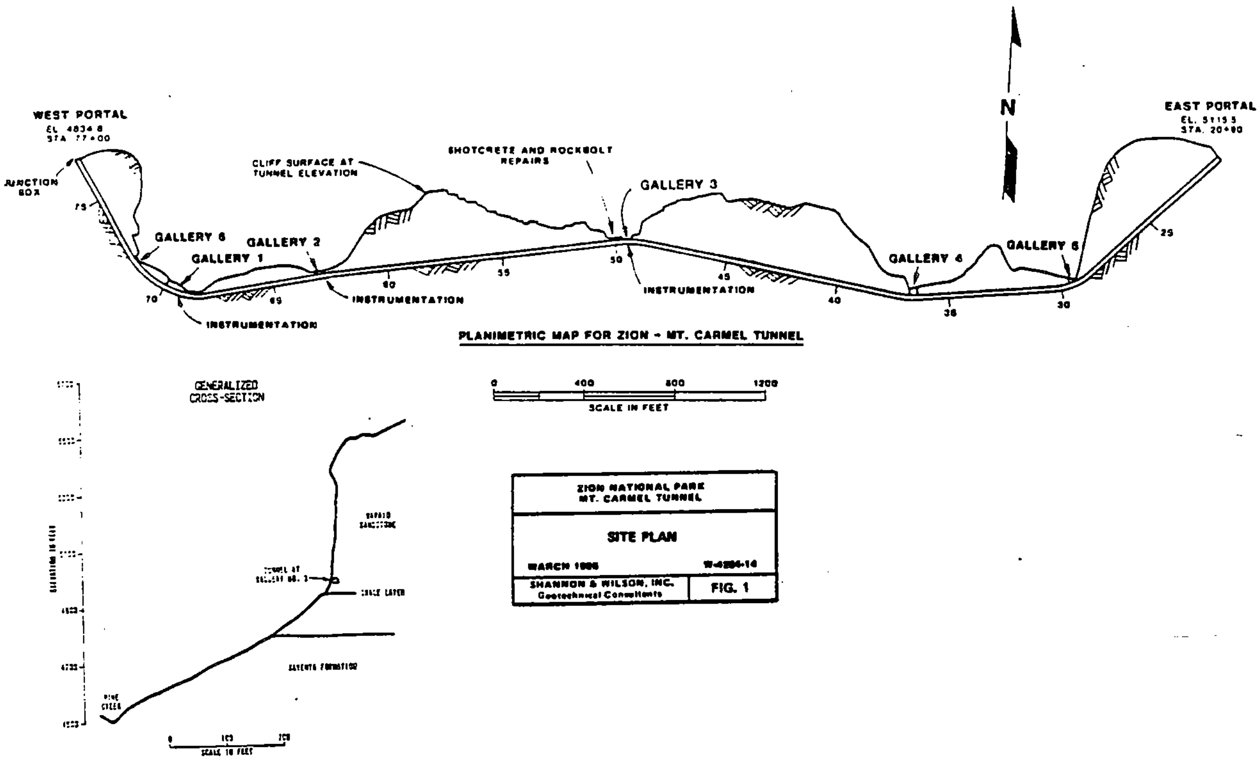

Fig. 1. Planimetric and cross-sectional veiws of Mt. Carmel Tunnel, from Robinson (1980).

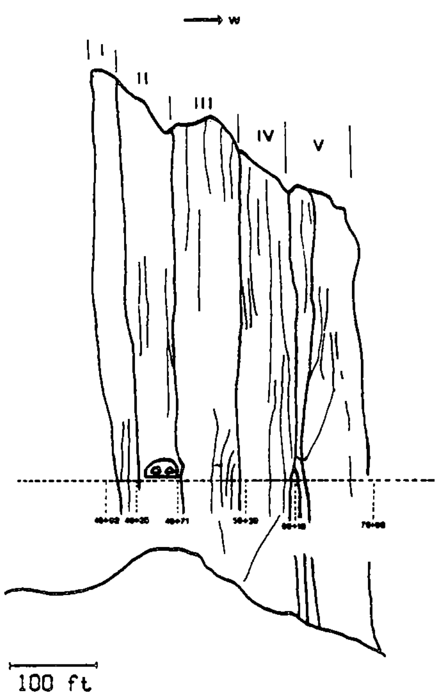

Fig. 2. Cliff face at Gallery No. 3 showing major joints.

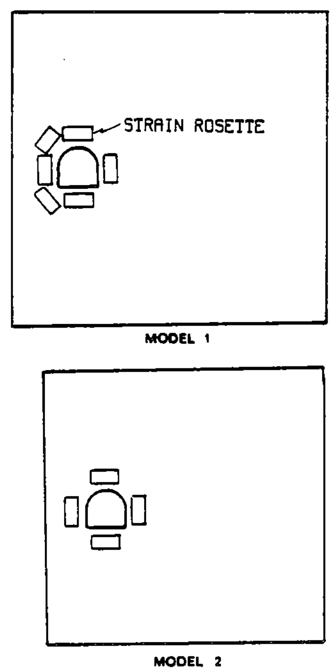

Fig. 3. Locations of strain rosettes.

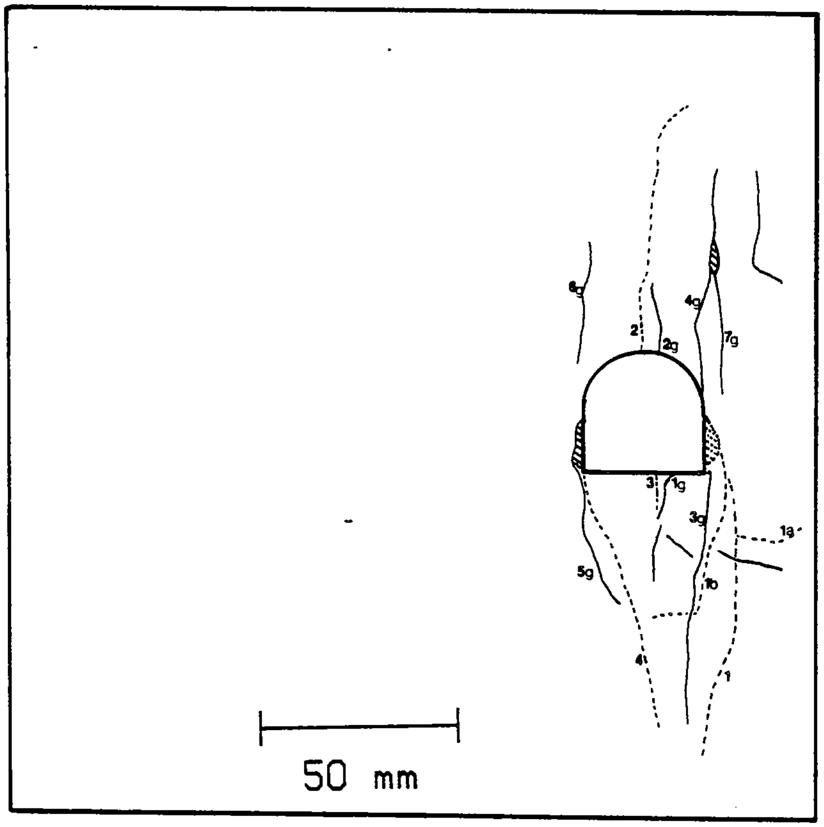

Fig. 4. Crack map of Model 1. Dotted lines represent cracks on front face, solid lines cracks on back face.

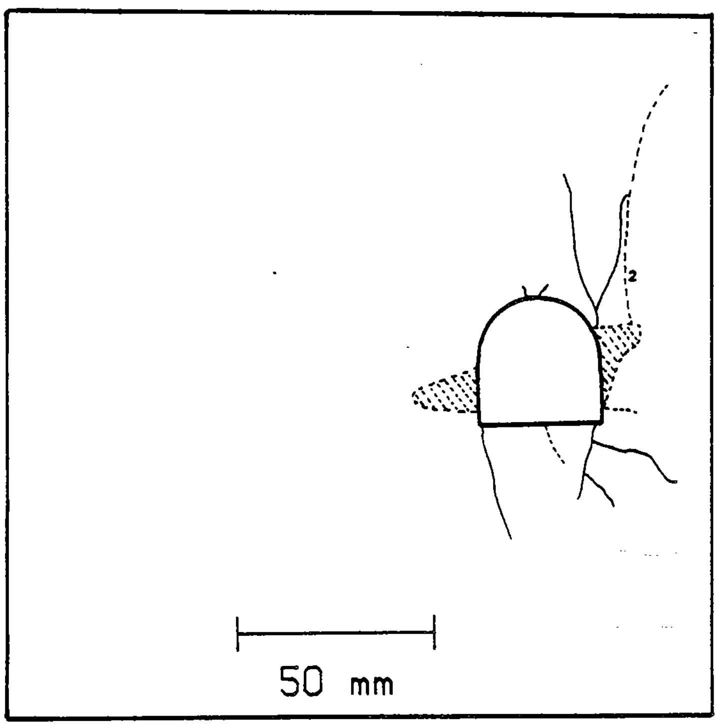

Fig. 5. Crack map of Model 2. Dotted lines represent cracks on front face, solid lines cracks on back face.

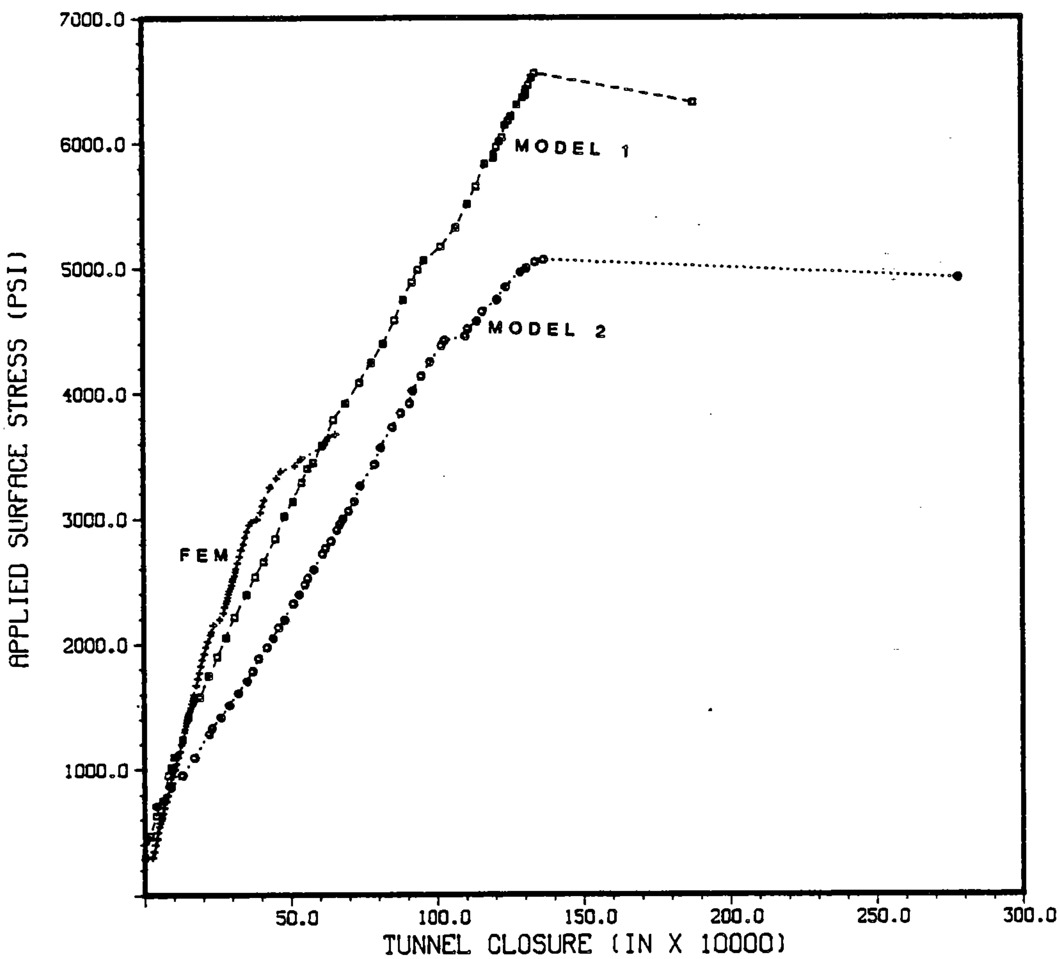

Fig. 6. Tunnel closure versus remotely applied stress for experimental and finite element (FEM) models. Finite element closure is based on elastic parameters appropriate for Model 1 rock.

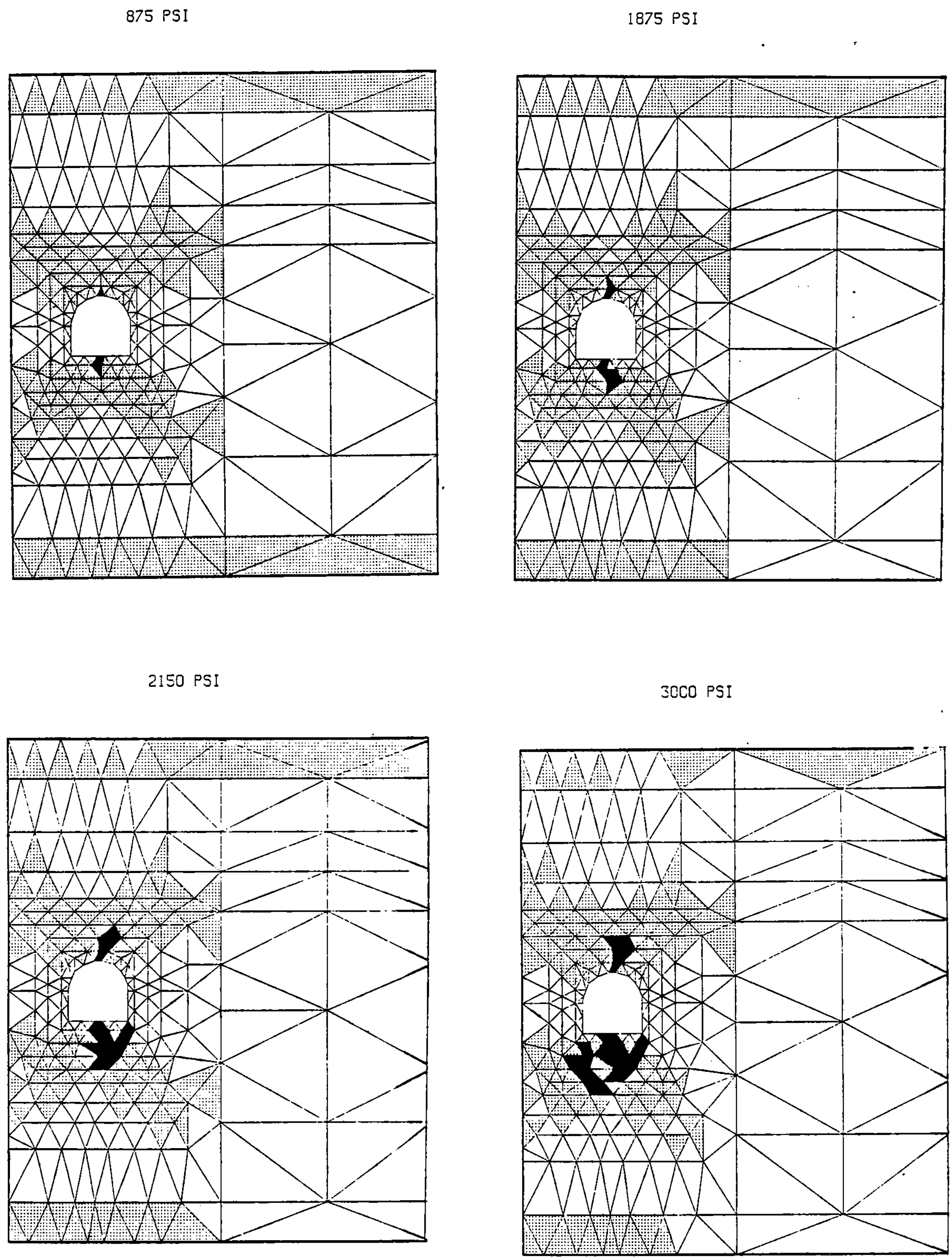

Fig. 7. Fracture sequence associated with loading of the finite element model. Dotted elements represent a tensile major principal stress. Black elements represent tensile failure. The top and bottom rows of elements represent the aluminum plates used in the model tests.

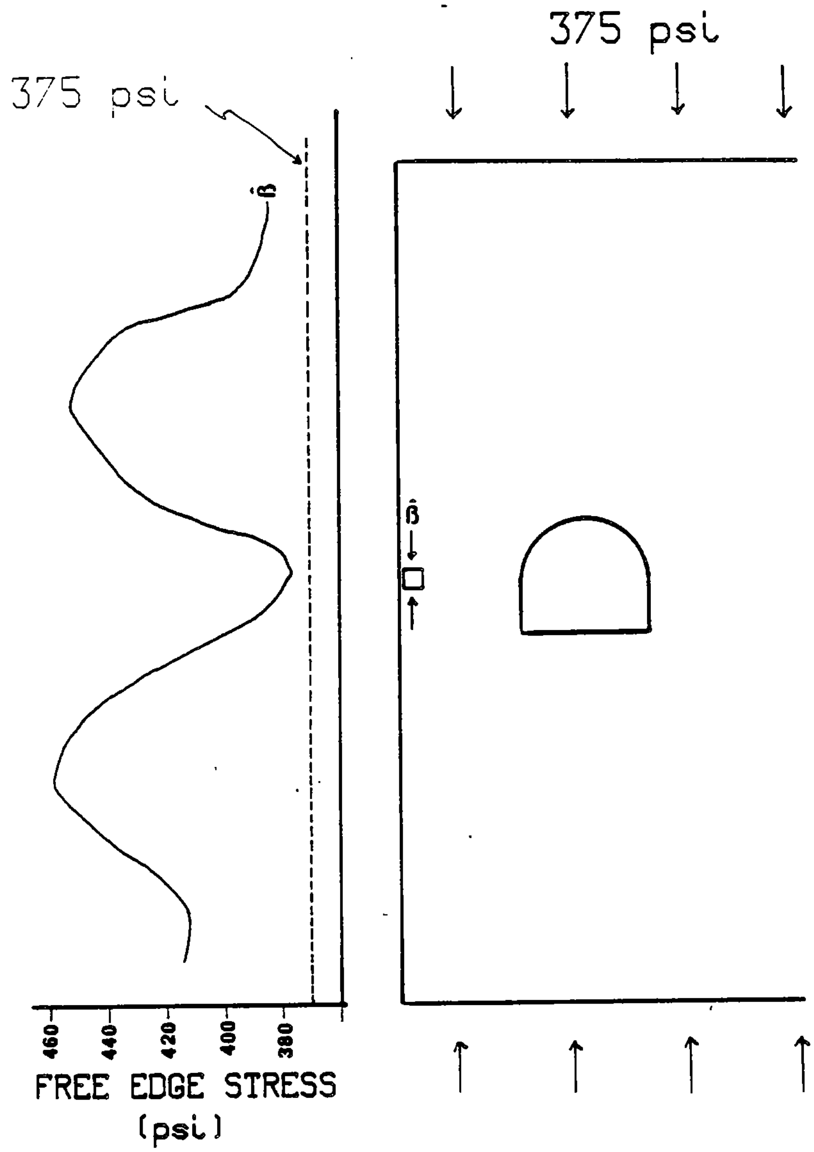

Fig. 8. Distribution of vertical stress at the free edge (cliff face) in the finite element model.

Landslip Mitigation At The
D & RGW RR / US 89 Intersection
East of Thistle, Utah

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ABSTRACT

With the rise of an approach fill for an overpass for US89 across the D&RGW tracks east of Thistle, Utah, tension cracks appeared which posed a direct threat to the continued operation of the railroad. Investigation of the developing slide involved geologic mapping, soil borings, slope inclinometer and piezometer installations, and a network of EDM survey targets.

Back analysis was used to derive shear strength parameters for the failure surface (c=200psf, ϕ= 10 degrees). Subsequent analyses were conducted to assess methods of remediating the slide.

The selected approach involved removing the approach fill and using as much of the material as possible to construct a berm on top of the toe area in the vicinity of Soldier Creek, which required diversion of the creek.

This berm has apparently stabilized the developing failure successfully. The temporary alignment of US89 with an at-grade crossing of the tracks has been upgraded to permanent status.

INTRODUCTION

The 1983 Thistle landslide necessitated the realignment of the Denver and Rio Grande Western tracks and U.S. Highways 6 and 89 which had passed through the Thistle area. Several alternative alignments of U.S. 89 across Little Paradise Valley were considered.

The selected alignment requires a crossing of the railroad tracks south of the tie in with the realigned US 6. Because of the heavy railroad traffic in Spanish Fork Canyon, a grade-separated crossing was desired. Such a crossing required construction of a substantial fill for the overpass approach.

Construction of the fill commenced about March 1, 1987 and approximately 70 feet of fill had been placed by about mid-April (fill to about elevation 5230 ft.)

On April 13, 1987 an area (shown in Figure 1) was delineated by cracks that crested along the D&RGW trackage near elevation 5222 ft., just west of the crossing of temporary US 89. The toe of the slide was not visible, but is probably close to Soldier Creek at elevation 5100 ft. An eastern bounding crack was well defined by an existing drainage path which crosses the old Highway 6 alignment due south of new alignment station 1116+60. The western edge of the slide had yet to develop a boundary crack, but the configuration of existing cracks suggested a western flank to the south of new alignment stations 1110 or 1111. Portions of the slide appear to involve old landslide debris while other portions are clearly within in-place alluvial material. Virtually all of the material involved is silty or clay-rich and some contains bentonitic, or active clays. Groundwater conditions were not known, but material in the cut immediately above the crest was damp and wetter than the adjacent cut slopes. A small bog is known to have been buried beneath the fill for the Highway 89 bridge approach.

A monitoring and investigation program was undertaken as a means of examining the stability of the embankment and conceptually formulating alternatives for remediating the problem. The program consisted of the establishment of a surface array of electronic distance measurement (EDM) survey targets and the installation of 12 inclinometers and 6 pore pressure transducers (piezometers) in the area of suspected movement.

Concurrently, 10 feet of fill were removed in an attempt to unload the slope and, hence, slow down the rate of movement.

Golder Associates was retained specifically to deal with the local slide area that threatened the railroad. The study did not include definition, evaluation, and mitigation of the significant landsliding that has been historically characteristic of the region. It is clear that the present slide is, at least partially, within a lobe of earlier slide materials.

The stretch of Soldier Fork Canyon from the Pennsylvanian rocks (just west of Diamond Fork) eastward to the Paleocene Green River Formation (approximately 9 miles) is uncommonly characterized by landslide activity. This activity has probably been essentially continuous since the draining of Lake Bonneville, (approximately 15,000 years ago) and perhaps even longer.

Periods of abnormally high precipitation (and therefore, high groundwater) have doubtless been the times of greatest slide activity.

An investigation sufficient to define the nature and overall extent of landslide activity such that permanent stabilization could be considered for the several mile stretch of US 6 east of Diamond Fork would be enormously expensive. Without such understanding, however, there is a risk that mitigation measures taken for a particular problem will aggravate other potential problem areas. UDOT has adopted the procedure of dealing with problems locally as they occur. It is recognized that throughout the area, landslide activity will affect both the highway and the railroad through the foreseeable future.

To some degree this would be the case even without the construction activities occasioned by the mid-April 1983 slide and the impounding of "Thistle Lake". The Thistle slide followed an unusually wet season. The subsequent years have also had a higher than average precipitation. As a result, by early 1987 there were at least three active landslide areas up-slope from US 6 east of Billies Mountain which are demonstrably independent from construction activities associated with the realignment of US 6 and 89. The mountain across the valley to the south has recently developed a significant failure scarp, again completely independent of construction activity. Because both the railroad and the highway have been relocated up-slope, the new alignments are within an area that is inherently less stable than that through which the previous alignments ran.

GEOLOGY

Two studies have been conducted within the project area: Young (1976), who worked on the Billies Mountain quadrangle, immediately to the north, and Pinnell (1972) who mapped the Thistle quadrangle, which covers the project area and the terrain to the south. In addition, the Utah Geological and Mineral Survey issued Map 69, on the Thistle, Utah area in June 1983. The UG&MS mapping was directed primarily to the Thistle slide that had occurred in April. This map combined the information collected by Young, Pinnell, and others and added more information on current and ancient landslides in the area.

Rocks within the vicinity of the site vary in age from the late-Pennsylvanian Oquirrh Formation to the Miocene volcanics and recent alluvium. Events leading to the present structure of the area have been identified as follows (Young, 1976):

1. Oquirrh basin development during the Pennsylivanian and the Permian.

2. Shelf deposition during the Permian and the Triassic.

3. Jurassic through development (deposition).

4. Sevier orogenic-related tectonism and deposition in the early to late Cretaceous.

5. Laramide orogenic-related tectonism and deposition in the late Cretaceous to Eocene.

6. Oligocene andesitic volcanism.

7. Miocene or later Basin and Range uplift and block faulting.

8. Pleistocene glacial action, stream rejuvenation and continued Basin and Range faulting.

The present configuration of the valley slopes is indicative of the relatively recent, but complex geologic history of the area. East of Billies Mountain, the north side of the Soldier Creek valley rises from the flat floor, which is composed of recent flood-plain deposits, including lake-bed sediments, to a gently sloping zone of young alluvial fan deposits which include some earth flow slides. Above that, at roughly 5150 ft. elevation, the topography is characterized by a series of truncated ridges trending perpendicular to the creek. These ridges are eroded remnants of an earlier generation of fans, flood plain deposits, flow slides and rock outcrops.

The realigned D&RGW trackage cuts across the lower portion of the truncated ridges at around elevation 5223 ft. The cut faces reveal a variety of geologic conditions and materials. The new US 6 alignment, at about 5334 ft. elevation, traverses a third break in the slope. Below this elevation, excavations reveal primarily alluvial and colluvial deposits with some weathered in-place rock units. Above US 6, excavations are generally through in-place rock materials.

These excavations are in ridges that bound cirque-like erosional scarps. These cirques are the source of some, but not all of the series of materials deposited below. The cirques top out at around elevation 5750 ft. in a ridge of flat-lying, resistant Flagstaff limestone. Above this ridge, the topography continues to rise gently to the north through a sequence of weak volcaniclastic sediments. Slopes above the Flagstaff are characterized by rolling, hummocky, boggy topography. This appears to be the source of much of the bentonitic earthflow material that streams into the valley below.

The visibly active portion of the earthflow slide that may influence movements in the US 89/D&RGW track area is above the Flagstaff, pushing debris over the ridge into the valley below. This does not appear to be affecting the debris lobe below the Flagstaff at present.

Given the prevalence of landsliding in the area, the frequency of instability in the new cuts along US 6, and the amount of highly weathered, clay-rich sediment in the area, it could be inferred that the lower section of gently sloping ground, between the tracks and the flood plain, is composed largely of coalescing tongues of earthflow slides and that the 8 to 10 degree slope angle represents the residual shear strength of the materials. Such an inference carries the implication that shear surfaces and other discontinuities are common throughout this zone and that the entire area is of marginal stability. Temporarily high groundwater conditions could trigger new sliding at virtually any location.

Undeniably, there are several locations in the area where these conditions exist. But much of the alluvial/colluvial material comprising the lower zone is silts and fine sands which are in place depositionally. The ground surface is not a reflection of residual shear strength and the earthflows appear to be local and discrete.

A complicating factor, however, is the repetitive cycles of erosion and deposition that probably have occurred in this area. It is possible that an earlier generation of earthflow slides has been overlain by a cycle of flood-plain, lake-bed, and alluvial fan deposits which are in place and seemingly stable. Very weak shear surfaces may exist locally beneath these deposits and it is conceivable that construction activities and climatic variations might reactivate some of these earlier-generation slides as well as the surface slide materials. UDOT rotary wash borings #5 and #6 both encountered a zone of coarse river gravel deposits at elevations of 5063.5-5052 ft. and 5057.3-5053.3 ft., respectively. In both holes, the present sliding surface appears to directly overlie this zone as would be expected if the slide material is a reactivated earlier slide.

THE LANDSLIP

As shown on Figure 1, the surface cracks delineated a block of ground about 600 ft. wide extending about 800 ft. upslope from Soldier Creek. Slope inclinometers indicate an average depth to the sliding surface on the order of about 50 ft. About 900,000 cubic yards of material are therefore included.

Fortunately, remediation of the slide was completed prior to the failure gaining full maturity in the sense of a continuous, well defined failure surface being formed. The slope inclinometers show somewhat erratic behavior in terms of both direction of movement and sliding surface elevations. The pattern of surface cracks is also indicative of a complex sliding surface as though a resistant knob were present underlying the lower portion of the slide.

Maximum displacement, as determined by accumulating the widths of individual tension cracks in the headwall area, has been on the order of two feet. Most of this movement had occurred prior to the installation of monitoring devices such that recorded movements have generally been on the order of two inches and less. Needless to say, the interpretation of the geology and mechanics of a landslide become easier as movement increases. The present slide has not developed to the extent it allows a clear understanding of the situation.

Figure 2 is a cross section through the landslip. As shown on the figure, the groundwater surface rises gently to the north away from Soldier Creek. But this drawing is deceptively simple: a 42 ft. head drop occurs west to east across the section line, with the water level in hole #3 standing around 10 ft. below surface.

The materials in the wet area are sandy while the water levels to the east are in silty clays.

Materials comprising the slide mass include silty clays and clayey silts for the most part, although some silty sands and fine sands are present. The materials are both colluvial accumulations and weathered sedimentary and volcaniclastic lithologies.

STABILITY ANALYSES

On the assumption that the slide is in meta-stable equilibrium (factor of safety close to unity), a series of stability analyses was run in order to develop realistic strength parameters for the failure surface. The parameter values most consistent with observed conditions are cohesion, (c),=200 psf, and friction angle, (Ø),=10 degrees, although another set of parameters (c=0, Ø =12 degrees) also satisfies the conditions.

Having derived a set of realistic strength parameters, analyses were then conducted to assess various means of remediating the slide. Broadly, the available methods were seen to involve improving the effective strength by reducing pore pressures, and altering the force balance through redistributing materials.

The analysis demonstrated that the slide is relatively insensitive to pore pressures: The factor of safety increases by only about 4 percent for each 10 foot drop in the phreatic surface. The size and geometry of the slide, and the significant clay content of the materials suggest that a reliable dewatering scheme would have been both difficult to achieve and quite expensive.

As shown on Figure 2, the bulk of the approach fill is not far north of the centroid of the failure mass. Consequently, removing the fill material also has a limited impact on stability. The analysis showed a 1 percent improvement in the factor of safety for each 10 foot increment of fill removed. More than anything else, it is this relationship that testifies to the marginal stability of the ground prior to construction. The rather delicate balance is doubtless responsible for the slow development of the slide and greatly enchanced the opportunity for remediation.

Beyond pore pressure reduction and removing the fill, only two other options for improving stability exist:

- stripping off material in the upper portion of the slide to reduce driving forces, and
- depositing material in the toe area to increase resisting forces.

The first option would have necessitated a realignment of temporary US 89 and the railroad tracks. It was not considered further. The preferred solution involved construction of a toe berm using materials from the approach fill to the degree posssible.

Several analyses were run to optimize the location, configuration, and volume of the toe berm. It should be noted that the slide has not matured to the extent that a visual expression of a toe is evident. The toe is someplace within the coarse flood plain sediments of Soldier Creek valley. A realistic assumption set the toe (for design purposes) in the immediate vicinity of Soldier Creek. Construction of the berm to the north of the creek would have been of relatively little value, again due to the proximity of the centroid of the failure mass. Each 10 feet of berm height would have improved the safety factor by a little more than 1 percent. However, better than 10 percent improvement in the factor of safety was seen for each 10 foot increment in height for a berm constructed over Soldier Creek. Mitigation of the slide, accordingly, required diversion of the creek.

Diversion of Soldier Creek and construction of a berm 25 ft. high with a 50 ft. wide crest and 2H:1V side slopes was completed in the late summer of 1987. Movement of the slide mass, as indicated by displacement in the slope inclinometers has slowed appreciably and possibly stopped (movement is within the error-band of the instrumentation).

CONCLUSIONS

A developing landslide which posed an immediate threat to rail traffic in Spanish Fork Canyon has been successfully mitigated with the construction of a berm to surcharge the toe area. Alternatives of removing material from the slide mass (unloading) and reducing pore pressure conditions along the failure surface were analysed and determined to provide relatively little benefit for the cost involved. Nevertheless, most of the fill for the overpass was removed with as much of the material as possible being utilized in the toe berm.

It has been decided to upgrade the temporary alignment of US 89 to permanent status and maintain an at-grade crossing of the railroad tracks in the area.

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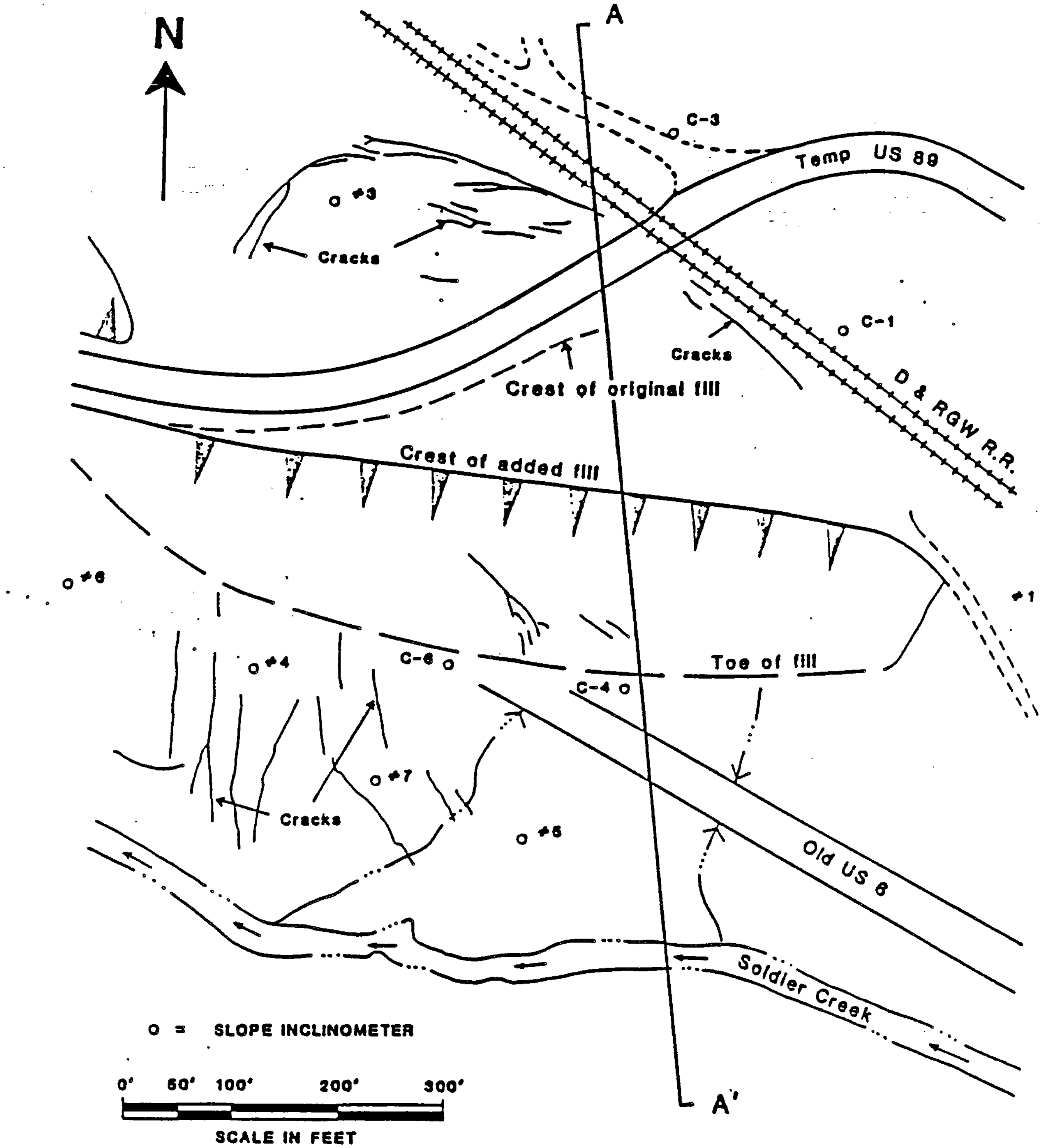
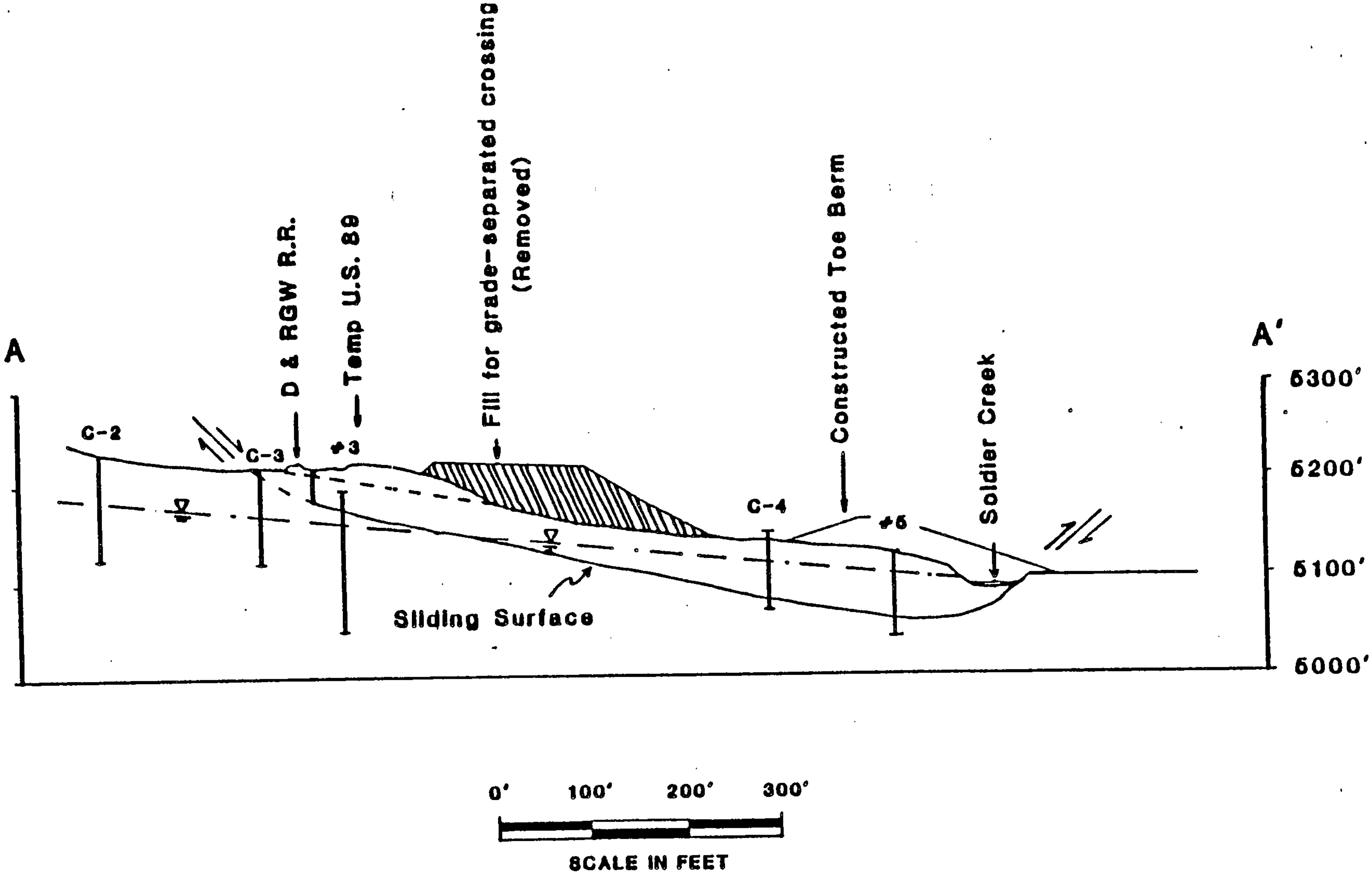


Landslip at US89/D & RGW R.R. Intersection
near Thistle, Utah

FIGURE 1



Section through Landslip
US89/D & RGW R.R. Intersection
near Thistle, Utah

FIGURE 2

THE MEASURED SLOPE STEEPNESS FACTOR AND ITS THEORETICAL ANALYSIS FOR PREDICTING SOIL EROSION ON HIGHWAY SLOPES[1]

J.-C. Fan[2] and C. W. Lovell[3]

ABSTRACT

Measurements of soil erosion have been performed on newly constructed highway slopes at Putnamville and Evansville, Indiana, in 1985 and 1986. Slope steepnesses of the tested highway slopes ranged from 9% to 50%. It is possible to use the Universal Soil Loss Equation (USLE), which is widely applied for prediction of soil erosion, for highway slopes. However, the USLE has been developed, mainly, for agricultural uses. For the USLE, as reported by Wischmeier and Smith in 1978, the slope steepness varied from 3% to 18%, which is less than usual highway slope steepnesses.

From the field erosion tests at Evansville, the slope steepness factor (S) is extended to 50%. The measured S factor reaches a maximum value of 1.5 at a slope steepness of 20% (11.2 degrees). This means that total erosion, which consists of both interrill and rill erosion, does not continue to increase with slope steepness, which is very

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different from the extrapolations of the S factor by previous researchers.

It is also found that the S factor may change with soil properties, slope length and elapsed time, and accordingly is not a factor independent of others. The S factor was assumed to be an independent factor by previous researchers.

Limited previous experimental research relative to soil erosion on steep slopes supports the above-mentioned findings. Theoretical analyses based on basic equations of soil erosion are proposed in this paper. The proposed theoretical analysis also supports the findings in this study.

INTRODUCTION

Sediment due to soil erosion has been found to be one of the greatest factors in causing water pollution. Because of growing environmental awareness, there is increasing concern with soil erosion. Regulations to limit the amount of soil sediment permitted in streams are becoming more common. Therefore, increased attention to measurement and control of erosion on construction sites has become essential. The control of erosion can produce significant construction costs. Increase in the accuracy of prediction of soil erosion can help to reduce excessive overdesign and accordingly, reduce the costs for erosion control.

Erosion of highway slopes is most severe during and immediately after construction. It is possible to use the Universal Soil Loss Equation (USLE) to predict soil erosion on highway slopes during and after construction.

The USLE by Wischmeier and Smith (1978) is written as follows:

$$A = R\ K\ L\ S\ C\ P \tag{1}$$

where

A = the computed soil loss per unit area (tons per acre)

R = the rainfall and runoff factor (hundreds of foot-tonf•inch per acre•hour)

K = the soil erodibility factor (ton•acre•hour per hundreds of acre•foot•tonf•inch)

L = the slope length factor

S = the slope steepness factor

C = the cover and management factor

P = the support practice factor

However, the USLE, which is widely applied for prediction of soil erosion, has been developed, mainly, for agricultural uses and as reported by Wischmeier and Smith (1978), the slope steepness varied from 3% to 18%, which is less than usual highway slope steepnesses.

The S factor proposed by Wischmeier and Smith (1978) is:

$$S = 65.41 \sin^2\theta + 4.56 \sin\theta + 0.065 \tag{2}$$

where θ = the slope angle (degree).

The S factor used by Foster (1982) for interrill erosion analysis is given by:

$$S = 2.96\ (\sin\theta)^{0.79} + 0.56 \tag{3}$$

Stein et al. (1983) proposed the S factor for two mine surface soils in Indiana. For the sites of the Ayshire mine of the Amax Coal Co., which is approximately 15 miles northeast of Evansville, the S value is:

$$S = 12.784 \sin\theta - 0.146 \tag{4}$$

For the sites of the Solar Sources, Inc., which is approximately 6.2 miles east of Petersburg, the S value is:

$$S = 10.742 \sin\theta + 0.037 \tag{5}$$

McCool and George (1983) proposed an equation for dry-formed croplands in the Pacific Northwest as given by:

$$S = (\sin\theta/\sin 5.14^{\circ})^{0.7} \tag{6}$$

After studying the S factors proposed by several researchers, McCool et al. (1987) recommended that the S factor be:

$$S = 10.8 \sin\theta + 0.03 \qquad \theta < 5.14^{\circ} \tag{7}$$

$$S = 16.8 \sin\theta - 0.50 \qquad \theta > 5.14^{\circ} \tag{8}$$

The curves of the S factor vs. slope angle (θ) for Equation (2) to (8) are plotted and shown in Figure 1.

All of the data sets used to develop the equations consist of a range of slope steepness from 0.1 to 18%. Application of the equations to slopes greater than 18% is an extrapolation beyond the observed data.

In Figure 1, it is obvious that the differences among these curves are very large. The six curves intersect at $\theta = 5.14^{\circ}$ (9% slope), where S = 1. But for other angles, especially the angles greater than 10 degrees (17.6% slope), the differences are significantly large.

Therefore, it is necessary to run erosion tests in the field to establish the S factor for slopes steeper than 18%. To achieve this goal, a programmable rainfall simulator was modified and successfully operated on highway slopes with steepnesses from 9% to 50% at Putnamville and Evansville, Indiana in 1985 and 1986 (Fan and Lovell, 1987).

In this paper, results of the tests which are related to the S factor are shown and discussed in detail.

RESULTS OF THE FIELD EROSION TESTS

As reported by Fan and Lovell (1988), there were three test plots at Putnamville and twelve test plots at Evansville. These test plots were on newly constructed highway slopes, without any cover, management or support practice.

The three test plots at Putnamville had 50% slopes. Each plot was 10 feet wide and 35 feet long along the slope.

There were four test sites at Evansville, and each site had three test plots. The slope steepnesses at sites No. 1, 2, 3 and 4 were 50%, 33.3%, 16.7% and 9.1%, respectively. The test plots of site No. 3 were 10 feet wide by 15 feet long along the slope, while the other test plots were 10 feet wide by 35 feet long along the highway slopes.

For each erosion test in the field, the sequence of rainfall simulation consisted of a one hour "dry run", a 30 minute "wet run", a 30 minute "very wet run", and a 40 minute "extra inflow run". The targeted intensity for each run was 2.5 inches/hour. The dry and wet runs were separated by a one hour wait, and the wet, very wet and extra inflow runs by a 15 minute data collection period. The extra inflow run was to study rill erosion by adding three successively higher rates of clear water to the top of the plot. After an initial ten-minute simulated rain (2.5 inches/hour), each inflow rate was successively applied for ten minutes.

Soil properties of the test sites, and data and results from the erosion tests are shown in Tables 1 and 2.

The soil properties of the four sites at Evansville were reasonably constant. These soil properties consisted of: specific gravity, Atterberg limits, grain size distribution, organic content and field unit weight. The soil properties of the site at Putnamville were rather different from those at Evansville.

For the test sites at Evansville, the slope steepness factor was established for slopes from 9% to 50%, as shown in Figure 2.

It is found that the S factor appears to reach a maximum value of 1.5 at an intermediate value of slope steepness of 20% (11.2 degrees).

The S factor of the site with a slope steepness of 49.7% (26.4 degrees) at Putnamville is estimated to be 1.50 to 1.75, which is much less than that proposed by previous researchers, but relatively close to the S value as shown in Figure 2.

The main purpose of extra inflow runs is to simulate the runoff from upper slopes to the test plots, from which the relationships between discharge rate and erosion rate may be obtained. From the erosion test results, the relationships between erosion rate and discharge rate for the sites at Evansville are shown in Figure 3.

After adjustment based on a rainfall intensity of 2.50 inches/hour, interrill erosion rates for sites No. 1 (50% slope), No. 2 (33.3% slope), No. 3 (16.7% slope) and No. 4 (9.1% slope) at Evansville were 0.201, 0.218, 0.235 and 0.361 $lb/ft^2 \cdot hr$ respectively. This means that interrill erosion rate decreases with slope steepness.

DISCUSSION

For convenience, Figure 1 and Figure 2 are combined as shown in Figure 4. In Figure 4, for slope steepness less than 18% (10.2 degrees) the S factor is between that defined by McCool and George (1983) and that used by Foster (1982), and is very similar to the S factor by McCool et al. (1986). The S factor proposed by McCool et al. (1986) for application of the USLE in the Pacific Northwest wheat and range region is:

$$S = (\sin\theta/0.0896)^{0.6} \tag{9}$$

where θ = slope angle (degree). This equation was derived from measured cross sections of rills on slopes ranging from 3 to 53%. But for steepness greater than 18%, the S factor reaches a maximum value of 1.5 at a slope steepness of 20% (11.2 degrees) and then decreases. This means that erosion due to interrill and rill erosion does not continue to increase with slope steepness. This is very different from the extrapolations of the S factor by a number of previous researchers, but similar to the results by Renner (1936), Horton (1945) and Foster and Martin (1969).

Renner (1936) analyzed the erosion due to rainfall and runoff on the Boise river watershed. He found that on the average, erosion increased with gradient only up to 35% slopes, after which erosion decreased with gradient.

Horton (1945) theoretically analyzed erosional development of streams and their drainage basins. He found that the relationship between erosion and slope gradient was similar to that proposed by Renner (1936). The total eroding force on a slope developed by Horton (1945) is:

$$F_1 = \frac{W_1}{12} \cdot \left(\frac{q_s\ nx}{1020}\right)^{0.6} \frac{\sin\alpha}{\tan\alpha^{0.3}} \tag{10}$$

where

F_1 = the total eroding force (lbf/ft^2)

W_1 = weight per cubic foot of water in runoff, including solids in suspension (lb/ft^3)

q_s = the runoff intensity (in./hr)

n = Manning's roughness coefficient

x = distance from the top of the slope (ft)

α = slope angle (degree)

This equation shows that there is a unique slope angle (about 40 degrees) at which the maximum total eroding force will occur.

Foster and Martin (1969) used a rainfall simulator to study the erosion of soils with different unit weights on different slopes in a laboratory. They concluded that for a given unit weight of soil, there is a unique slope from which the maximum amount of erosion will occur; and for a given slope, there is a unique unit weight from which the maximum amount of erosion will occur.

The following reasons are also advanced to explain the differences between the measured S factor in Figure 2 and the S factor by previous researchers.

[1] Under the conditions of this study, interrill erosion dominates the total erosion.

[2] Interrill erosion decreases with slope steepness.

[3] During dry runs, hydraulic tractive force is considered to be important to transport the loose particles on soil surfaces.

The results plotted in Figure 3 may be considered to be supporting evidences for the fact that interrill erosion dominates the total erosion. Figure 3 shows that when the discharge is less than a certain value, the erosion rate increases very little, or does not increase at all with the discharge rate. Discharge rate is directly related to the length of slope. The discharge rate of runoff for a broad sheet flow can be determined as (Foster et al. 1977):

$$q = \sigma x \tag{11}$$

where

q = discharge rate per unit width (volume/unit width/time)

σ = excess rainfall rate (rainfall rate minus infiltration rate)(depth/unit time)

x = distance downslope

This means that the slope lengths of the tested plots of this study are too short to allow rill erosion to occur markedly. In fact, during the field erosion tests at Evansville, no apparent rill development was found in dry, wet, very wet and extra inflow runs.

Using a rotational shear device which was originally developed by Moore and Masch (1962), Fan (1987) measured critical hydraulic shear stresses of soils, and their relationships with erosion rates. A typical test result is shown in Figure 5. A critical shear stress exists in Figure 5. For shear stress below this value, erosion rate increases very little, while for shear stress beyond this value, erosion rate increases markedly. The results are similar to those of previous studies by Espey (1963), Arunanlandan et al. (1975, 1980 and 1983), Chapuis (1986) and van Wijk and Lovell (1987).

Hydraulic shear stress acting on the surface of the soil sample increases with rotational speed, or flow velocity.

For a broad sheet flow, using Manning's formula and Equation (11), flow velocity can be determined as:

$$V = \frac{1.27(\sigma x)^{0.4}(S_e)^{0.3}}{n^{0.6}} \tag{12}$$

where

V = flow velocity (length/unit time, ft/sec)

σ, x = same as those in Equation (11)

S_e = the slope of energy line

n = Manning's roughness coefficient

From Equation (12), flow velocity is a function of excess rainfall rate, distance down slope, the slope of the energy line and Manning's roughness coefficient. For a given soil under a specific condition, a critical hydraulic shear stress, or a critical flow velocity is expected. For flow velocity below this value, erosion rate increases slowly, while for flow velocity beyond this value, erosion rate increases rapidly.

From Figures 3 and 5, slope lengths of the tested plots are too short to provide a flow velocity greater than the critical value.

As aforementioned, after adjustment based on a rainfall intensity of 2.5 inches/hour, interrill erosion rates for the sites No. 1 (50.3% slope), No. 2 (33.3% slope), No. 3 (16.1% slope) and No. 4 (9.4% slope) at Evansville were 0.201, 0.218, 0.235 and 0.361 $lb/ft^2 \cdot hr$ respectively. This evidence that interrill erosion rates decrease with slope steepness.

As Poesen (1986) pointed out, the following mechanisms may explain this:

- Steeper slopes have smaller amount of raindrop impact, since rain falls vertically.
- The normal component of raindrop impact decreases with increasing slope steepness (cosine effect).
- Low slopes have more opportunity for the occurrence of thin water layer covering the soil surface, through which the compactive force of the impacting raindrop is increased (Mutchler and Hansen (1970), and Poesen (1983)).

Before dry runs, the soil surfaces were naturally dry and had some quantity of loose particles on them. During dry runs on low slopes (such as 9.4%) the flow may not be able to transport all of the loose particles and those detached by the raindrop impact. This is true even though the interrill detachment magnitudes will be greater. Therefore, slopes with steepness of 16.1% and 33.3% may have more erosion. However, steep slopes such as 50.3% have the least erosion, since although their hydraulic tractive forces are high enough to transport the loose particles, their interrill detachments are smaller. The experimental data shown as Item 2 on Table 2 for the sites at Evansville support this explanation. These data show that the erosion rate increases and reaches a maximum value and then decreases. After the loose particles on dry soil surfaces were nearly totally eroded in dry runs and wet runs, the erosion rates in very wet runs are expected to be less for steeper slopes. The measured data for Item 4 on Table 2 support this explanation. Aside from the loose particles on soil surfaces, dry runs may have much higher erosion rates than wet and very wet runs due to slaking of soils at lower water contents.

For the slope steepness of 50%, the S factor at Evansville is about 0.88, but the S factor at Putnamville is about 1.50 to 1.75. The difference may be attributed to be soil properties. That is to say, the S factor changes with soil properties.

In Figure 3, it is also found that for different discharge rates or slope lengths, the ratio of an erosion rate of a certain slope to that of an other slope is also different. That is to say, the S factor changes with slope length or discharge rate.

In Table 2, for the dry runs at the Evansville sites (see Item 2), it is found that erosion rate increases with slope steepness only up to a certain steepness, beyond which erosion rate decreases with greater slope steepness. However, for the very wet runs at the same sites, from Item 4, it is found that erosion rate decreases with slope steepness. That is to say, the S factor changes with elapsed time.

From the above findings, the S factor changes with soil properties, slope length and elapsed time, and accordingly is not a factor independent of others. However, the S factor was assumed to be an independent factor by previous researchers.

THEORETICAL ANALYSIS

For a bare slope without any support practice, Foster (1982) used the following equations to determine detachment rate:

$$\frac{dq_s}{dx} = D = D_r + D_i \tag{13}$$

$$D_r = a(\tau - \tau_{cr})^b \tag{14}$$

$$D_i = 0.0138\ K_i \cdot i^2\ [2.96(\sin\theta_r)^{0.79} + 0.56] \tag{15}$$

where

q_s = sediment load (kg/m·h

x = distance along the slope (m)

D = detachment (or deposition) rate ($kg/m^2 \cdot h$)

D_r = rill erosion rate ($kg/m^2 \cdot h$)

D_i = interrill erosion rate ($kg/m^2 \cdot h$)

a and b = constants

τ = average shear stress of the flow channel in a rill for a given location (N/m^2)

τ_{cr} = critical shear stress of the soil to rill erosion (N/m^2)

K_i = soil erodibility factor for detachment by raindrop impact ($kg{\cdot}h/N{\cdot}m^3$)

i = rainfall intensity (mm/h)

θ_r = slope angle to rill (degree)

Chow (1959) used the following equation to determine τ:

$$\tau = w\ RS_e \tag{16}$$

where

w = unit weight of water

R = the hydraulic radius

S_e = the slope of energy line (can be approximated by the slope steepness)

For a broad sheet flow, using Equation (11) and Manning's formula, R can be determined as follows:

$$R = \frac{(\sigma\ x\ n)^{0.6}}{1.27\ S_e^{0.3}} \tag{17}$$

where

σ, x, n, S_e = same as those in Equations (11) and (12)

R = hydraulic radius (ft)

Substituting the R of Equation (17) into Equation (16), τ becomes:

$$\tau = 0.787\ w\ (\sigma x n)^{0.6} \cdot S_e^{0.7} \tag{18}$$

From Equation (18), shear stress (τ) increases with slope length (x) and slope steepness (S_e).

Critical shear stress of the soil to rill erosion (τ_{cr}) in Equation (14) is directly dependent on soil properties. τ_{cr} has been considered to be negligible by many previous researchers, such as Foster et al. (1977), Hussein and Laflen (1982), etc. This may be true for most agricultural soils. But for compacted and cohesive soils on highway slopes or construction sites, τ_{cr} is no longer negligible.

According to Equation (14), if τ_{cr} is not a zero value, when τ is less than τ_{cr}, rill erosion rate does not increase with shear stress. However, when τ is greater than τ_{cr}, rill erosion rate increases with shear stress. This matches the test results in Figures 3 and 5. If two slopes are under the same shear stress (τ) with the same unit weight of water (w), excess rainfall rate (σ), slope length (x) and slope steepness (S_e) in Equation (18), but with different soil properties, or different critical shear stress (τ_{cr}), the two slopes will have two different rill erosion rates according to Equation (14). That is to say, the S factor changes with soil properties. This explains not only the difference of the S factors at the sites of Evansville and that of Putnamville, but also the significant differences among the S factor curves proposed by previous researchers as shown in Figure 1.

Also from Equation (14), for a higher critical shear stress, the S factor is estimated to be less, and for a lower critical shear stress, the S factor is estimated to be greater. This matches the experimental results in this study and the S factor curves in Figure 4.

Assume there are two slopes, say slopes A and B. The two slopes have same soil properties or critical shear stress (τ_{cr}) and slope length (L), but different slope steepnesses, say 9% and 20%. Under an erosive condition, the ratio of the erosion rate of slope B (20%) to that of slope A (9%) is the S factor of the 20% slope steepness. (The S factor at a slope steepness of 9% has been chosen as unity by researchers for years). Now, if the lengths of both the two slopes increase from L to L_1, the S factor of slope B may change. This is true because under such a condition, shear stress which is originally lower than the critical stress may increase to a value greater than the critical shear stress, while the shear stress of slope A (9%) is still lower than the critical shear stress. Therefore, the S factor changes with slope length or discharge rate. This matches the test results as shown in Figure 3.

During rainfall, the critical shear stress changes with elapsed time. Similarly, using the same theory, the S factor changes with elapsed time. This matches the test results as shown on Items 2 to 4 of Table 2.

After analyzing a large set of data, Yang (1972, 1973, 1976), Yang and Molinas (1982), and Yang and Stall (1976) concluded that unit stream power is the dominant factor in determining the total sediment concentration of streams with alluvial and gravel beds. Yang (1973) and Yang and Song (1979) defined unit stream power (P_{us}) as the time rate of potential energy dissipation per unit weight of water:

$$P_{us} = \frac{dY}{dt} = \frac{dx}{dt}\frac{dY}{dx} = VS_e \tag{19}$$

where

Y = the elevation above datum

x = the longitudinal distance

t = time

V = flow velocity

S_e = the energy gradient and can be approximated by the slope of the soil surface

Moore and Burch (1986) demonstrated that the unit stream power for sheet flow, P_{us}, is:

$$P_{us} = VS_e = q^{0.4} \frac{S_e^{1.3}}{n^{0.6}} \tag{20}$$

where

q = discharge rate per unit width

S_e = the slope of the soil surface

n = Manning's roughness coefficient

Substituting Equation (11) into Equation (20), P_{us} becomes:

$$P_{us} = (\sigma x)^{0.4} \frac{S_e^{1.3}}{n^{0.6}} \tag{21}$$

where

σ,x = same as those in Equation 11)

From Equation (21), unit stream power (P_{us}) increases with slope length (x) and slope steepness (S_e).

If the shear stress (τ) and critical shear stress (τ_{cr}) in Equation (14) are replaced by unit stream power (P_{us}) and critical unit stream power (P_{cr}), Equation (14) becomes:

$$D_r = a(P_{us}-P_{cr})^b \tag{22}$$

where

D_r, a and b = same as those in Equation (14)

Using Equations (19) to (22), similar results as the aforementioned can be obtained, viz., the S factor changes with soil properties, slope length and elapsed time.

PROBLEMS AND SUGGESTIONS

From Items 2 to 4 in Table 2, while critical shear stresses decrease with elapsed time, soil erosion rates decrease. This is in disagreement with Equation (14), which shows that lower critical shear stresses yield higher erosion rates. Therefore, erosion rate is related not only to critical shear stress, but also other factors. These factors may include the amount of loose particles on soil surfaces or the slaking of surface soils. Slaking is more significant when the soil is very dry.

Another problem is how to determine the critical shear stress of the soil. It could be evaluated by direct shear, unconfined compression, triaxial test, rotational shear device test, vane shear test, etc. Further, the critical shear stress decreases with the degree of saturation of the soil; and under what soil condition should it be determined? Foster (1982) demonstrated that critical shear (τ_{cr}) is, in effect, "fictitious" and is a function of factors other than soil properties. It depends on the number of rills, distribution of the discharge rate among the rills, variation of shear stress both in time and space in the rills and other nonuniform factors. However, the determination of the number, locations and sizes of rills is a difficult problem.

In Equation (15), interrill erosion rate (D_i) is determined by the soil erodibility factor for detachment by raindrop impact (K_i), the rainfall intensity (i), and the slope angle of the rill (θ_i). However, it is not clear how D_i is related to slope steepness. Poesen (1986) and Fan (1988) proposed that the interrill erosion rate decreases with slope steepness. Research on this topic is needed in the near future.

Though there are many problems in using the S factor to predict soil erosion, it is suggested that engineers can still select the proper S factor using experience and judgment. For example, when soils are compacted and cohesive, the S factor should be much less than when these soils are tilled and loose. For compacted and cohesive soils on highway slopes in Indiana, it is recommended that the upper bound of the S factor be 2.

CONCLUSIONS

I. From field erosion tests on highway slopes at Evansville, Indiana, the S factor is extended from 18% to 50% as shown in Figure 2. The measured S factor reflects the development of rills, and the impact of raindrops, and appears to reach a maximum value of 1.5 at an intermediate value of slope steepness of 20% (11.2 degrees). This means that erosion due to interrill and rill erosion does not continue to increase with slope steepness. This is very different from the S factor values proposed by previous researchers, but similar to the results proposed by Renner (1936), Horton (1945) and Foster and Martin (1969).

II. The S factor of the site with a slope steepness of 49.7% (26.4 degrees) at Putnamville is estimated to be 1.50 to 1.75 which is much less than the extrapolated S values proposed by previous researchers, but close to the S factor as shown in Figure 2.

III. The S factor changes with soil properties, slope length and elapsed time. Accordingly, the S factor is not a factor independent of the others.

IV. It is proposed that the interill erosion rate decreases qualitatively with slope steepness.

V. For the erosion tests at Evansville, a critical discharge rate or a critical slope length seems to exist for a given slope steepness. When discharge rate is less than this critical value, erosion rate increases very little or does not increase at all with discharge rate, when beyond this critical value, erosion rate increases markedly with discharge rate. Test results of the rotational shear device by Fan (1987) also support this.

VI. Theoretical analyses are developed to estimate the S factor. The results match the above-mentioned findings from the field erosion tests.

VII. Both field test results and theoretical analyses show that for soils with higher critical shear stresses, the S factor will be less than those with lower critical shear stresses.

VIII. It is proposed that engineers can use the Universal Soil Loss Equation to predict soil erosion on highway slopes, with particular attention to selection of the S factor.

IX. It is recommended that the upper bound of the S factor be 2 for compacted and cohesive soils on highway slopes in Indiana.

ACKNOWLEDGEMENTS

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Table 1 Soil Properties of the Test Sites

Item of Test	Location of Site and Site No.	Evansville 1 (2 to 1 slope)	Evansville 2 (3 to 1 slope)	Evansville 3 (6 to 1 slope)	Evansville 4 (11 to 1 slope)	Putnamville (2 to 1 slope)
Specific Gravity		2.754	2.776	2.762	2.773	2.712
Atterberg Limits	Liquid Limit (%)	38.9	38.6	36.7	41.3	30.9
	Plastic Limit (%)	24.3	24.5	22.1	22.6	20.7
	Plastic Index (%)	14.6	14.1	14.6	18.7	10.2
Grain Size Distribution (% Finer)	4.76 mm (#4)	100.	100.	100.	100.	97.
	2.00 mm	100.	100.	100.	100.	94.
	0.100 mm	91.	91.	85.	88.	67.
	0.074 mm (# 200)	89.	88.	80.	84.	63.
	0.05 mm	85.	84.	77.	81.	60.
	0.002 mm	36.	31.	29.	32.	17.
Organic Matter (%)		1.0	1.0	0.8	1.0	0.5
Soil Classification	USCS	CL	CL	CL	CL	CL
	AASHTO	A-6 (close to A-7)	A-6 (close to A-7)	A-6 (close to A-7)	A-7 (close to A-6)	A-4 (close to A-6)
	USDAC	Silty Clay Loam (close to Clay Loam)	Silty Clay Loam (close to Clay Loam)	Clay Loam (close to (Silty Clay Loam)	Silty Clay Loam (close to Clay Loam)	Loam

Table 2 Data and Results from Erosion Tests

Item No.	Items			Evansville Site 1	Evansville Site 2	Evansville Site 3	Evansville Site 4	Putnamville
1	Field Density (PCF)			109.0	110.5	108.3	109.9	106.8
2	Dry Run (60 minutes)		(a) Erosion (lb) (b) Discharge (lb)	185.41 (3538.7)	197.24 (3297.2)	83.67 (1483.8)	190.70 (3548.3)	454.81 (3406.3)
3	Wet Run (30 minutes)		(a) Erosion (lb) (b) Discharge (lb)	43.54 (1781.8)	80.34 (1782.9)	27.73 (777.4)	58.66 (1803.2)	105.45 (1801.9)
4	Very Wet Run (30 minutes)		(a) Erosion (lb) (b) Discharge (lb)	44.13 (1849.5)	49.62 (1792.1)	20.94 (756.3)	51.14 (1802.8)	97.16 (1877.7)
5	Extra Inflow Run	1st 10 min.	(a) Erosion (lb) (b) Discharge (lb)	10.48 (619.9)	9.14 (501.9)	5.29 (237.9)	16.25 (608.7)	33.69 (571.0)
6	Extra Inflow Run	2nd 10 min.	(a) Erosion (lb) (b) Discharge (lb)	27.07 (1494.1)	34.14 (1660.4)	20.65 (994.1)	38.63 (1576.1)	77.65 (1564.9)
7	Extra Inflow Run	3rd 10 min.	(a) Erosion (lb) (b) Discharge (lb)	50.45 (2126.2)	79.93 (2272.9)	34.40 (1220.6)	63.93 (2241.7)	113.82 (2494.5)
8	Extra Inflow Run	4th 10 min.	(a) Erosion (lb) (b) Discharge (lb)	124.78 (3764.8)	155.65 (3403.2)	46.52 (1595.8)	109.37 (3913.3)	82.44 (3951.1)
9	Rainfall Intensity, I (in/hr)			2.500	2.175	2.389	2.203	2.720
10	Factor of Rainfall & Runoff for One Hour Rain, R			50.00	37.85	45.66	38.83	59.19
11	Length of Slope on Horizontal Base, l_h (ft)			31.273	33.203	14.809	34.843	31.338
12	Factor of Slope Length, L			0.6563	0.6763	0.4516	0.6928	0.6570
13	Slope Steepness, s (%)			50.3	33.3	16.1	9.4	49.7
14	Area of Test Plot on Horizontal Base, A_h (sq. ft.)			312.73	332.03	148.09	348.43	313.38
15	Factor of Cover & Management, C			1.0	1.0	1.0	1.0	1.0
16	Factor of Support Practice, P			1.0	1.0	1.0	1.0	1.0
17	Previous Approx. of S. Factor (Assume S = 1.0, when slope steepness = 9.419%)			0.853	1.187	1.354	1.0	
18	Factor of Slope Steepness, S			0.880	1.222	1.395	103	

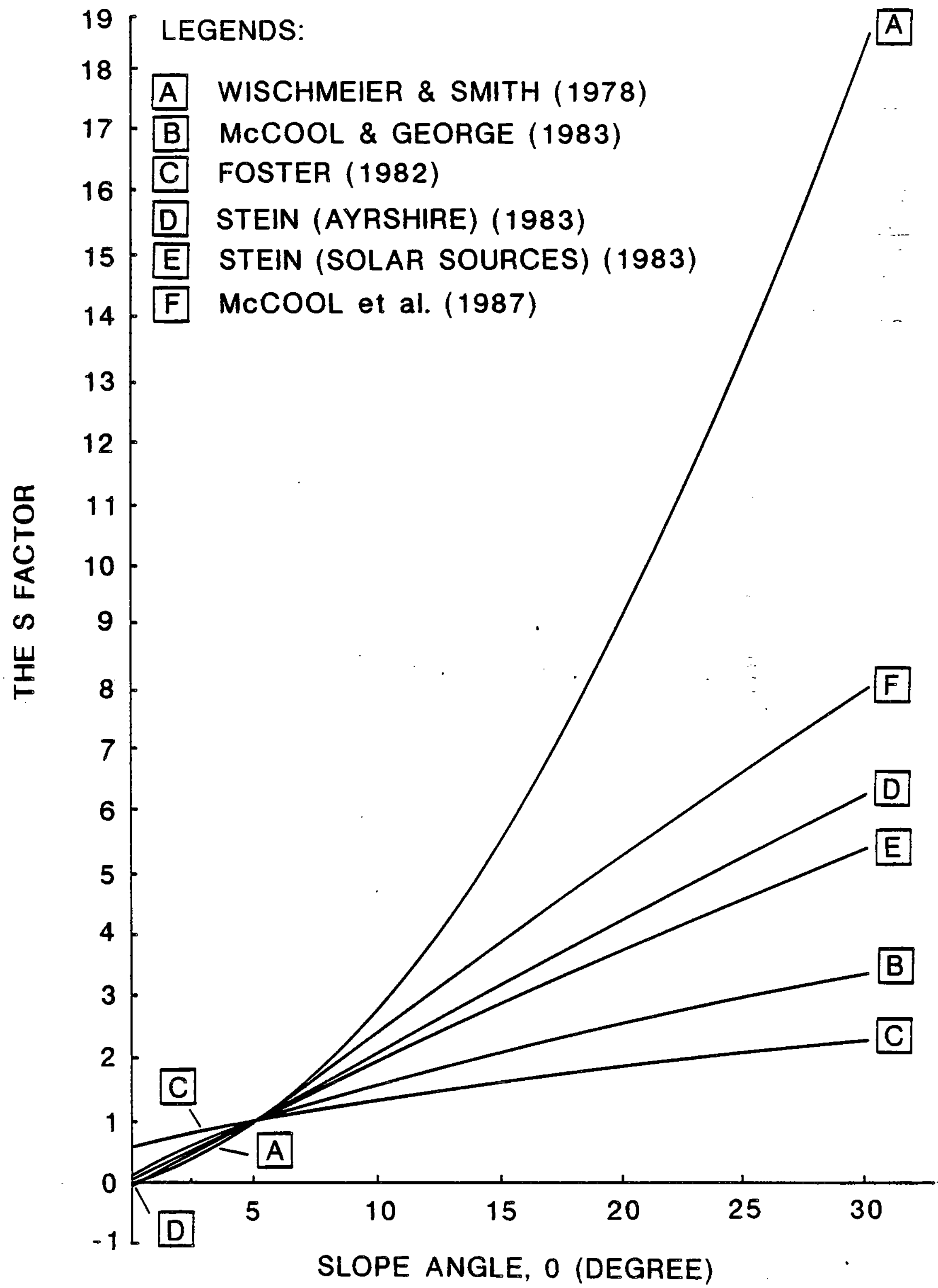

Figure 1. The S factor vs. slope angle

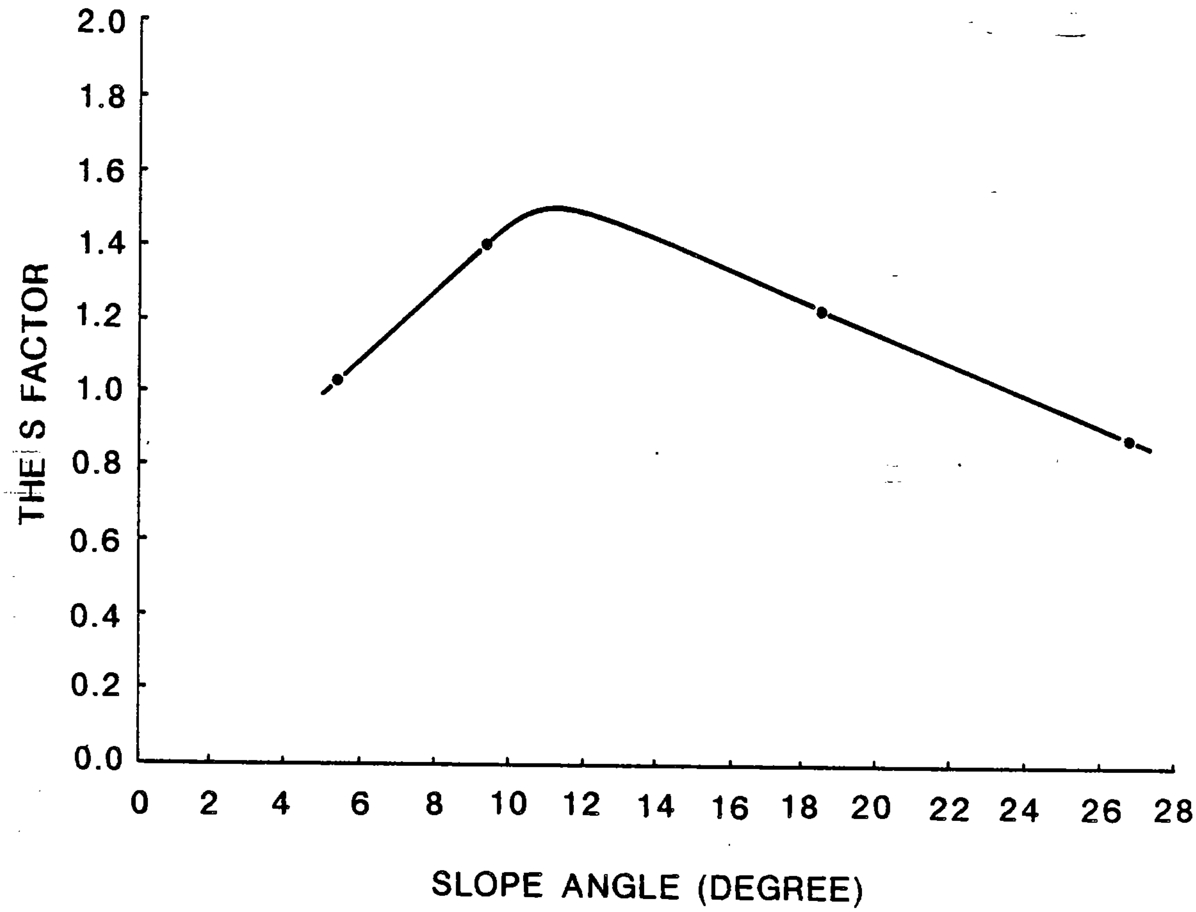

Figure 2. The S factor vs. slope angle

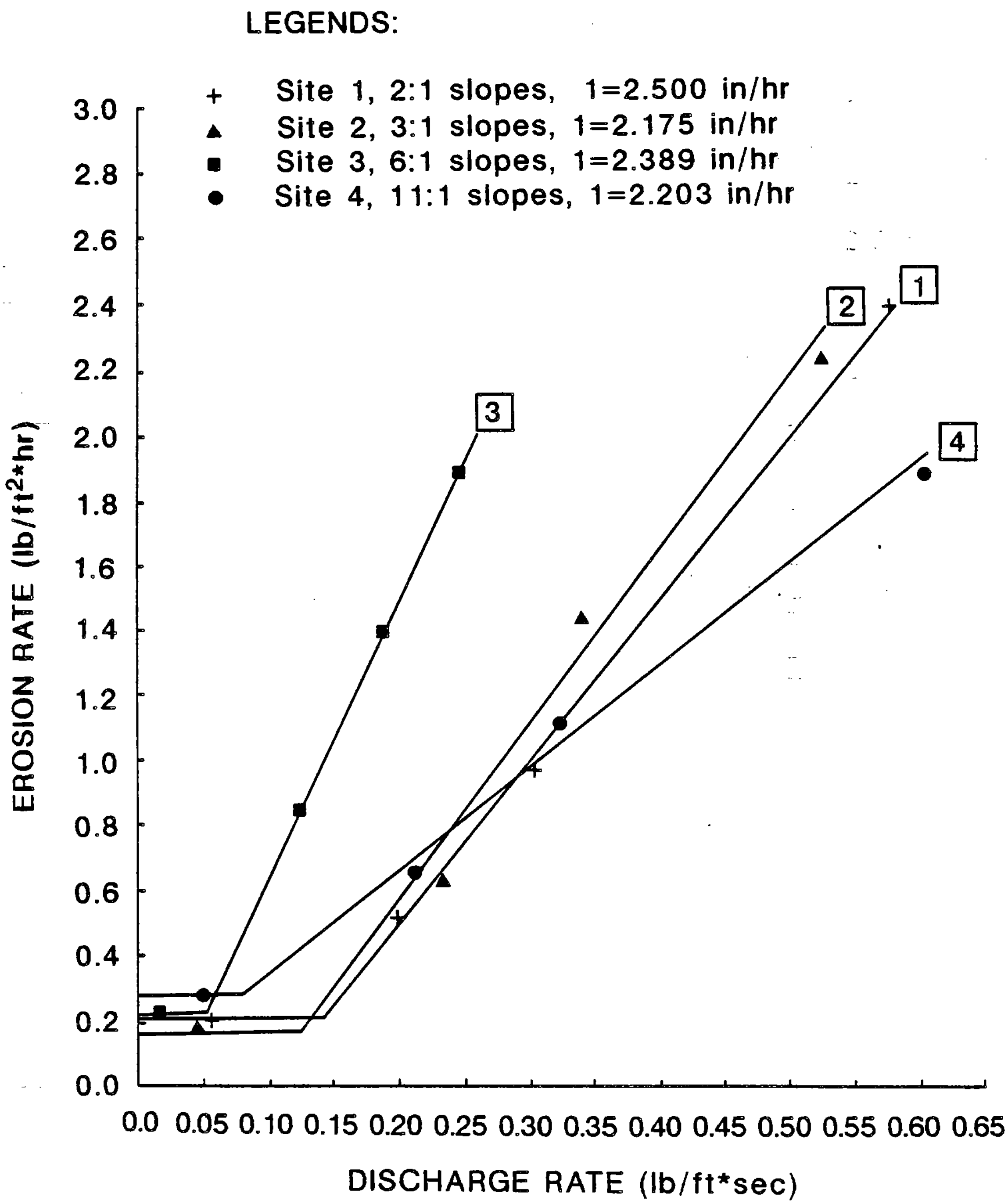

Figure 3. Erosion rate vs. discharge rate

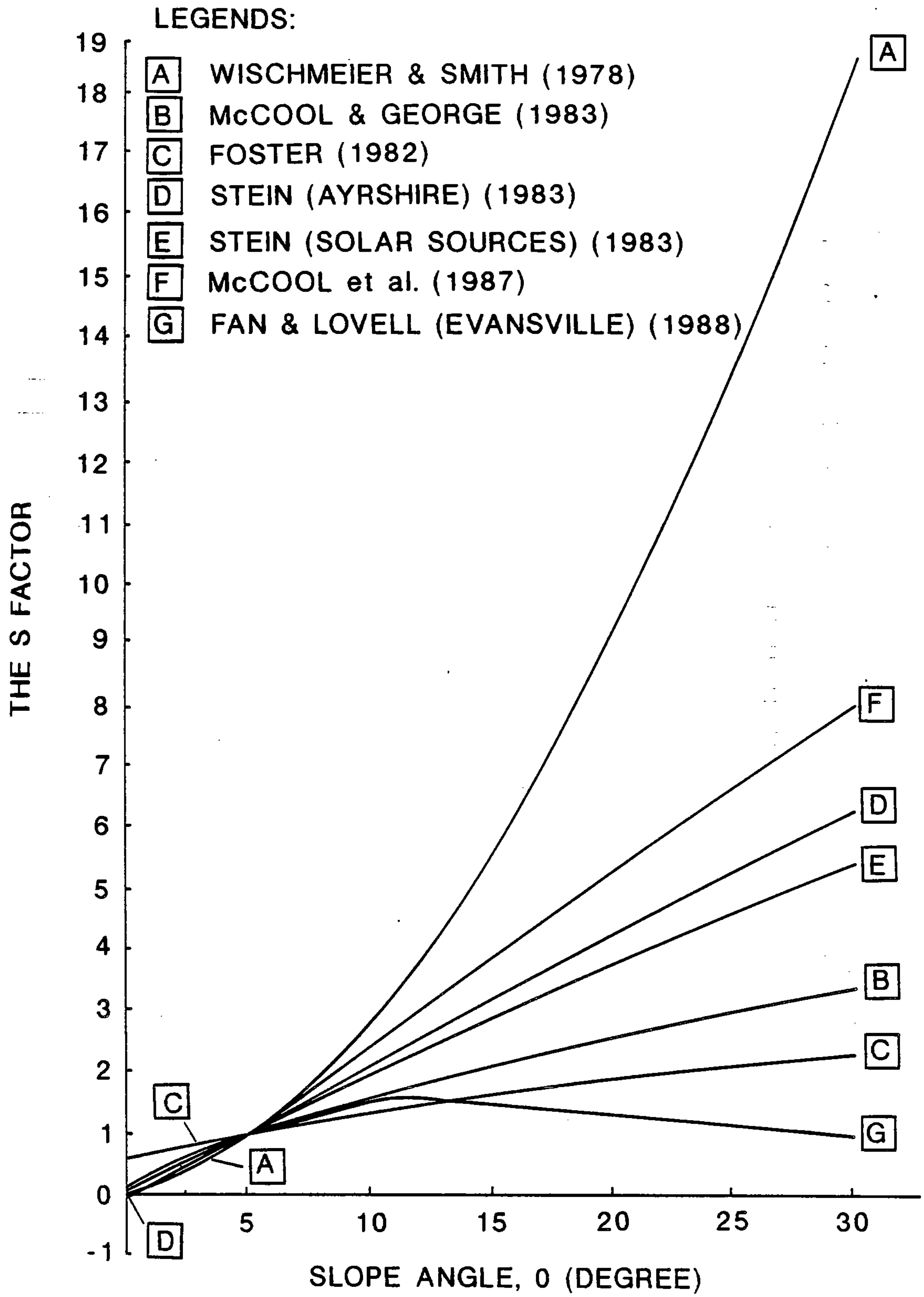

Figure 4. The S factor vs. slope angle

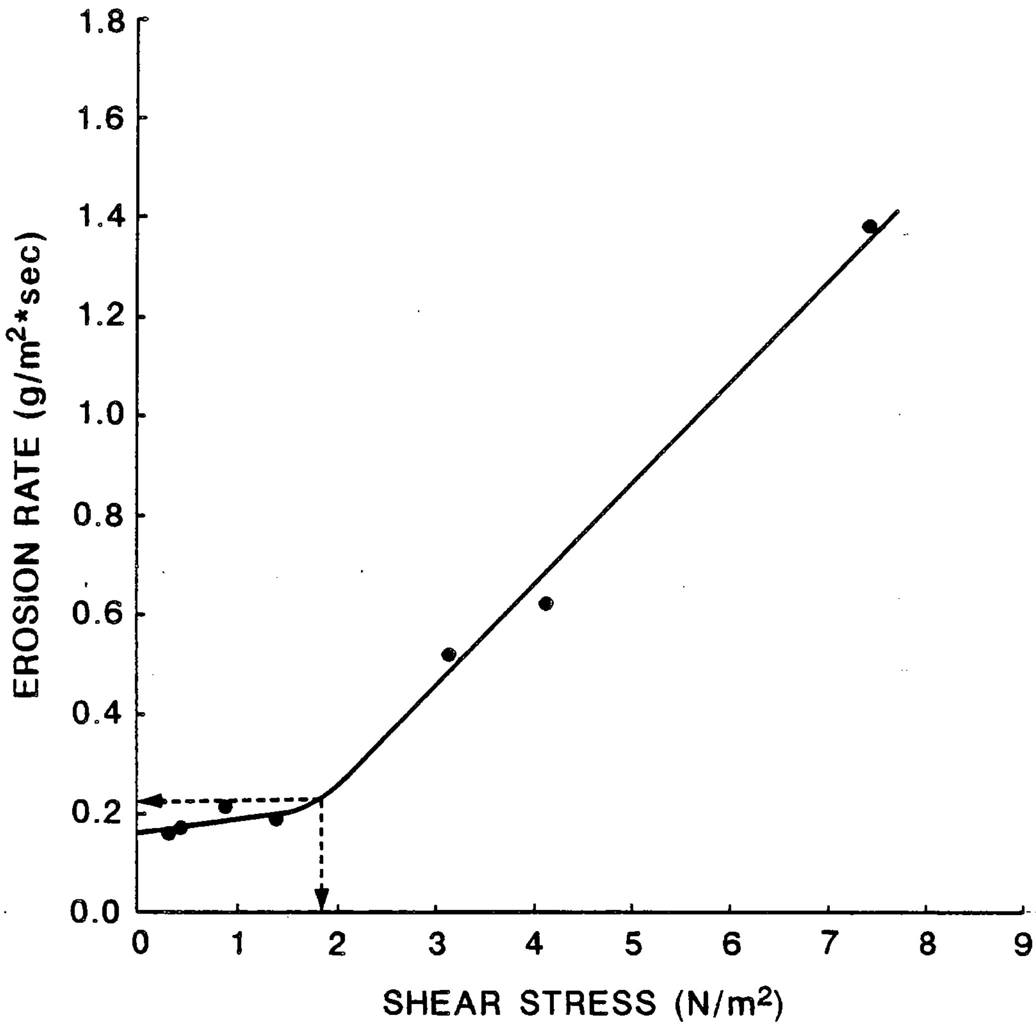

Figure 5. A typical test result of rotational shear device on the soil sample from the site at Evansville, Indiana

RELIABILITY ANALYSIS WITH PCSTABL5M

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ABSTRACT

This paper presents a method which enables an engineer to perform reliability analysis with the deterministic slope stability computer program PCSTABL5M. The assumptions made of the reader include an understanding of the following: 1) PCSTABL5M (1;6;7); 2) basic statistics, i.e., how to find the expected value, standard deviation, and correlation coefficient from a set of data; 3) probabilistics; and 4) a general knowledge of reliability analysis. The paper addresses the specific case of a dry slope with no loading conditions, treating the two strength parameters as random variables and the safety factor as the dependent variable. The expected safety factor and its distribution is determined as a function of the uncertainty of the strength parameters. An example slope condition is given to illustrate the procedure.

POINT ESTIMATE METHOD

The point estimate method (PEM) developed by Rosenblueth (4;5) is a simple yet powerful procedure of reliability analysis. It does not require attainment and evaluation of derivatives as does the Taylor series method nor does it require thousands of simulations as does the Monte Carlo method. These two restrictions make the Taylor series and Monte Carlo methods inefficient, sometimes impossible, as reliability analysis tools when dealing with chart solutions or complicated computer solutions.

The PEM is based on an analogy established between a probability distribution and a distributed vertical load on a horizontal rigid beam. Rosenblueth suggested a two-point reaction supporting the distribution; p_- acts at the location $x=x_-$ and p_+ acts at the location $x=x_+$ (see Fig. 1A). Rosenblueth termed p_- and p_+ as the two point estimates of the distribution of f(x).

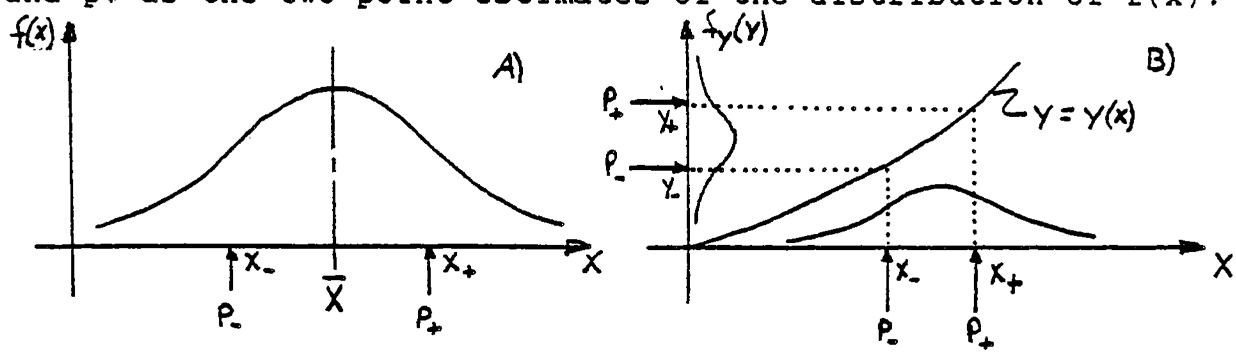

FIGURE 1 (AFTER 3)

Using "force" and "moment" equilibrium to solve the probability distribution Rosenblueth was able to define the unknowns:

$$p_+ = p_- = 1/2 \quad (1a)$$
$$x_+ = \overline{x} + \sigma(x) \quad (1b)$$
$$x_- = \overline{x} - \sigma(x) \quad (1c)$$

Derivations of these equations can be found in Harr (p.206, 3). It needs to be stated that Eq. (1) is a simplified version that assumes x is normally distributed. This is not a restrictive assumption; the basic information usually known about a distribution, i.e., the mean and standard deviation, suggest the use of the normal distribution.

The two-point estimates and their points of application serve to "transfer information about the distribution of the variate" (3). The information concerning the random variate x (x_+, x_-) is transferred through the functional relationship $y(x)$ producing the two values of y_+ and y_- (see Fig. 1b). The weighting factors p_+ and p_- then scale these estimates producing information about the distribution of the dependent random variable:

$$E(y) = \overline{y} = p_-y_- + p_+y_+ \quad (2a)$$
$$E(y^2) = p_-y_-^2 + p_+y_+^2 \quad (2b)$$
$$\sigma(y) = (E(y^2)-E(y)^2)^{.5} \quad (2c)$$

For bivariate situations, e.g., a slope environment with both strength parameters as random variables and the safety factor as the dependent variable, the PEM needs to be expanded. Rosenblueth considered the situation of two variates similar to the beam analogy but now considered the probability distribution to be analogous to a distributed vertical load acting over a rigid plate supported at four points. As before, information about the dependent variable is transferred by similar equations.

$$E(y) = \overline{y} = p_{++}y_{++} + p_{+-}y_{+-} + p_{-+}y_{-+} + p_{--}y_{--} \quad (3a)$$
$$E(y^2) = p_{++}y_{++}^2 + p_{+-}y_{+-}^2 + p_{-+}y_{-+}^2 + p_{--}y_{--}^2 \quad (3b)$$
$$\sigma(y) = (E(y^2)-E(y)^2)^{.5} \quad (3c)$$

where,

$$y_{\pm\pm} = y(x_1 \pm \sigma(x_1),\ x_2 \pm \sigma(x_2)) \quad (3d)$$
$$p_{++} = p_{--} = 0.25(1+\rho) \quad (3e)$$
$$p_{+-} = p_{-+} = 0.25(1-\rho) \quad (3f)$$

The ρ value is called the correlation coefficient. The correlation coefficient is an indication of how well two variables vary together. The correlation coefficient ranges from -1 (one variable tends to increase as the other decreases) to +1 (both variables tend to increase or decrease together) to 0 (no correlation exists between variables). Ideally, geotechnical engineers want the two strength parameters to be negatively correlated; in a section of the slope where the strength intercept is lower than expected, the strength angle tends to be higher than expected. As will be discussed later the ρ between the strength angle and strength intercept depends on the test type ,i.e., consolidated undrained triaxial test, ect., which in turn simulates the field loading conditions (long term, short term).

Rosenblueth and others have extended the PEM method to include multivariates (2;3).

PEM WITH PCSTABL5M

In a typical slope environment there are numerous uncertainties including strength parameters, unit weights, pore pressures, surface loads, and earthquake loads. When multiple soil layers are present within the slope the number of uncertainties increases dramatically even more.

This paper addresses the case of only two variables. An extension of this method to include more variables is straight forward. A dry slope with no surface loads and one soil layer will be considered. The two strength parameters, strength angle (ϕ) and strength intercept (c), will be treated as the random variables.

The first step of the method is to gather information about the two strength parameters. This is usually done by obtaining samples from boreholes in the natural slope or samples of the borrow pit material. Standard laboratory tests, e.g. triaxial tests, are run on these samples to obtain strength parameter data. Using statistical methods the mean and spread of the strength parameters as well as the correlation between them can be obtained.

In some situations data is limited such that it complicates accurate determination of the random variables spread. Harr (3) recommends that if no additional information is available the coefficient of variation of the strength angle can be assumed to be around 10% while that of the strength intercept can be assumed to be around 40%. (Recall that the coefficient of variation is defined as the ratio of the standard deviation to the mean of the random variable.) Harr based these percentages on data he obtained from literature searches. (Harr also presents data on the coefficient of variation of unit weight, dead and live loads, and earthquake loads.)

Harr in addition gives guidelines for ρ selection when data is limited. He states ρ being positive for CU tests ($\approx$+0.25 found by one source), negative for CD tests, and inconclusive for UU tests.

After the data is analyzed and the mean and distribution of both parameters is defined, the $c_{\pm}$ and $\phi_{\pm}$ values and their weighting factors are calculated using Eqs. 3d, 3e, and 3f:

$$c_{\pm} = \overline{c} \pm \sigma(c)$$
$$\phi_{\pm} = \overline{\phi} \pm \sigma(\phi)$$
$$p_{++} = p_{--} = 0.25(1+\rho_{c\phi})$$
$$p_{-+} = p_{+-} = 0.25(1-\rho_{c\phi})$$

The dependent variable for this situation is the safety factor; $SF=f(c_{\pm},\phi_{\pm})$. To obtain the four SF estimates PCSTABL5M is run four times with the different combinations of c and ϕ. Applying the weighting factors p to the estimates will produce the expected SF and its spread (Eqs. 3a, 3b, and 3c).

Now the user has in their possession not a single deterministic number describing the adequacy of the proposed slope design, but instead a distribution giving more information about the expected safety factor and its chances of varying. The distribution of the safety factor gives the designer a better grasp of their design's adequacy.

The probability of the safety factor being lower than a specific value (risk assessment) is now also possible. Since only the mean and standard deviation of the SF distribution is known it can be assumed as normal. (The safety factor would not be a normal distribution since the safety factor cannot be less than 0, but the normal distribution is a good estimate.) Knowing the distribution is normal, probabilities can easily be found in normal distribution charts.

EXAMPLE

The described procedure is best understood with a simple example. An embankment is to be constructed on a stable foundation; a toe failure through the embankment will be considered. The material for the embankment will be obtained from a borrow pit. Numerous samples were gathered from the pit area and prepared to the design compaction. These samples were then tested to find their long term strength parameters (CU test). The laboratory values obtained include:

$\overline{c}$=200psf; $\sigma(c)$=80; $\overline{\phi}$=25°; $\sigma(\phi)$=2.5; γ=140pcf.

The embankment is 40 feet high with slopes of 2H:1V. The water table is below the foundation and no significant surface loads are anticipated.

Using the mean values and slope geometry a deterministic SF of 1.410 was obtained.

To begin the probabilistic investigation the input strength parameter estimates are calculated.

c_{+}=200+80=280psf
c_{-}=120psf
ϕ_{+}=27.5°
ϕ_{-}=22.5°

The correlation coefficient between c and ϕ was determined from the laboratory tests to be +0.25. From this the scaling factors are calculated using Eqs. 3e and 3f:

$$p_{++} = p_{--} = 0.25(1+.25)=0.3125$$
$$p_{+-} = p_{-+} = 0.1875$$

PCSTABL5M is now run four times with the different combinations of strength parameters. The safety factors obtained:

$$SF_{++} = 1.685$$
$$SF_{+-} = 1.454$$
$$SF_{-+} = 1.373$$
$$SF_{--} = 1.140$$

From these the mean and standard deviation of the SF can be calculated using Eqs. 3a, 3b, and 3c:

$$E(SF)=0.3125(1.685)+0.1875(1.454)+0.1875(1.373)$$
$$+0.3125(1.140) = 1.413$$
$$E(SF^2)=0.3125(1.685)^2+0.1875(1.454)^2+0.1875(1.373)^2$$
$$+0.3125(1.140)^2 = 2.043$$
$$\sigma(SF) = (2.043-(1.413)^2)^{.5} = 0.216$$

The expected safety factor and its standard deviation is now known. Assuming a normal distribution the designer can figure the probability of the safety factor being lower than 1.3 (typical design criteria).

$$P(SF<1.3) = 30\%$$

Another use available to the engineer working with reliability analysis is risk design. In this example the engineer wants to reduce the risk (P(SF<1.3)) to ≈15%. For a risk of 15% the embankment slope would need to be 2.2H:1V. The corresponding deterministic safety factor is 1.537.

By reducing the slope from 2H:1V to 2.2H:1V the designer has reduced the risk by 50%. Using the traditional deterministic SF the designer would have no idea how much safer the design is by increasing the SF from 1.410 to 1.537.

CONCLUDING REMARKS

The PEM applied to PCSTABL5M presents the engineer with the ability to design using reliability analysis. It can present additional information to the engineer about the deterministic value obtained. It allows the engineer to have a better feel of their results. Reliability analysis also is a tool which can be used to quantify the increase in safety (or reduction in risk) between different designs.

The potential for the PEM as a reliability analysis tool in geotechnical engineering has been illustrated. The PEM can be applied to computer solutions, chart solutions, or simple equation solutions.

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GROUNDWATER INFLUENCE ON HIGHWAY FILL SLOPE STABILITY

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R & R INTERNATIONAL, INC.

ABSTRACT:

A study of a one-mile section of the Ohio Turnpike located in the Lower Cuyahoga River Valley was conducted by R & R International to determine the factors contributing to its chronic gradual and progressive slope failures. Portions of the highway were constructed on approximately 100 feet of fill consisting primarily of glacial till with two (2) interstratified layers of sand and sandstone spoil.

The research tasks involved in our study consisted of a review of highway construction records, previous subsoil and groundwater data, and a literature search of pertinent local and regional geological studies. Field tasks included geomorphological mapping, subsurface investigation and installation of dual elevation piezometers. Continual synthesis of the data obtained from these tasks enabled us to focus the investigation on the underlying source of the slope instability problems and to recommend effective measures to correct them.

It was determined that the primary cause of the slope failures was excessive groundwater entering the fill system at an upgradient ridge cut for the highway. The groundwater traveled downgradient through the permeable sand and sandstone layers within the fill. This flow was blocked down gradient by another ridge and ultimately exited as seepage out the sides of the fill system's slopes. The saturation of the fill weakened the slopes and initiated the slope's instability. The continuous surcharge of the groundwater into the weakened areas propagated the progressive slope failures for many years to the extent that the entire fill system and roadway above were in imminent danger of massive failure.

ABSTRACT (Continued):

The remedial measure implemented consisted of construction of an interceptor drain along the ridge where the ground water was entering the fill. A program of monitoring and evaluating the effectiveness of this remedial action over a period of 1 1/2 years has recently been completed. It has been documented that the interceptor drain is successfully drying the fill slopes. Slope regrading, additional interceptor drains, toe berms and lateral structural supports have been recommended now that the groundwater problem has been rectified.

INTRODUCTION

Since completion of construction in 1954, the section of the Ohio Turnpike shown in Figure 1 has been affected by slope instability problems. Previous engineering studies by other firms have been conducted during the past 15 years in an attempt to stabilize the slopes. The remedial measures implemented prior to our investigation consisted of installing and monitoring slope indicators, installing and monitoring groundwater observation wells, surface drainage modifications; and temporary reinforcements of critical local failures by driving walls of guard rail posts. It seems that the connection between excessive water on the fill slope and the slope failures was made at an early date. Efforts to dry the slopes, however, were focused on surface drainage alterations in the apparent belief that the groundwater seepage out of the slope's sides was the result of surface water infiltration at the top of the slopes. These measures were ineffectual in halting the flow of seepage and the slope failures continued. Reinforcing walls made of guard rail posts have also failed to cease the downhill creep of the slopes.

In 1986, R & R International was retained by URS Consultants, Inc. to investigate the cause of the stability problem and to recommend possible solutions. A preliminary field reconnaissance concurred with the previous studies that a strong correlation existed between the water seepage on the highway fill slopes and the slope failures. This paper details the investigative process employed in determining the most probable cause of the slope failures and the engineering solution designed to remediate them.

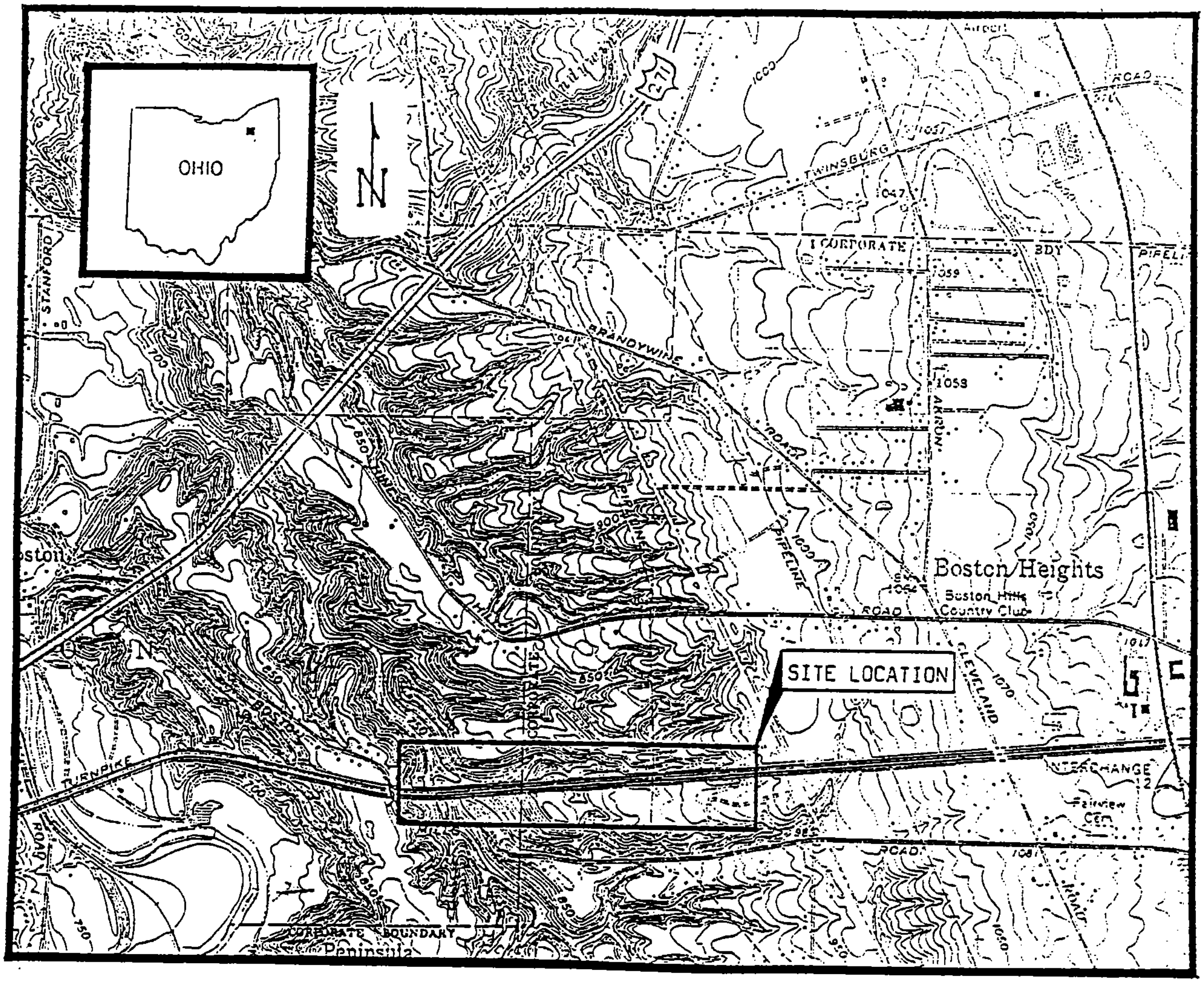

FIGURE 1 - SITE LOCATION MAP

SITE GEOLOGY

The project site is located within the glaciated northwest portion of the intensely dissected Allegheny Plateau. It is characterized by a youthful drainage system evidenced by deeply incised valleys and narrow, steep-sided ridge lines that broaden to rolling hills to the east. Drainage is generally to the west where it is ultimately confluent with the northward flowing Cuyahoga River.

The soils at the project site consist of silt and/or clay loams with stringers of sand. The silts and clays contain numerous high-angled, sand-filled and mineralized fracture zones.

The sedimentary bedrock beneath this site is composed primarily of the Olentangy and Ohio shales of the Devonian period. Approximately one (1) mile east of the site, a coarse clastic unit of the Mississippian period is exposed in an Ohio Turnpike road cut.

The hydrogeology of the region is very complex due to the high relief, youthful drainage, and highly variable hydraulic conductivity of the till. The ridges typically contain natural springs and evidence of groundwater seepage on their slopes.

The high silt and clay content of the glacial till, coupled with the steep-sloped terrain and complex hydrogeology, make this region highly susceptible to natural land slumping and slope failure. When these natural factors inducing slope instability are compounded by man-made interference such as construction of roadways and artificial drainage, the potential for landslides greatly increases.

INVESTIGATIVE METHODOLOGY

The field tasks conducted for this investigation in their respective chronological order of completion were as follows:

1. Geomorphological Mapping
2. Advancing test borings
3. Installing piezometers
4. Laboratory analysis of soil samples
5. Monitoring ground water levels

The geomorphological mapping was conducted to document all surface expressions of the slope instability and the factors contributing to it. Figure 2 plots these features on a site plan. This task was completed prior to beginning the remainder of the field tasks so that information obtained from the mapping could be used to direct the boring plan being developed. The following pertinent features of the slope instability were noted:

1. Slump scarps
2. Evidence of downhill creep
3. Tension cracks
4. Water seepages
5. Erosion gullies
6. Drainage patterns

Evidence of ground water seepage was associated with each major area of slope instability. Several depressions or "dishes" resulting from local slumping support hydrophillic vegetation indicating persistent groundwater seepage.

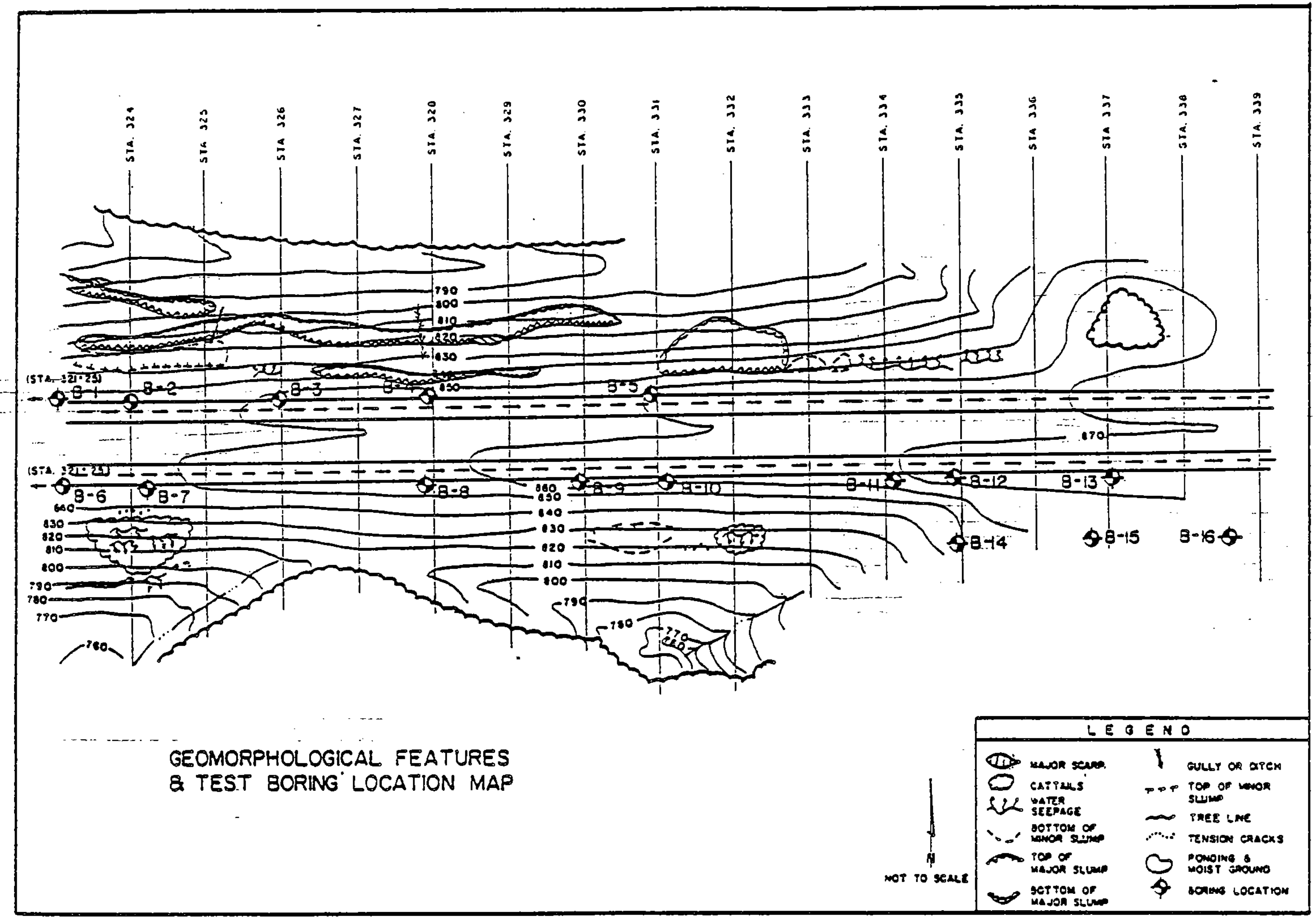

FIGURE 2 - GEOMORPHOLOGICAL FEATURES AND
AND TEST BORING LOCATION MAP

INVESTIGATIVE METHODOLOGY (Continued)

Based on the geomorphological mapping it became apparent that the excessive water seepage along the unstable portions of the fill slopes were either causing the slope failures or were a major contributing factor aggravating the instability. A subsurface investigative plan initially consisting of ten (10) test borings was then developed in order to determine the subsurface stratigraphy, the physical characteristics of the slope's soils at depth and to trace the origin of the ground water seeps. Please refer to Figure 2 for boring locations.

It was decided to continuously sample the test borings so that probable thin lenses of ground water or relatively weak strata could be detected. Borings were extended to approximately 50 feet so that the entire strata interval experiencing the water seepage and instability would be penetrated. Borings B-11 through B-16 were added in a second phase boring plan designed to verify assumptions developed during reduction and synthesis of data obtained from the initial test borings. All soil samples were logged in the field based on physical examination. Laboratory testing on selected samples was used to classify those samples according to the Unified Soil Classification System.

Piezometers were installed in the test borings to intersect and monitor zones where groundwater was encountered during drilling. Because the test borings consistently penetrated two (2) distinct groundwater lenses, nested piezometers were installed to monitor the separate groundwater lenses within the same borehole.

INVESTIGATIVE METHODOLOGY (Continued)

Laboratory testing conducted on selected soil samples consisted of the following:

1. Moisture content (ASTM D-2216-80)
2. Gradation analysis (ASTM D-422-63)
3. Atterberg limits (ASTM D-4318-84)
4. Direct Shear Strength (ASTM D-3080-72)

The moisture content of each sample was determined in order to establish trends in moisture with depth and/or location. Several gradation analyses were conducted to verify soil classifications and aid in stratigraphic correlations between borings. Atterberg limits were determined to document the variance in behavior of the silts and clays with changes in their moisture content. Direct shear tests were conducted on undisturbed shelby tube samples obtained in accordance with ASTM D-1587 to determine the soil's strength parameters.

Monitoring of the ground water levels within the piezometers was conducted for a period of 1 1/2 years from the completion date of the drilling in October of 1986. The direction of ground water flow within the fill was dictated by the turnpike gradient. Empirical information concerning the relative amounts of groundwater flow within the fill slopes was the objective of this monitoring.

RESULTS AND DISCUSSION

The connection between the slope's ground water seepage and the slope failures was evident from the geomorphological mapping. The source of the groundwater, then, became the focus of our subsurface investigation. The previous engineering studies at this site had concentrated on surface drainage improvements based on the assumption that surface water infiltration was the source of the seepages. Our study examined the entire hydrogeologic regime of the site and investigated possible subsurface sources of the excessive groundwater.

As the logs from the continuously sampled test borings were correlated, it was established that two separate and distinct sand and sandstone layers were deposited within the predominant silt and clay of the fill. Figure 3 is a subsurface profile showing these correlations. A review of turnpike construction records revealed that the source of the sand and sandstone was a sandstone ridge cut for the turnpike approximately one (1) mile east of the site. The sand and sandstone was apparently spoiled into this area simply for use as fill. According to construction records, sandstone blanket drains were installed at nearby locations to enhance drainage below the turnpike in areas with obvious groundwater problems. No mention of such deliberate sandstone blanket drain construction was noted in the construction documents for this particular site.

There is no evidence of the sand and sandstone layers on the slope's surface. They were apparently only deposited in the fill's core. The

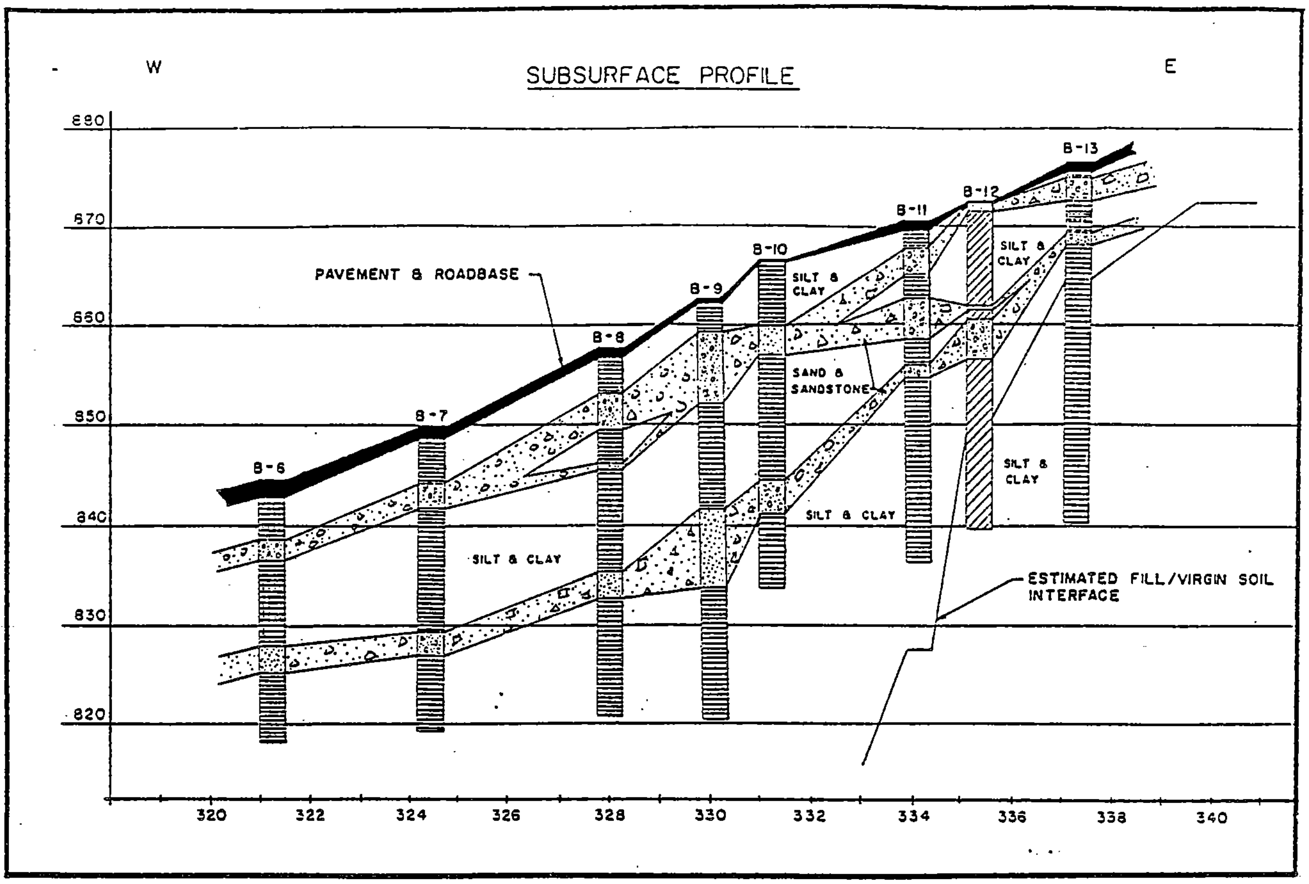

FIGURE 3 - SUBSURFACE PROFILE

RESULTS AND DISCUSSION (Continued)

sand and sandstone layers were penetrated in each boring which indicates lateral continuity. In addition, the elevation of the layers correlated very well with elevations of the seepages on the fill slopes. Soil samples from the bottoms of the sand layers were often saturated evidencing groundwater flow. Based on this information it was concluded that the sand and sandstone layers within the fill were acting as a conduit for groundwater and were hydraulically connected with the seepages.

The establishment of the sand and sandstone layer as a conduit for the groundwater, however, did not explain why the water exited the slopes as seepage at this particular location. Nor did it identify the source of the groundwater. Without the answers to these questions, development of remedial design plans were still not feasible. An examination of the geomorphology of the site provided an answer to the first question and suggested a possible solution to the second that would require further investigation to verify.

The turnpike fill was placed in a valley between two ridges striking approximately perpendicular to the turnpike. The turnpike, and therefore the sand and sandstone layers within its core, slope to the west between the two (2) ridges. The western ridge is dissected completely by the turnpike and the depth of the fill at the point where the highway crosses the ridge is essentially reduced to zero. Because the glacial till composing the

RESULTS AND DISCUSSION (Continued)

ridge has a significantly lower hydraulic conductivity, downgradient flow of the ground water within the sand and sandstone layer of the valley fill is blocked at this point. The ground water backs up in the sand and sandstone layer until excessive pore pressure finds relief via the seepages along the valley fill slopes.

The eastern ridge is not dissected by the turnpike. Instead, the ridge's northern nose is truncated by the highway. It was deduced that this ridge was supplying the ground water to the system. In order to verify this deduction, borings B-14 through B-16 were advanced into the ridge on virgin ground just south of where the ridge abuts against the turnpike's shoulder.

The borings primarily encountered silt and clay and each boring penetrated a zone of moist to wet, high angled, calcite mineralized fractures. The bottom of the fracture zone was saturated. The mineralization deposits were up to 1.0 centimeter in thickness indicating that groundwater has passed through the fractures for many years. Further, the depth interval of the fracture zone corresponded in elevation to that of the sand and sandstone layer within the fill. This information confirmed that the ridge was an entry point of groundwater into the highway fill. Please refer to figure 4 for a schematic diagram of the groundwater flow patterns at this site.

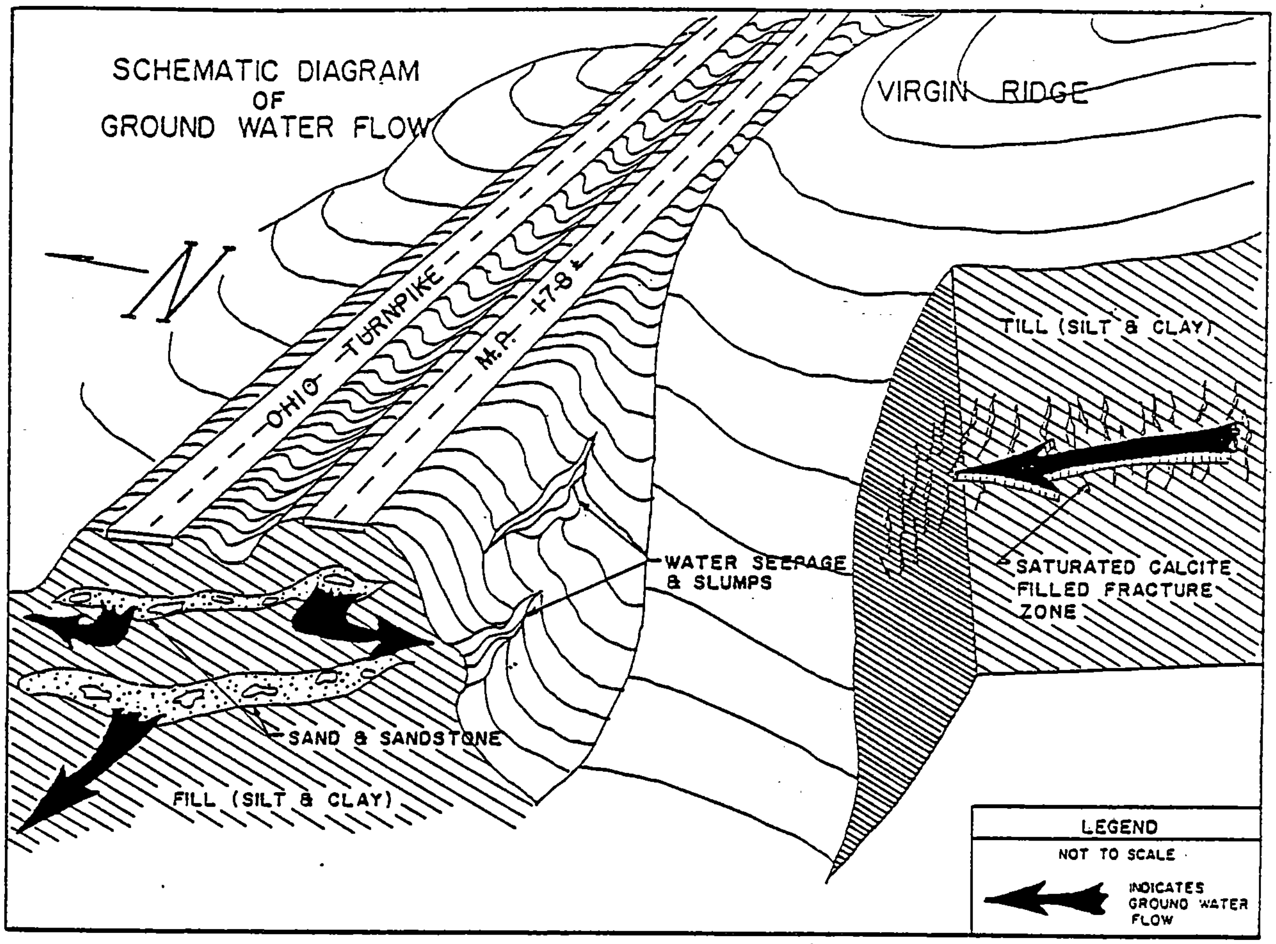

FIGURE 4 - SCHEMATIC DIAGRAM OF GROUNDWATER FLOW

REMEDIAL MEASURES

Remedial design plans focused primarily on the interception and diversion of the groundwater away from the turnpike fill. It was decided that the most suitable location to attempt such a diversion was at the point where the ridge abutted against the turnpike's shoulder. Figure 5 is a schematic diagram of the interceptor system implemented. The figure also shows the resulting changes in groundwater flow patterns.

The depth to which the interceptor system had to be excavated in order to intersect the bottom of the water bearing zone complicated development of the design plans. Right-of-way restraints and the need to maintain the 2 to 1 slope, which was determined to be safe for the soils at this site, dictated the maximum depth that the bottom of the interceptor ditch could achieve. This depth ranged from 5 to 15 feet above the bottom of the saturated zone. To overcome this problem the bottom of the ditch was trenched the remaining depth to reach an elevation 1 to 2 feet below the saturated zone. A 12 inch perforated corrugated metal pipe was placed in the bottom of the trench, surrounded by aggregate and a silt blanket. The trench was then backfilled with on-site material. The bottom of the ditch was lined with concrete.

Similar to a phased investigative plan, a successful remedial plan can also achieve great benefits through a phased approach in implementation. Although the complete recommended solution to the slope instability problems at this site consists of slope regrading, toe berms, lateral structural supports and further surface drainage improvements, it was decided that it

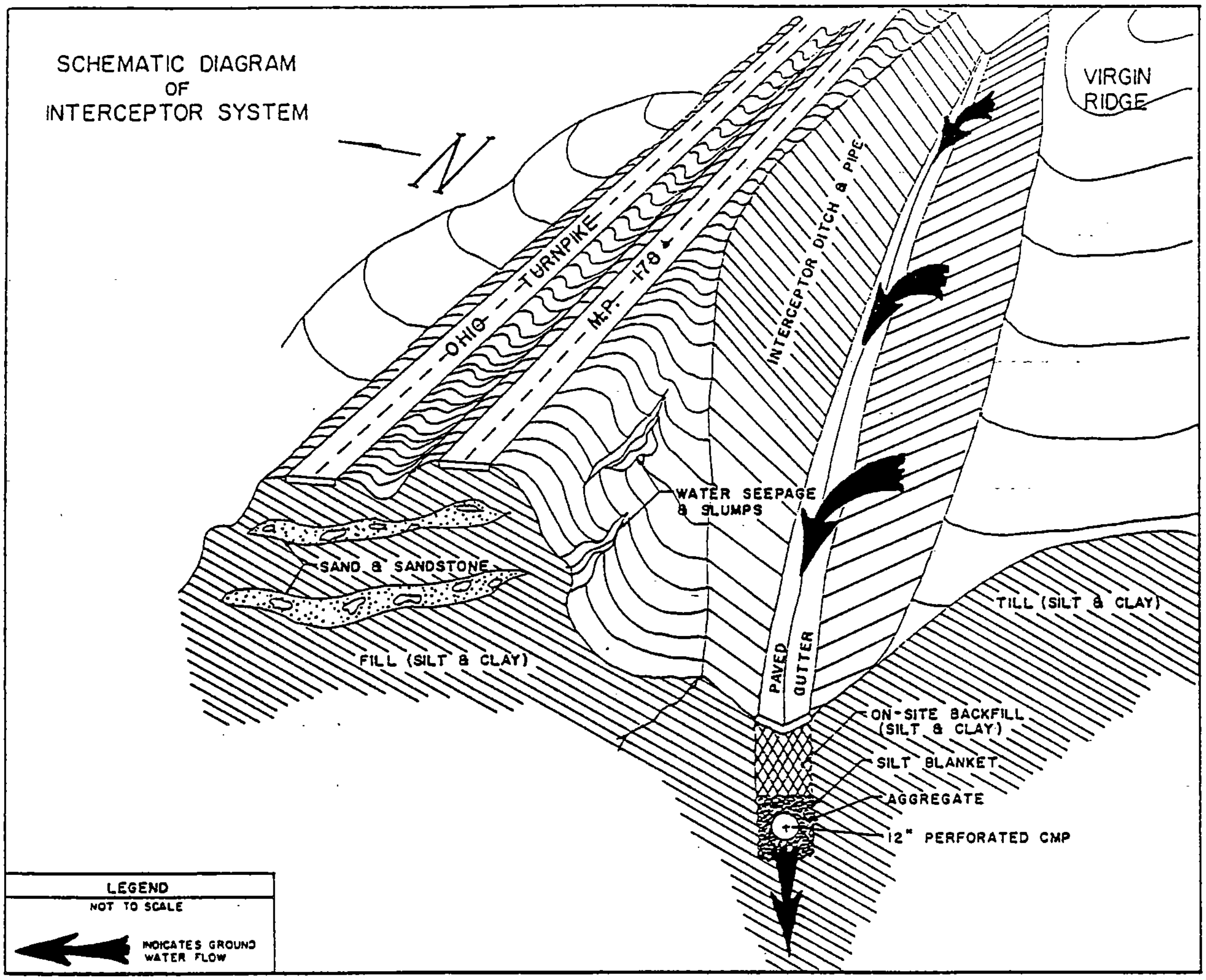

FIGURE 5 - SCHEMATIC DIAGRAM OF INTERCEPTOR SYSTEM

REMEDIAL MEASURES (Continued)

would be most cost effective to implement the total corrective measures in phases. The first phase would consist of installing the interceptor system and evaluating its effectiveness. In this case logic dictated the approach taken. If the interceptor system had failed to dry the slopes the subsequent corrective measures would also have eventually failed and substantially increased the total monies lost on insufficient or inappropriate remedies. In addition, if all corrective measures are implemented simultaneously, it is difficult to assess which action corrected which problem and to what extent. Such information is vital in focusing attention on the appropriate corrective measures to adopt if specific aspects of the problem recur.

INTERCEPTOR SYSTEM

CONSTRUCTION OBSERVATIONS

Subsurface soil conditions were carefully monitored during excavation of the interceptor drain. It was noted that the fracture zone encountered in the test borings was continuous both laterally and vertically. Oxidation, indicated by a brownish color in the dominantly gray till, was evident along individual fractures for a width of one (1) to several centimeters.

It was anticipated that during construction of the interceptor drain groundwater may seep slowly from the fractures. While this light seepage was typical, at a few locations along the trench line groundwater poured into the trench at an extremely high rate of flow. This indicated that the interceptor drain had intersected preferred groundwater flow paths through the fractures at these locations.

DISCUSSION OF EFFECTIVENESS

In May of 1988, R & R International completed a 1 1/2 year long study of the effectiveness of the interceptor system. This study primarily involved monitoring of groundwater levels within the piezometers installed during our initial investigation and periodically checking the flow exiting the interceptor pipe. The amount of ground water within the fill has slowly but steadily declined except for a period during the summer of 1987 when unrelated turnpike construction denuded portions of the fill and disrupted normal drainage patterns.

The flow discharging from the interceptor pipe has been low but steady. The average flow has been measured at 0.20 gallons per minute. The flow rate appears to be virtually unaffected by seasonal fluctuations in precipitation. The flow was steady throughout the dry summer months of 1987.

Seepages still exist along the fill slopes despite the reduction in ground water measured within the fill. This suggests the possibility of additional ground water entry points further east along the turnpike. A lateral draining from south to north beneath the turnpike immediately east of the interceptor drain has been documented to be discharging greater volumes of water than are entering at its intake. This indicates that the lateral is picking up ground water flow from the east. Further investigation of this possibility is warranted. Depending upon the outcome of such an investigation, it is suspected that an additional ground water interceptor will have to be tunneled beneath the turnpike to intercept eastward groundwater influx into the valley fill of the highway.

CONCLUSIONS

The two primary objectives of this paper were to provide a narrative description of the investigative methodology and of the remedial measures implemented on an actual highway fill instability project so that the effectiveness of both could be evaluated. We believe that the case study presented herewith clearly demonstrates the value of a phased investigation and implementation approach. This is especially true in investigations involving a complex interaction of unknown subsurface variables. Information from one phase of the investigation should be evaluated to determine its effect on the focus of the study and future tasks redirected accordingly.

After the causes of a geotechnical problem are clearly understood, the necessary remedial measures can be logically developed. The solution adopted for this particular site effectively diverted the groundwater away from its entry point into the turnpike fill. The highway fill is now experiencing a drying trend that should enable remediation of the slope instability which initiated the first engineering study many years ago.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the cooperation of the Ohio Turnpike Commission and URS Consultants Inc. of Akron, Ohio in releasing the data used in the preparation of this paper.

ORIENTED PRE-SPLIT FOR CONTROLLING ROCK SLIDES

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ABSTRACT

WEDGE FAILURE LANDSLIDES HAVE REPEATEDLY COVERED PORTIONS OF INTERSTATE 40 IN THE MOUNTAINOUS SECTIONS OF EAST TENNESSEE. THESE WEDGE FAILURES ARE THE RESULT OF BOTH UNFAVORABLE GEOLOGIC CONDITIONS AND BLASTING DAMAGE INCURRED DURING CONSTRUCTION.

DUE TO TOPOGRAPHIC CONSTRAINTS AND TRAFFIC CONTROL REQUIREMENTS. UNIQUE AND INNOVATIVE REMEDIAL TECHNIQUES WERE NECESSSITATED. THE USE OF ORIENTED PRE-SPLIT BLASTING WAS EMPLOYED FOR REPAIR OF THESE WEDGE FAILURE PRONE ROCK SLOPES.

THE INTENT OF THIS PROCEDURE IS TO REMOVE EXISTING WEDGES OF UNSTABLE ROCK BY DEVELOPING A PRE-SPLIT FACE ORIENTED TO INTERCEPT STABLE PLANES OF BEDROCK. CONSTRUCTION PLANS DETAILING GEOTECHNICAL ASPECTS OF THE PROJECT WERE DEVELOPED. TO INSURE A BETTER DEGREE OF COMMUNICATION BETWEEN THE CONTRACTOR AND STATE FORCES. PHOTO MOSAICS OF THE SUBJECT ROCK SLOPES WERE INCLUDED ON THE PLANS.

DETAILS ABOUT THE ORIENTED PRE-SPLIT (INCLUDING LOCATION OF THE PRE-SPLIT LINE. HOLE SPACING. STEMING. AND HOLE DIAMETER) WERE OUTLINED ON THE CONSTRUCTION DOCUMENTS. IN ADDITION TO THE ORIENTED PRE-SPLIT PROCEDURES. ROCK SLOPE PROTECTION TECHNIQUES INCLUDING POST-TENSIONED ROCK BOLTS. DRAPED WIRE MESH. AND ROCK FALL CONTROL FENCES WERE EMPLOYED.

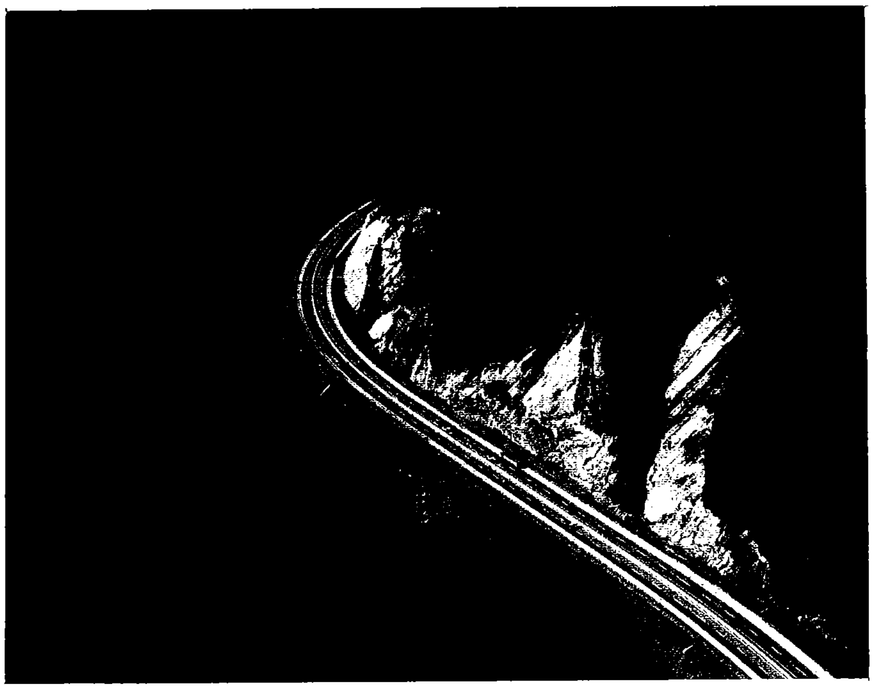
THE USE OF ORIENTED PRE-SPLIT FOR CONTROL OF ROCK SLIDES SHOWS PROMISE AS A VALID GEOTECHNICAL REMEDIAL TECHNIQUE. CONTINUED DOCUMENTATION OF NEW GEOTECHNICAL REMEDIAL CONCEPTS IS REQUIRED TO FURTHER THE SCIENCE.

INTRODUCTION

Wedge failure landslides have repeatedly covered portions of Interstate 40 in the mountainous sections of East Tennessee. These wedge failures are the result of both unfavorable geologic conditions and blasting damage incurred during construction. As a result of these repeated landslides. the existing rock slopes have deteriorated severely to become very precipitious and hazardous.

The repair of these rock slopes has resulted in the use of a number of unique remedial concepts (Moore. 1986). These include draped wire meshing. trimming and scaling. catchment fences. and rock bolting.

A unique method of trimming the landslide and rockfall-prone slopes was implemented. This concept involved the use of oriented pre-split, rock faces to effect a stable slope in a controlled situation.



The project cut slope along Interstate 40 in Cocke County, Tennessee as it appeared in January of 1974.

The purpose of this paper is to document the use of an oriented pre-split concept in the control of landslide and rock-fall prone highway cut slopes.

The oriented pre-split concept has been successfully employed to provide stability of rock fall prone highway cut slopes. The project described in this text is the first such implementation of the oriented pre-split concept in Tennessee. Detailed in this narrative are conceptual design parameters, actual construction techniques and procedures, and a discussion of problems/concerns/benefits surrounding the oriented pre-split concept.

LOCATION

The location of this project is in the mountainous section of East Tennessee where Interstate 40 traverses the Blue Ridge Province .

The project site is specifically located 3/10 mile past the Interstate mile marker 449 (Sta. 1520) between the Hartford Exit in Tennessee and the North Carolina/Tennessee State Line.

This section of Interstate 40 has been plagued with a history of catastrophic landslides and rock falls (Moore, 1986). Most of these landslides can be classified as wedge failures which involve wedges of rock, soil, and vegetation which slide along the line of intersection of two planar discontinuities.

At least six of these wedge failures have closed two lanes of I-40 for up to 2 weeks at a time . However, since the completion of this rock slope remedial project there have been no rock fall related closures of that section of Interstate 40.

The project area is underlain by Pre-cambrian age rocks of the Roaring Fork Sandstone. The strata are composed chiefly of slightly metamorphosed siltstone and sandstone which strike N 78 degrees E with a dip of 42 degrees to the southeast. Structural influences from nearby thrust faults have resulted in numerous fractures (both joints and cleavage). It is the combination of tilted strata and large highway cutslopes that has produced the serious rockfall problems along this stretch of Interstate 40.

CONCEPTUAL DESIGN

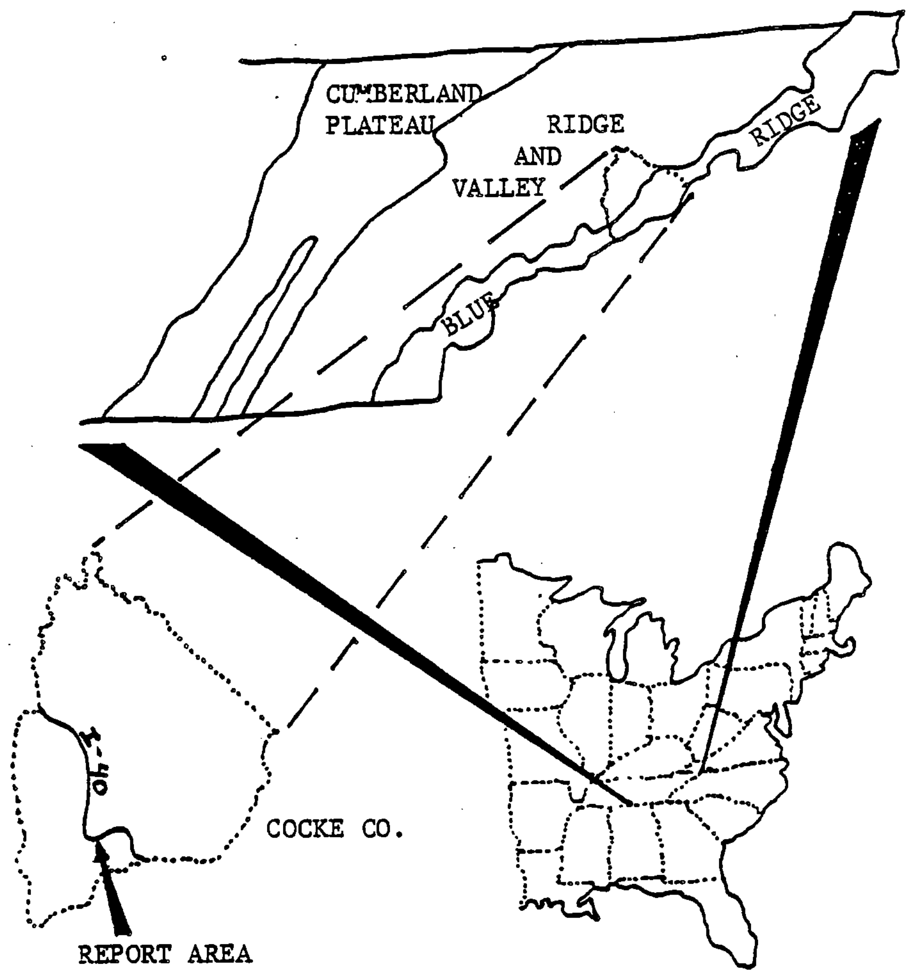
To effectively use the oriented pre-split concept, detailed information about the site geology must be obtained. Numerous strike and dip measurements of the bedrock discontinuities are required. Additional geologic data that are needed includes mode of slope failure, groundwater conditions, lithology type, and bedding thickness. The use of steronet structural analysis and stability calculations are necessary in the development of an overall remedial plan and application of the pre-split concept.

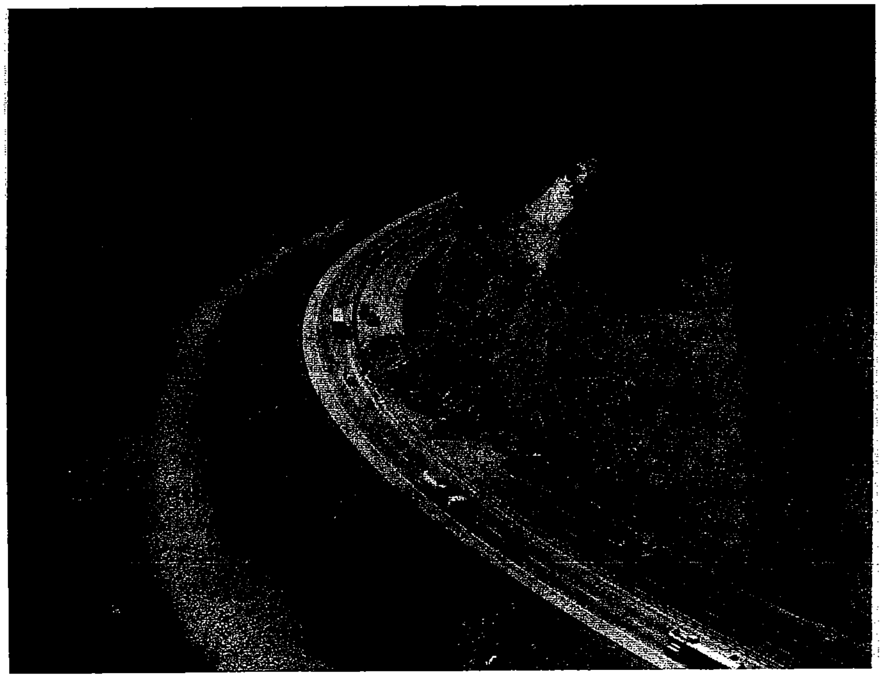
The geologic data is transferred to survey information (plan sheets) for proper orientation relative to the roadway. This is accomplished by using photo mosaics and plan layout sheets of the subject roadway.

The location of the pre-split faces are based upon the intersection of two lines: the strike line (daylight) of the major

LOCATION OF REPORT AREA

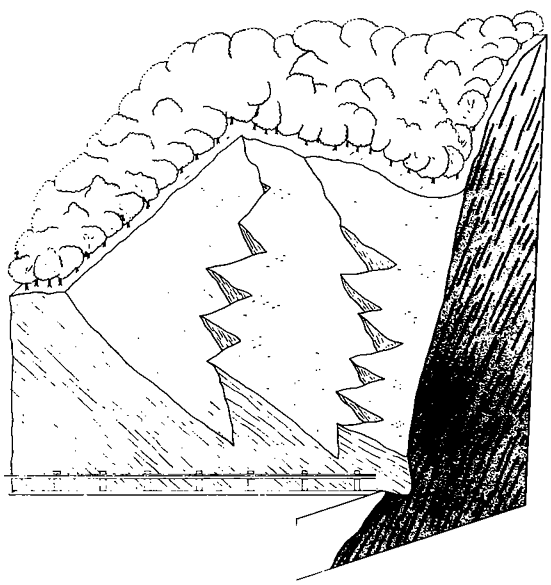
COCKE COUNTY, TENNESSEE





A rock slide in the project area closed the two west bound lanes of Interstate 40 in July of 1976.

EXISTING ROCK SLOPE



I-40 COCKE COUNTY

Illustrated is a schematic block diagram of a rock-fall prone cut slope.

discontinuity (usually a bedding plane) and an imaginary line oriented along the surface which will effect the minimum removal of unstable rock. The pre-split face is carried down to the subject discontinuity and is oriented toward the release plane and axis of movement of the unstable rock.

The pre-split is an ordinary pre-split design with hole spacings of 24" or smaller. Based upon the kind of rock and depth of the pre-split hole,the blasting factors are then derived including hole diameter and blasting agents.

Briefly, the oriented pre-split concept is outlined as follows:

*Geotechnical analysis of rock slope.
*Design of pre-split orientation and blasting plan.
*Field layout of oriented pre-split line.
*Drilling and blasting of pre-split face.
*Removal and clean-up of shot rock.
*Additional production blasting (necessary if pre-split face does not remove unstable rock).
*Installation of rock bolts or horizontal drains.
*Installation of rock slope protection devices (wire meshing, catchment fences)

The main purpose of using the oriented pre-split concept is to remove unstable distressed rock in a controlled situation. The pre-split face (which is used in removing the unstable rock) results in a clean rock slope which is less prone to evolve into an unstable, precipitious rock face condition.

PROJECT HISTORY

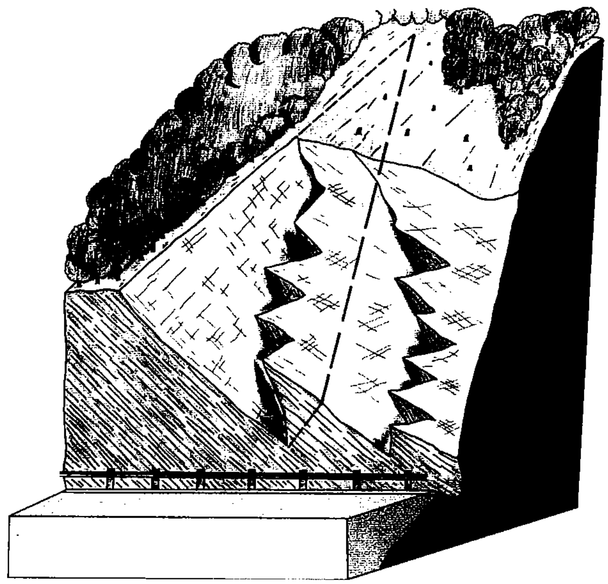
During the late sixties construction of Interstate 40 through the mountainous Blue Ridge section of East Tennessee was nearing completion. The new interstate was an improvement for traffic between East Tennessee and Western North Carolina where tourism and industry were increasing.

Rock slides began to plague the new stretch of interstate in the early to mid seventies and continued until the completion of a major rock slope remedial project in 1986.

Part of that remedial project included the use of oriented pre-split for the correction of a very precipitious rock cut. The subject rock cut had experienced a number of rock slides requiring temporary closure of the interstate. The rock slides were the result of wedge and plane failures which had developed due to the roadway geometry and the rock structure.

It was decided that the best remedial approach would be to remove the existing wedges of overhanging rock in a controlled situation. The oriented pre-split concept was selected as the appropriate remedial approach. Additional geotechnical measures applied to the subject cut slope included scaling, rock bolting, wire meshing, and horizontal drains. Project plans were developed which included the use of photo mosaics of each subject cut aslope. Construction details regarding the oriented pre-split line, rock bolting, scaling, and wire meshing were drawn onto the photo mosaics to insure a better degree of communication

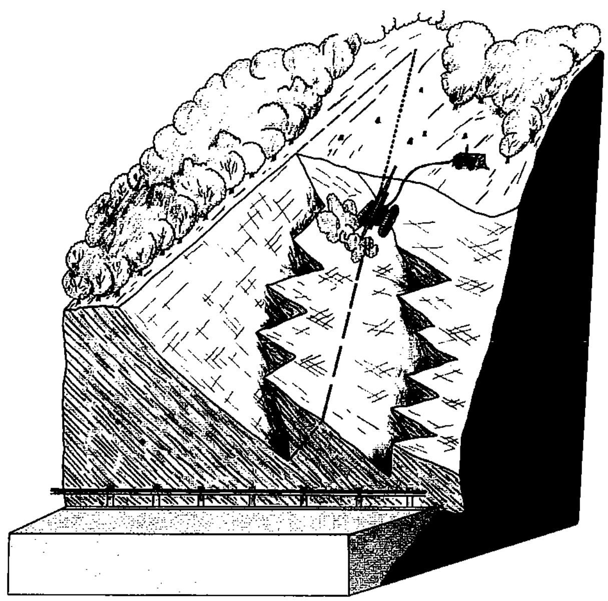
ORIENTED
PRE-SPLIT LINE



I-40 COCKE COUNTY

Above is a drawing of the project cut slope with the oriented pre-split line illustrated.

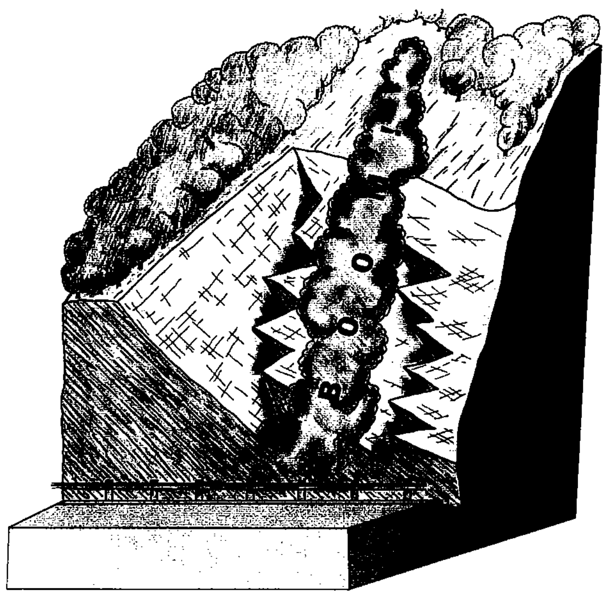
DRILLING
PRE-SPLIT FACE



I-40 COCKE COUNTY

This schematic drawing shows the drilling of the oriented pre-split line.

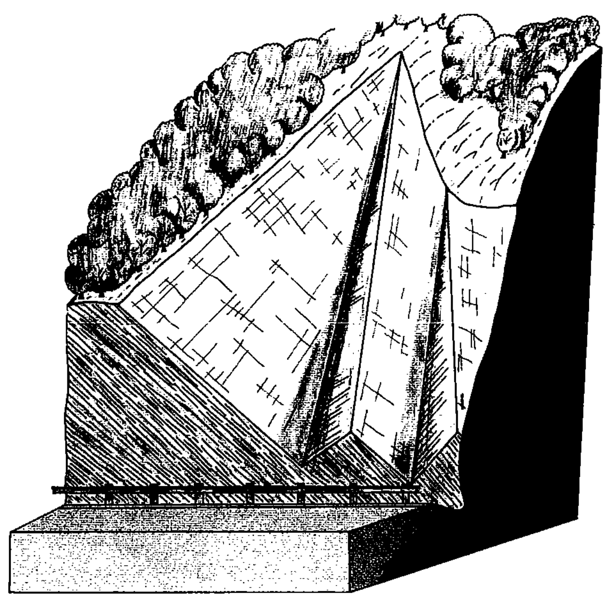
BLASTING OF PRE-SPLIT FACE



I-40 COCKE COUNTY

This schematic diagram illustrates the blasting of the oriented pre-split face.

COMPLETED ORIENTED PRE-SPLIT FACE



I-40 COCKE COUNTY

This three dimensional block diagram shows a repaired rock cut using oriented pre-split.

between the contractor and state forces. In addition, conventional plans and cross-sections were included as part of the contract plans.

The project was let to contract in the summer of 1985. Cut slopes along three miles of interstate which required scaling, trimming, wire meshing, rock bolting and horizontal drains were included in the contract which was let for just over 9 million dollars.

The subject cut slope required six oriented pre-split faces to remove the unstable material. Approximately 30,000 cubic yards of material was removed requiring about 1700 square yards of pre-split.

A pre-split hole spacing of 24 inches was used. In addition, Hercudet pre-split powder WR78x24 (Herco-split) was loaded in every other pre-split hole (along with primer cord and one cap per hole).

The cost of the pre-split was approximately $16.00 per square yard plus an additional $3.32 per ton for hauling the rock to the waste area. Approximately $250,000.00 was required for the oriented pre-splitting and removal of rock from this cut section. Additional costs for wire meshing, rock bolts, and horizontal drains elevated the total costs for the subject cut slope to over $300,000.00.

After the initial clearing and surveying was complete the contractor began drilling the pre-split holes according to a pre-determined layout and drilling sequence. Due to the geometry of the cut slope and the stair-step arrangement of the failure planes, the oriented pre-split procedure was initiated in stages. The pre-splitting began on the right side of the cut and progressed to the left side (from the highest failure plane to the lowest).

Air track drills were wenched down from the top of the cut where they drilled the pre-split holes as they progressed down the slope. In some instances the rock slope was innaccessible to the wenched air track drills. Consequently, crane suspended platforms were used to provide access for drilling some of the more precipitious slopes.

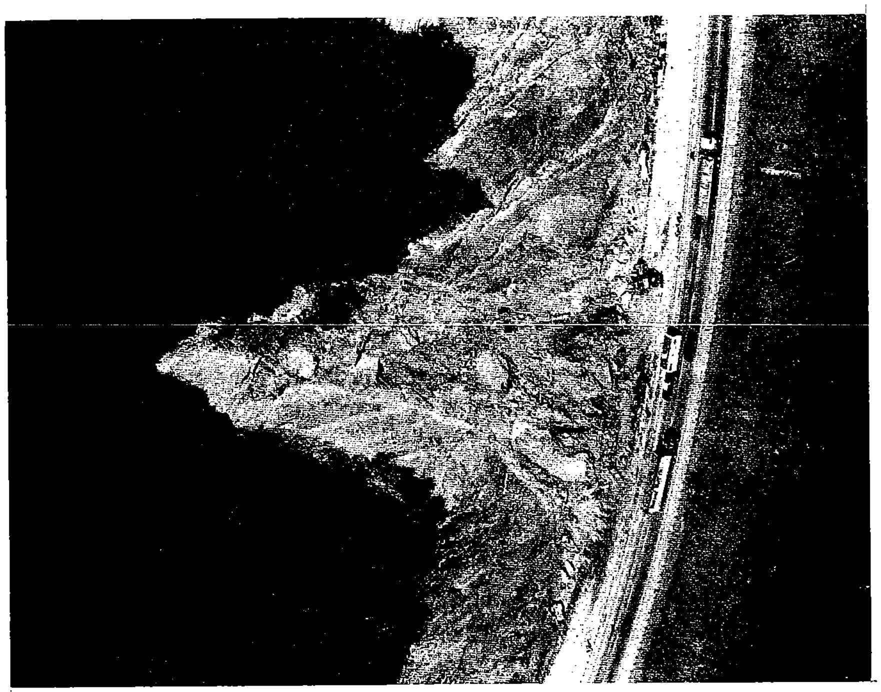
Due to the size of the rock wedges, production blast holes were also drilled. The "production holes" were located no closer than 8 feet from the pre-split face and varied in depth up to 30 feet.

A total of three of the six rock wedges were liberated from the rock cut with only the use of the pre-split blasting. Where production blasting was used, care was taken to prevent damage to the pre-split face by using delays.

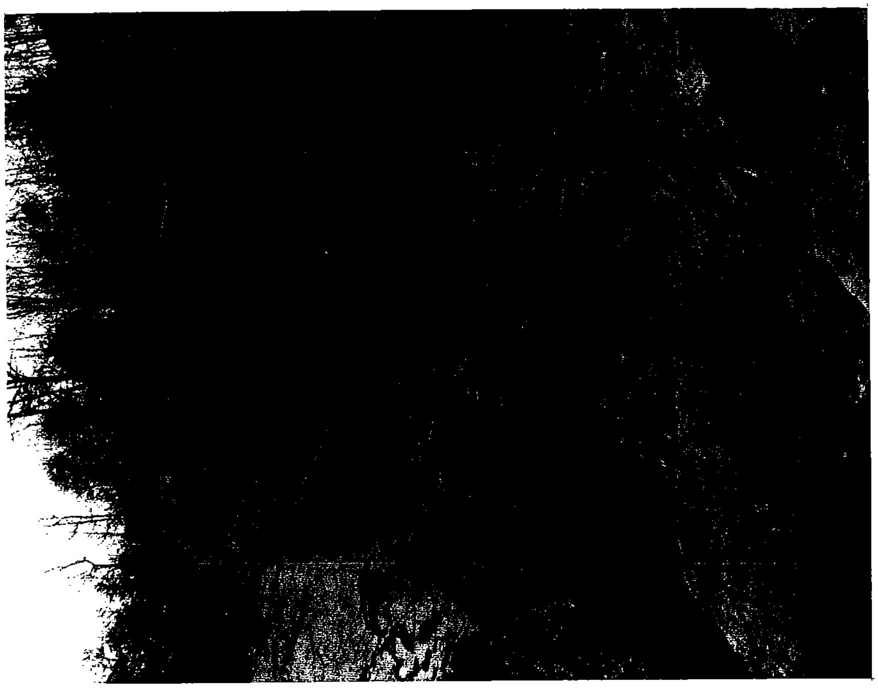
Total removal of the unstable rock material was accomplished within one working season. The installation of draped wire meshing, rock bolts, and horizontal drains completed the rock slope remedial work at the subject site. There have been no major rock slides and only one significant rockfall to date. The protection measures installed during this project more than adequately handled the minor rock fall which has occurred since the project was finalized.

PROBLEMS/CONCERNS/BENEFITS

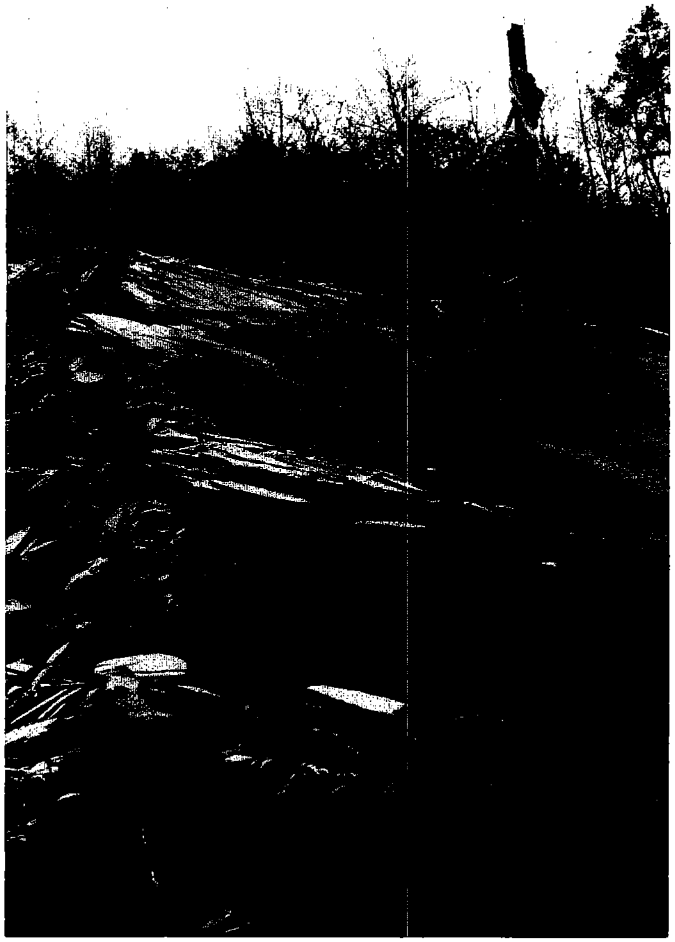
It is necessary to properly evaluate new geotechnical concepts to further enhance the science. Certain problems, concerns, and benefits arise from any new procedure and a discussion of such matters is worthwhile.



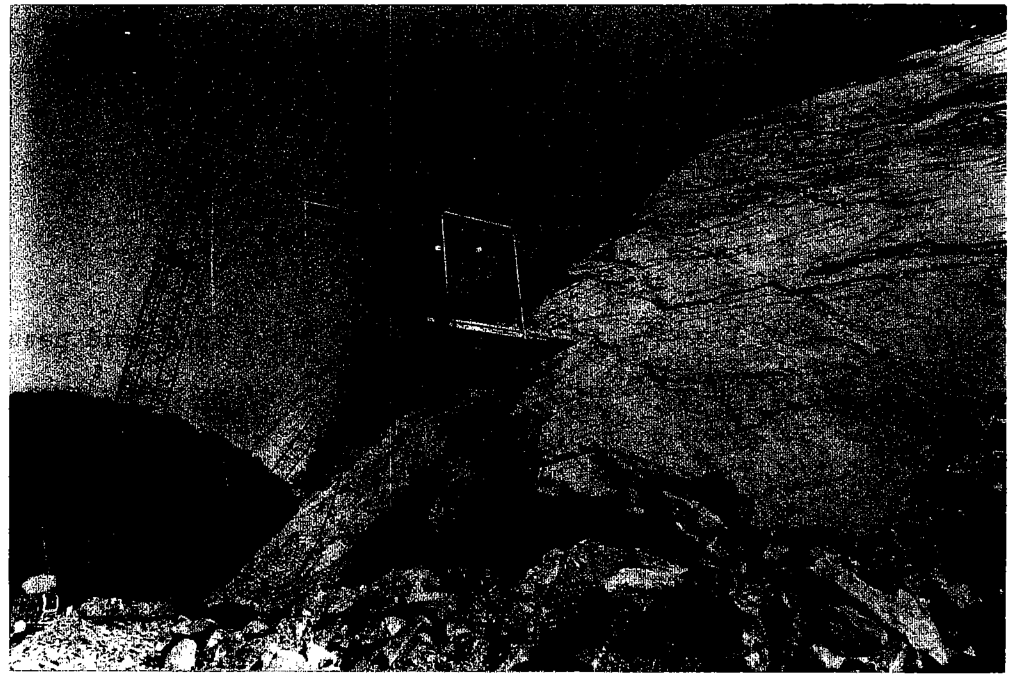
Two lanes of Interstate 40 were closed for two weeks when this rock slide occurred in March of 1977.



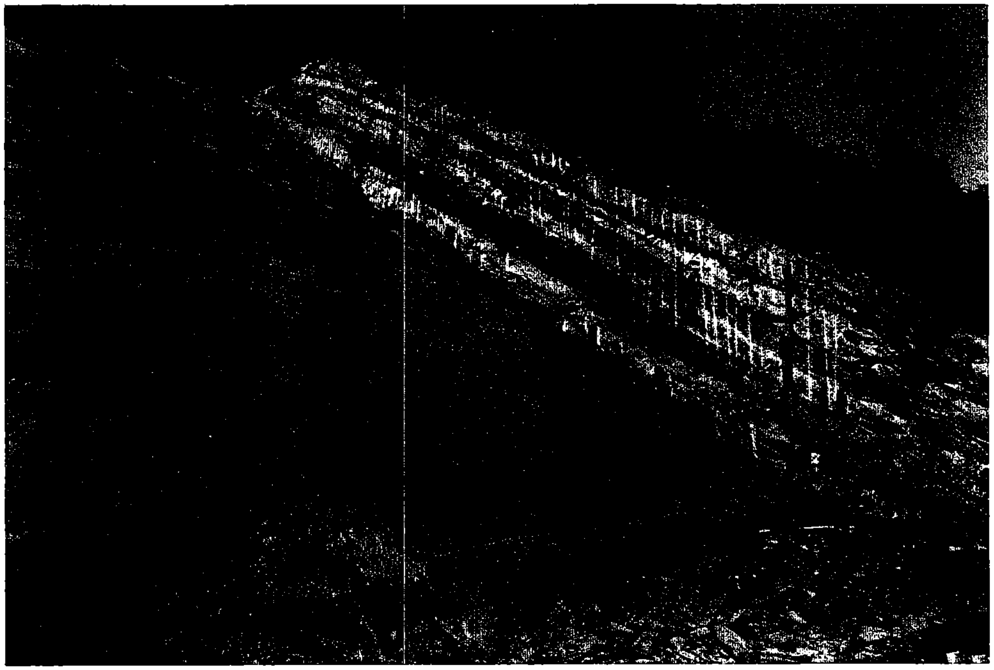
This precipitious rock slope, typical of the project area, was repaired using the oriented pre-split concept.



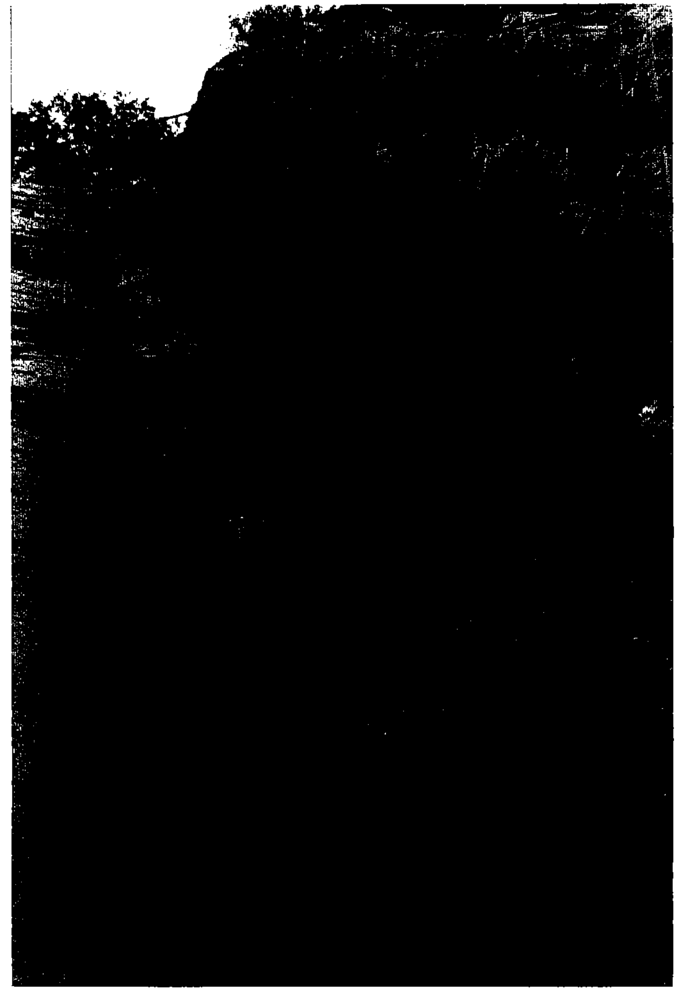
Air track drills were wenched down the subject rock slope in order to drill the oriented pre-split face.



Some of the rock slopes required the use of crane suspended platforms to drill the oriented pre-split holes.



This photograph shows one of the six oriented pre-split faces constructed on the subject cut slope.



Protection devices such as wire meshing and catchment fences were installed upon completion of the oriented pre-split remedial work.

One of the major problems about using oriented pre-split is determining the location of the failure plane to be intercepted by the pre-shear face. Exact cross-sections of the ground surface along the proposed pre-split line is mandatory. A three-dimensional understanding of the rock slope is required in establishing the location of the failure plane. However well the measurements are taken, such things as drift of the drill steel, unexpected bedrock structure, and misfire of the blasting elements can all contribute to a poorly located and executed oriented pre-split face.

A major concern regarding this technique is the possible sudden release of the unstable rock mass once the pre-split face has been developed. The concern arises over the fact that the roadway may be completely covered by the immediate fall of the rock mass. Roadway protection devices such as portable catchment fences and barriers must be used to prevent injury to the motoring public. Blasting on an open rockface along a highway is, of course, always a concern.

One of the major benefits of using oriented pre-split to control rock fall problems is that it can be accomplished (with certain safeguards) while maintaining traffic. By using crane suspended platforms and adequate protection fences, drilling of the pre-split face can be performed without major disruption to traffic flow. In addition, controlled blasting techniques can minimize any impact on the roadway and traffic.

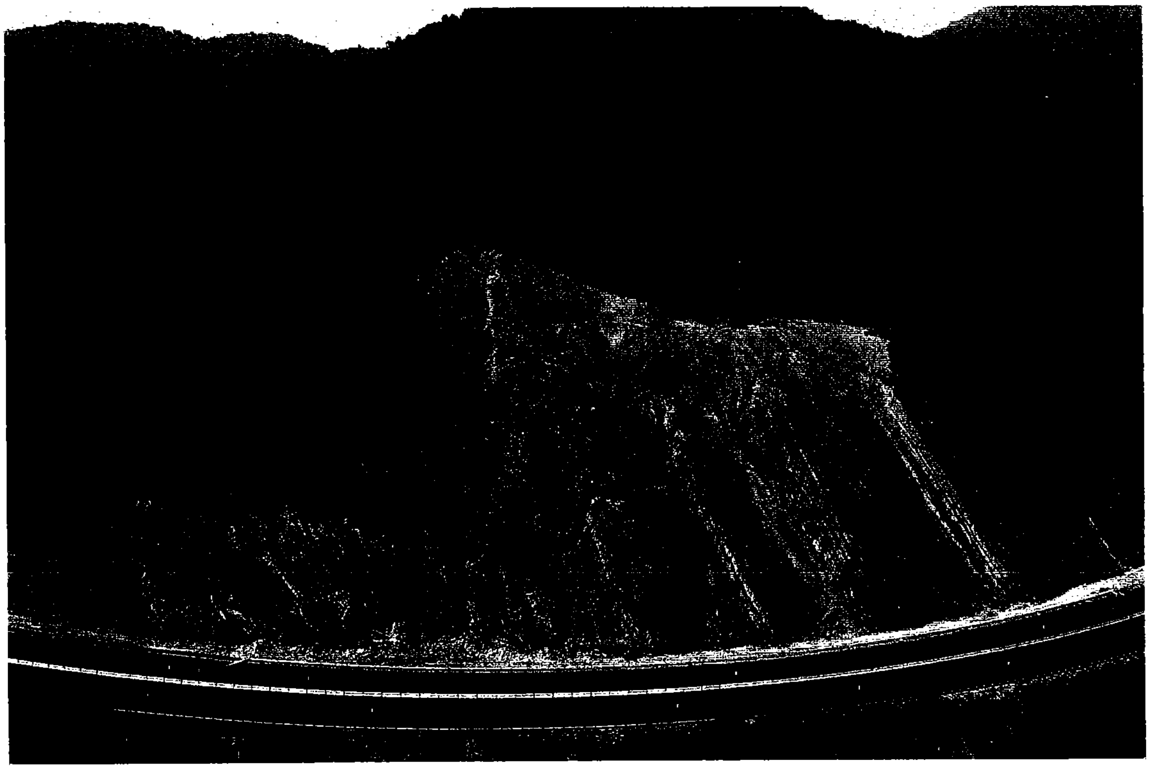
The use of oriented pre-split can also prevent the unnecessary removal of volumes of stable bedrock (on the subject project the excavation quantities would have easily quadrupled by using a uniform stable slope ratio which would have been flatter than the finished slope). By designating only the unstable rock masses for removal, the volume of excavation can be kept to a minimum. However, the final appearance of the rock slope will be somewhat unorthodox.

An additional benefit of this project was the successful use of photo mosaics which were incorporated in the design and construction plans. Details of the oriented pre-split concept (along with the other geotechnical remedial measures) were printed on a photo mosaic of each cut slope. This provided a clear and precise picture of the intended remedial work to both the contractor and state personnel.

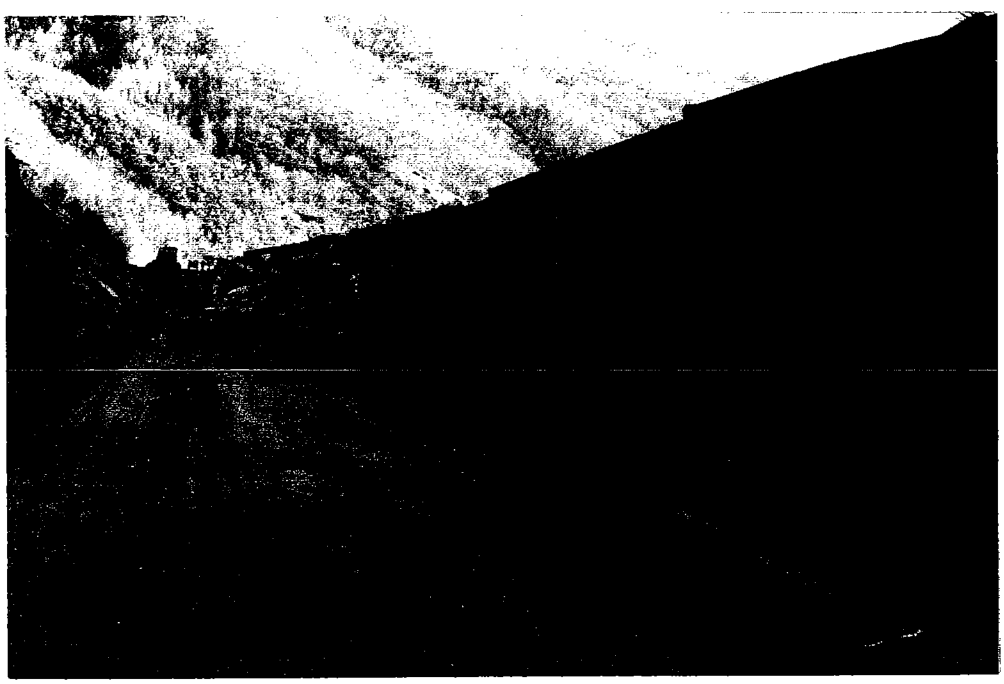
CONCLUSIONS

The use of oriented pre-split to control the occurrence of rock fall has been effectively implemented along Interstate 40 in the Blue Ridge Province of East Tennessee. Proper rock slope engineering evaluation and design are necessary in order to achieve adequate results.

As in all new innovative techniques there will be certain aspects of the procedure that need to be re-evaluated. However, based upon the results to date the oriented pre-split concept has proven to be a unique and effective method of controlling rock fall problems. The oriented pre-split technique is not being advanced as a "cure-all" for rock fall problems. However, one of the purposes of this paper is to document the



This aerial photograph shows the project cut slope after completion of the oriented pre-split remedial work.



Protection devices such as these metal portable catchment fences were used while implementing the oriented pre-split concept. Traffic was maintained during the construction period.

use of new geotechnical concepts which may be considered as possible remedial techniques.

ACKNOWLEDGEMENTS

The writing of this paper required the help of valuable persons who I would like to acknowledge. Judy Sayne typed the manuscript for which I am gratefully thankful. The many drawings which appear in this paper were drafted by Jeff Snyder to who I owe thanks. The many aerial photographs were made by George Hornal whose expertise is gratefully acknowledged. I am thankful to Jim Aycock who patiently proof read the manuscript and offered constructive comments.

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INSTALLATION OF AN UNDERDRAIN FOR SLOPE STABILITY

HIGHWAY 191, NEAR JACKSON HOLE, WYOMING

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ABSTRACT

A landslide along Highway 191, near Jackson Hole, Wyoming, has been causing movement of a section of roadway since 1931. Following a Wyoming Highway Department investigation in 1967, an underdrain system and berm were installed in an attempt to achieve stability of the roadway but proved to be ineffective. Further movement of the road embankment initiated a second investigation in 1983. Due to land ownership, limited funding, and environmental restrictions, it was decided to concentrate on stabilizing the roadway area only. Five methods of stabilization were considered as a result of the second investigation. A slope stability analysis determined that the most effective method would be construction of a much longer and deeper underdrain than the original design.

The underdrain system design called for a trench, 12-25 feet deep and 380 feet long to be constructed with 6 inch pipe and drain gravel, all enveloped in filter fabric. The underdrain would outlet below the roadway at its north and south ends via two pipes drilled through the existing roadway embankment. Two receiving pits would be excavated above the roadway to facilitate connection to the underdrain pipe and to aid in the location and testing of the outlets.

Construction for the underdrain began in April of 1987. Completion of the drilling for the north and south outlet pipes was achieved in two weeks. The underdrain trench was constructed by first making a 8-12 foot deep dozer cut and then excavating the remaining depth to flowline with a backhoe. Completion of the gravel/fabric envelope was achieved with the use of a steel box for trench support. The envelope was built up within the trench in two to four lifts of the steel box. Caving of the trench created problems during construction that were not addressed in the design stage.

After completion of the underdrain system, monitoring holes were drilled and indicated that the water table had been lowered close to the underdrain's flowline. Since the installation of the drain, the roadway and embankment have stabilized. Construction of the drain was completed in June, 1987, at a total cost of $92,000.

INTRODUCTION

This landslide is situated in Northwest Wyoming approximately twelve miles south of Jackson Hole along Highway 191, adjacent to the Snake River (Figure 1). The slide is approximately 4500 feet long x 1200 feet wide and consists of a series of large slump blocks (Figure 2). A 750 foot long x 50 foot high roadway embankment is situated near the base of the slide. This embankment and roadway have been unstable ever since their construction in 1931. Investigation and stabilization of the roadway/embankment area is the focus of this report.

LANDSLIDE DESCRIPTION AND GEOLOGY

The area surrounding this particular landslide is mapped almost entirely as Quaternary Landslide deposits (Figure 3). The landslide deposits are derived from the Cretaceous Age Bear River Formation which consists of a dark gray to black siliceous shale and siltstone containing thin bentonite zones. Intact bedrock outcrops in the area dip toward the roadway at 8 to 10 degrees. Field studies have shown that the slide debris material ranges from 10 to 50 feet thick in the vicinity of the roadway.

HISTORICAL BACKGROUND AND EXPLORATION ACTIVITIES

Seven horizontal drains were installed by the Bureau of Public Roads from the toe of the slide to beneath the roadway around 1940. The drains initially had some effectiveness, but plugged off not long after their installation. The

Geology Section of the Wyoming Highway Department first investigated the slide area in 1967. A perforated trench drain system and earth berm were recommended as a result of the investigation. However, the drain was not installed deep enough and the berm was not large enough to stabilize the area. Continued movement of the roadway created a constant patching problem for Highway Maintenance Crews and initiated a second investigation by Wyoming Geology personnel in 1983. The area of investigation was limited to 200 feet above and 300 feet below the roadway due to land ownership restrictions and impact to this environmentally sensitive area. Fourteen test holes were drilled and cased to determine the slide plane profile and to monitor the ground water level (Figure 4).

STABILITY AND STRENGTH PARAMETERS

Slope stability analysis was performed with the PCSTABL4 program utilizing the simplified Janbu method. Failure parameters were determined by back analyzing to a safety factor of 1.0 along a slide plane profile determined from drill hole data. This resulted in a phi=13.5 degrees and a cohesion of 0 psf for the material along the slide plane (Figure 5).

REMEDIAL ALTERNATIVES

Limited funding was an additional constraint along with land ownership and environmental restrictions as previously mentioned. Consequently, it was decided to concentrate on stabilizing the roadway and embankment area only. The five following alternatives were considered in the stabilization of the site.

1. Alignment Shift

Because the toe area (left of centerline) and backslope (right of centerline) lie within active slide material, a shift of the alignment towards either of these directions would not increase the stability of the roadway.

2. Lightweight Embankment

Reconstruction of the existing embankment with wood chips would reduce the weight of the embankment by two-thirds. However, since the embankment constitutes a small percentage of the driving forces, the safety factor was increased by only a negligible amount.

3. Horizontal Drains

Horizontal drains installed randomly throughout the slope could lower the water table and yield a safety factor of 1.2. However, this alternative was not chosen for the following reasons.

a. Drilling would be difficult due to the presence of boulders within the slide debris.

b. Previous Wyoming Highway Department projects have shown that intercepting all the seepage with this method was not successful.

c. Horizontal drains installed previously in 1967 by the Bureau of Public Roads, failed to stabilize the roadway.

4. Toe Berms

A 35 foot high earth berm constructed on top of ground would require approximately 80,000 cubic yards of material and extend from near centerline to the toe. This would result in a safety factor of 1.1. Because of the small amount of safety factor increase and the visual impact, this alternative was eliminated. A rock berm keyed into bedrock was also considered and eliminated because bedrock is 30 feet deep at the toe and trenching for the berm would have to be completed below the water table.

5. Underdrain System

A safety factor of 1.2 could be achieved if an underdrain system was installed and lowered the water table close to bedrock. To achieve this the drain would have to be 6 to 15 feet deeper and 250 feet longer than the drain installed in 1967. This alternative was thought to be a practical method of stabilization and was chosen as the recommended solution. The following are a list of reasons that influenced this decision:

a. The drain achieved a safety factor of 1.2 which was the same as the horizontal drains and higher than the other alternatives.

b. The trench would provide a positive means of intercepting most of the seepage above the roadway.

c. Unlike the rock berm, most of the excavation would be above the water table.

UNDERDRAIN TRENCH CONSTRUCTION

Design

The design called for excavation of a trench right of centerline and the installation of two horizontal outlet pipes beneath the roadway (Figure 6). The trench was to be located in the ditch area 25 to 60 feet east of the roadway where bedrock and ground water were closest to the surface. It was decided that construction would begin with the installation of the horizontal pipes to provide an outlet for water encountered during excavation of the trench. Two receiving pits would be excavated right of centerline to facilitate location and testing of the outlet pipes (Figure 7). The trench would be 380 feet long and parallel the total length of embankment. The underdrain system would consist of a 6 inch PVC pipe and drain gravel all wrapped in a filter fabric envelope. Planned heights of the gravel/fabric envelope ranged from 6 to 14 feet. To minimize caving of backslopes, the top 4 to 12 feet would be removed with a dozer. The remaining depth would be excavated with a backhoe (Figure 8). To keep the trench open during placement of the underdrain materials, it was recommended that the contractor provide some means of trench reinforcement, such as a steel box or shoring. However, this was left up to his discretion.

Depth to bedrock varied from 10 to 30 feet along the length of the trench (Figure 9). It was originally proposed that the flowline of the pipe would be 1 foot below bedrock. However, limited funds restricted trench depths to 25 feet.

A 24 inch corrugated metal pipe installed as an outlet for the 1967 drain would be encountered during construction. Since the pipe had been leaking into the embankment, it was to be sealed off.

Horizontal Outlet Pipe

Construction for the outlets began at the south end in April, 1987, by excavating a pad for the drill on the west side of the roadway and excavation of the receiving pit on the east side. Drilling was accomplished with an auger drill utilizing 5 foot long flights and 6 inch steel casing (Figure 10). Footage achieved with the drill was approximately thirty feet per day and the south outlet was completed in eight days. Drilling for the north outlet took only 3 days, however when the boring was completed, it was 1 foot lower at the receiving pit (inlet end) than the outlet. An additional boring was completed parallel to the first and achieved planned flowline. Springs yielding 0.5 to 2 gallons per minute (gpm) were encountered during drilling.

Underdrain Trench

The contractor began construction for the trench by first making a 8 to 12 foot deep dozer cut along the entire length of the proposed drain. Steep backslopes combined with unusually wet weather caused immediate caving of the roadway at the south receiving pit (Figure 11). To expedite construction of the trench and reduce caving, it was decided to raise the grade of the pipe 4 feet at this location. This resulted in an adjustment of the flowline grade two to four feet above planned heights along the south one-third of the project. Excavation to flowline depth then proceeded south of the south outlet with the backhoe.

The contractor initially attempted to lay the pipe, fabric and gravel in the backhoe trench without shoring. Caving of this trench forced the contractor to use a 5 foot high x 20 foot long steel box for support. Construction of the drain system was then completed by placing fabric inside the box, followed by 4 inches of gravel as bedding and then the 6 inch PVC pipe. Gravel was then backfilled to the top of the 5 foot high box and the fabric was overlapped around the gravel. The box would then be pulled ahead 15 feet to complete another section. The contractor would lay pipe and gravel in this manner for approximately sixty feet. He would then come back and add on two to three more lifts with the box to complete the planned 10 to 15 foot high gravel and fabric section (Figure 12). The remainder of the drain system was completed utilizing this procedure.

It should be noted that caving of the trench backslopes was a continual problem during the installation of the drain. This resulted in trench widths being 1.5 to 2 times the original 3.5 foot design. Since the support box was only 3 feet wide, pulling the box ahead or raising the box vertically to add another lift would result in the fabric and gravel envelope collapsing 1 to 2 feet. This resulted in twice as much fabric and gravel being utilized on the project.

Seeps 1/2 to 10 gpm were encountered randomly while excavating for the trench. This seepage was often observed flowing from the top of 3 inch to 1 foot thick bentonite seams (Figure 13). Any of this seepage that occurred above planned gravel and fabric elevations was tapped and diverted to the main drain by adding additional gravel. After encountering these seeps, their flow would usually diminish in one to two days.

After completion of the project in June 1987, the north outlet was flowing 5 gpm. The south outlet, immediately after construction yielded 0.75 gpm. Because it was felt that this outlet should yield more than this, the outlet was flushed with 3500 gallons of water, at which time a large volume of clay balls flowed out of the pipe. The flow increased to 3 gpm, but then fell back to 0.75 gpm. A week later the outlet was jet flushed, but still did not change the flow. Since flushing the pipe did not increase the flow rate at the outlet, it was felt that the decrease was due to a reduction in the springs output, as had occurred previously.

Monitoring and Construction Costs

Six monitoring holes were drilled in June 1987, on either side of the underdrain to determine it's effectiveness. Monitoring of these indicates that the ground water table is being held at the same level as the flowline of the pipe. Since the installation of the drain, the embankment/slide area has stabilized and no further patching of the roadway has been necessary.

The total cost for construction of the system was $92,000. The cost for drilling the two horizontal pipes was $22,500. Construction of the trench required approximately 1700 cubic yards of trench excavation, 850 cubic yards of drain gravel, 3300 square yards of filtration fabric and 850 lineal feet of 6 inch PVC pipe.

CONCLUSION

When attempting to stabilize landslide problem areas, geotechnical personnel may often encounter many restrictions which may limit the scope of their investigation. Land ownership, environmental impact and funding were all major limitations on this project. In addition, problems often occur during construction that have not been fully addressed in the design phase. On this project, caving of the trench backslopes occurred to a greater extent than realized in design. The result was that the underdrain pipe grade was raised 4 feet over one-third of the projects length. Also, due to caving, the larger trench widths doubled gravel and fabric quantities.

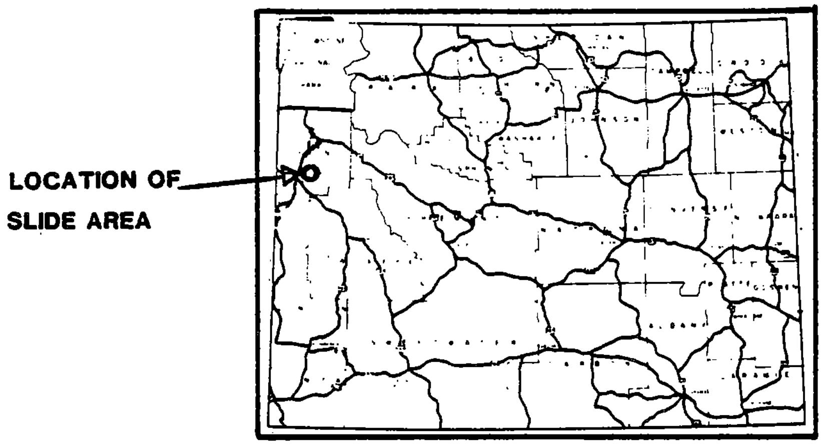
Stabilization of this slide area was possible while still working with the above constraints and problems. However, the problems encountered on this project have demonstrated to Wyoming Highway Department personnel that the design of future deep underdrain systems will require a more adequate method of trench reinforcement.

ACKNOWLEDGMENTS

The author would like to thank Marilyn Foster for typing this paper, Ron Kaiser for helping with drafting of figures, Michael Hager and my very pregnant wife Nancy for reviewing the text.

REFERENCE

Schroeder, M. L., 1974, Geologic Map of the Camp Davis Quadrangle, Wyoming U. S. Geological Quadrangle Map, GQ 1160, Scale 1:24,000



Modified Wyoming State Map

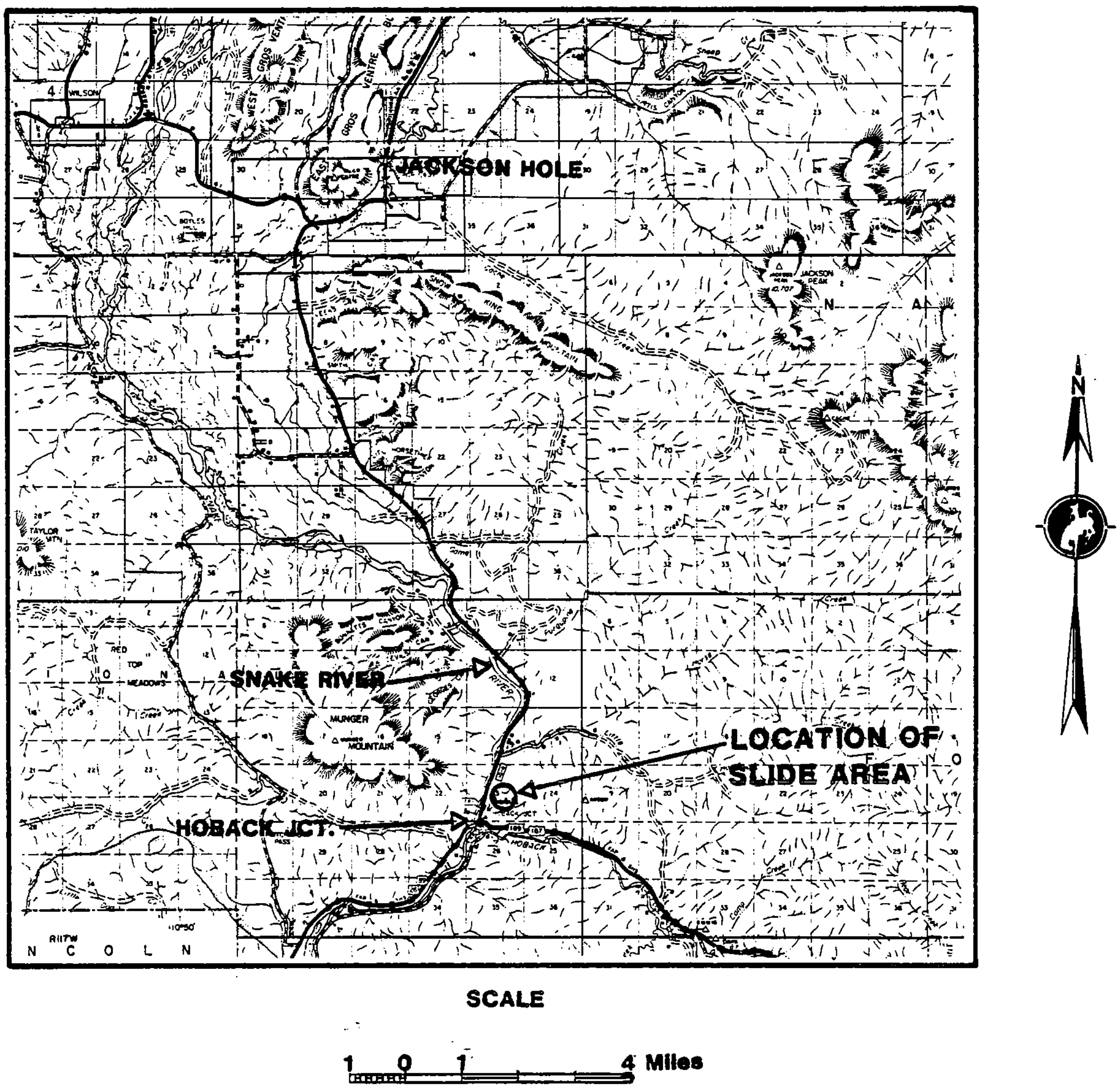


Figure 1. Slide Location Map (Modified from General Highway Map of Teton County, prepared by the Wyoming Highway Department.

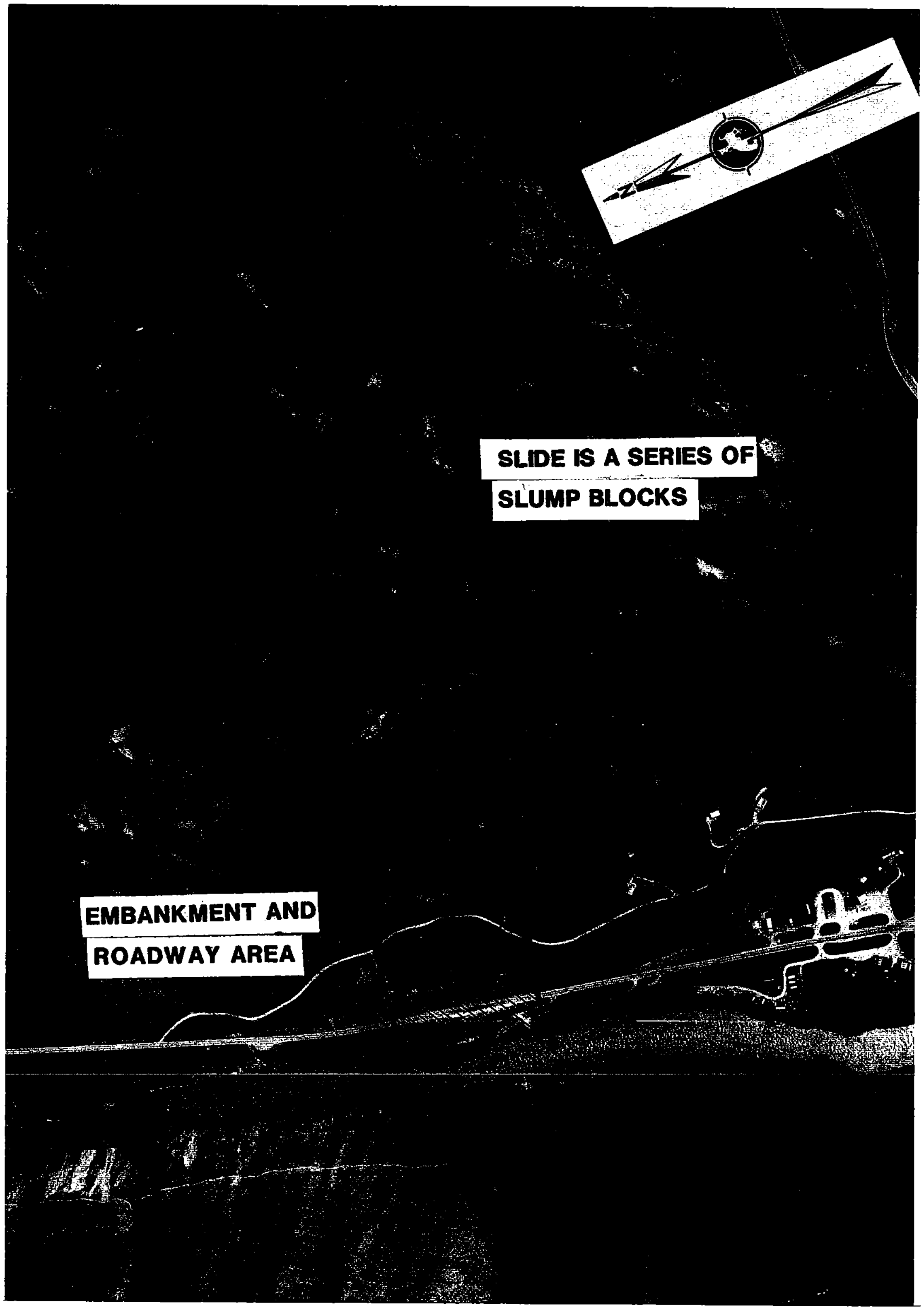


Figure 2. Aerial Photo Of The Slide Area **SCALE 1"=750'**

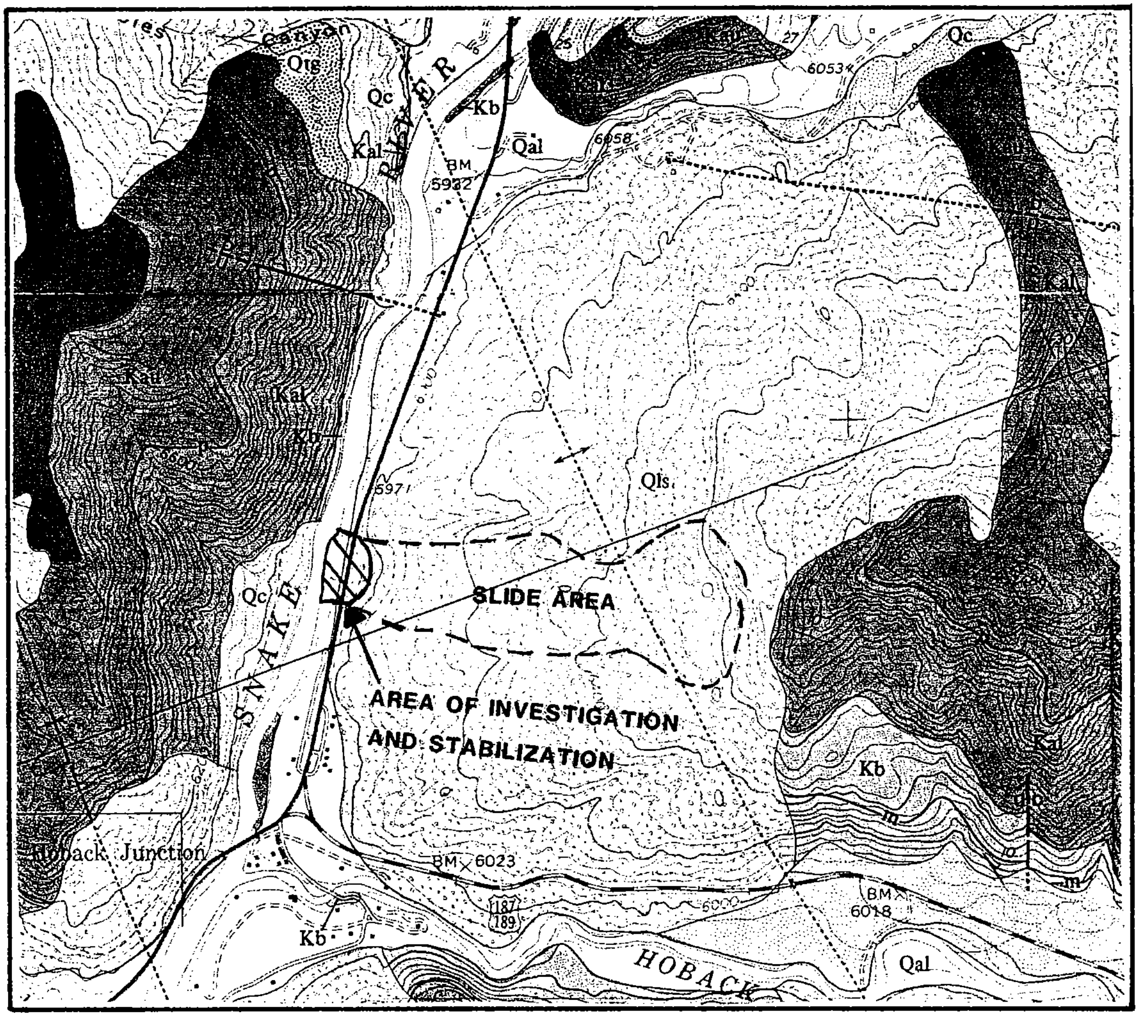


Figure 3. Geology of the Landslide Area (Modification from U.S.G.S. Quad. Map, Schroeder, 1974)

SCALE 1:24 000

0 1 MILE

CONTOUR INTERVAL 40 FEET
DATUM IS MEAN SEA LEVEL

DESCRIPTION OF MAP UNITS

SURFICIAL DEPOSITS (HOLOCENE)

- Qal — Alluvium
- Qc — Colluvium
- Ql — Loess
- Qtg — Terrace deposits
- Qls — Landslide deposits and mudflows

Tcl — CAMP DAVIS FORMATION (PLIOCENE)

Kf — FRONTIER FORMATION (UPPER CRETACEOUS)

Kau, p, Kal — ASPEN FORMATION (LOWER CRETACEOUS)

Kb, m, Kb — BEAR RIVER FORMATION (LOWER CRETACEOUS)

CONTACT – Approximately located

FAULT – Long dash where approximately located; short dash where inferred; dotted where concealed. U, upthrown side; D, downthrown side

ANTICLINE – Showing crestline. Dashed where approximately located; dotted where concealed

SYNCLINE – Showing troughline. Dashed where approximately located; dotted where concealed

STRIKE AND DIP OF BEDS

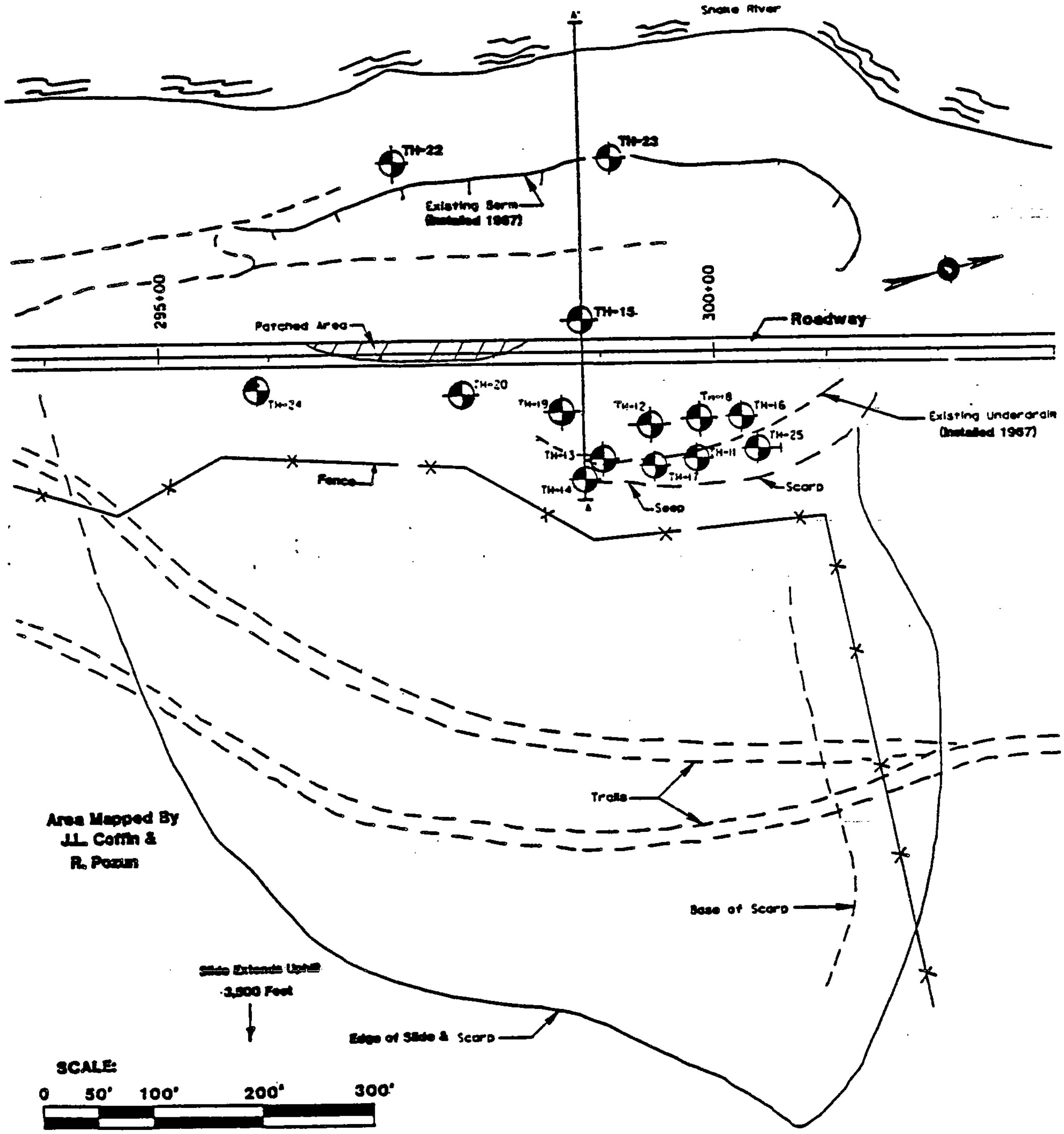


Figure 4. Plan View Of Test Hole Locations

(Including Berm And Underdrain Installed in 1967)

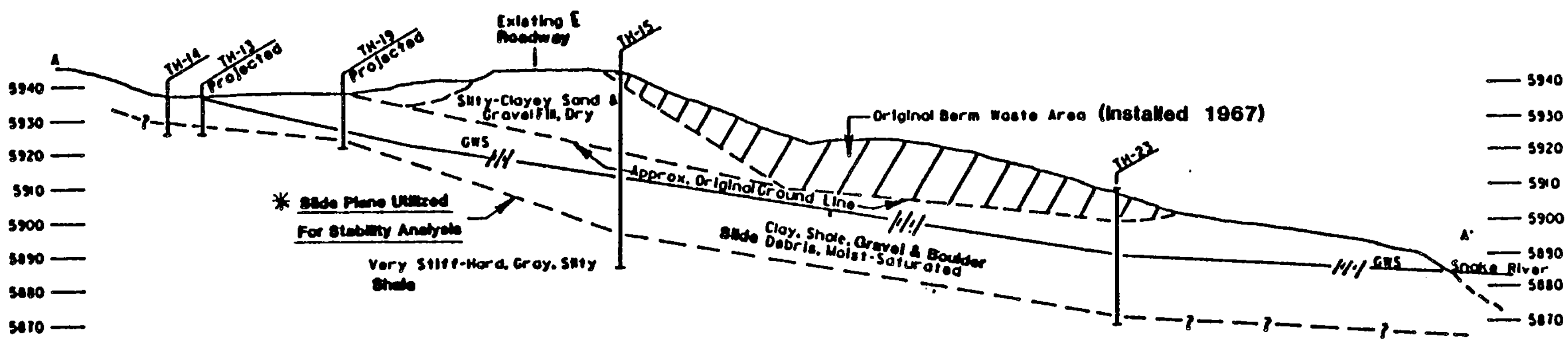


Figure 5. Cross Section A-A' Through Center Of Slide Area

* (Used For The Slope Stability Analysis)

(Vertical And Horizontal Scale Are The Same)

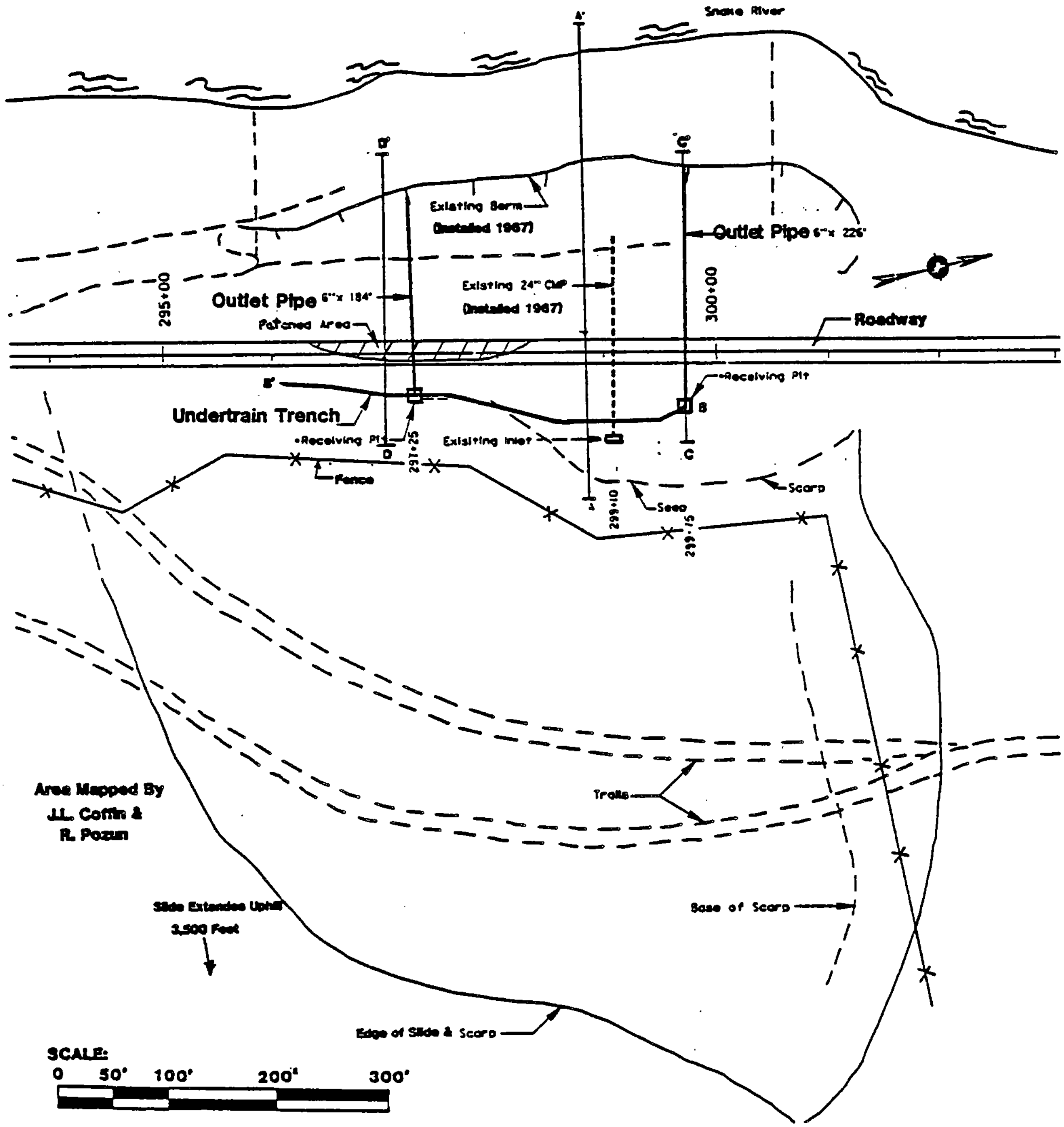


Figure 6. Plan View Of Underdrain Trench And Outlet Pipes

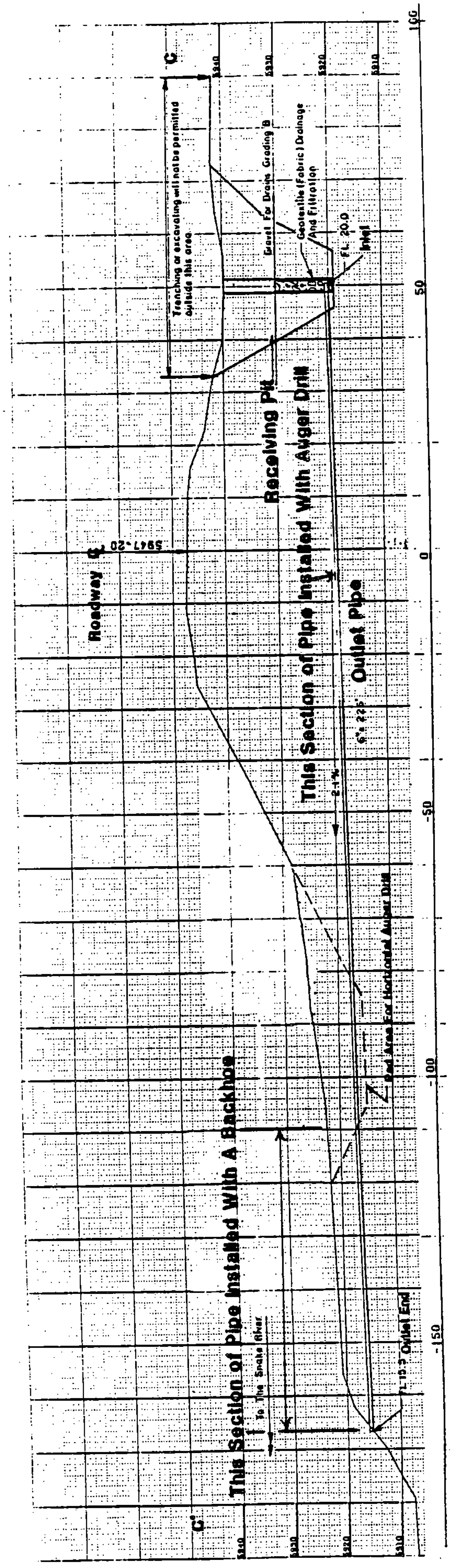


Figure 7. Cross Section C–C', Outlet Pipe And Receiving Pit

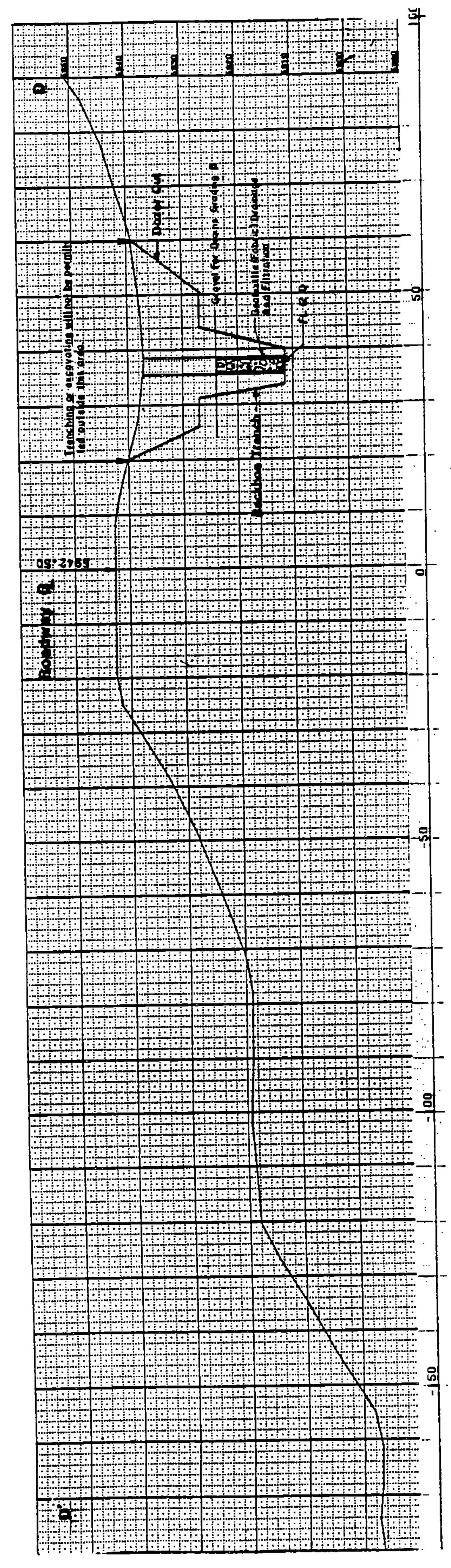


Figure 8. Cross Section D–D', Dozer Cut And Backhoe Trench

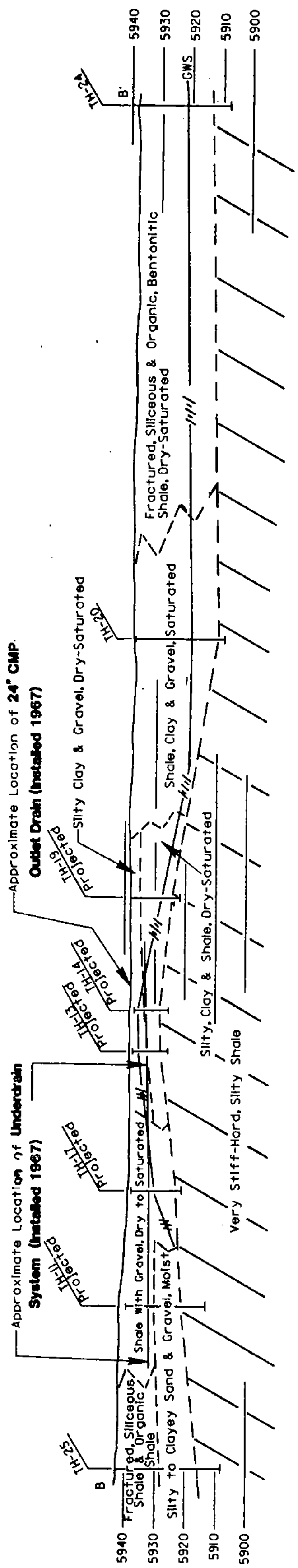


Figure 9. Cross Section B-B', Showing Depth To Bedrock Along The Proposed Underdrain Trench

(Vertical And Horizontal Scale Are The Same)

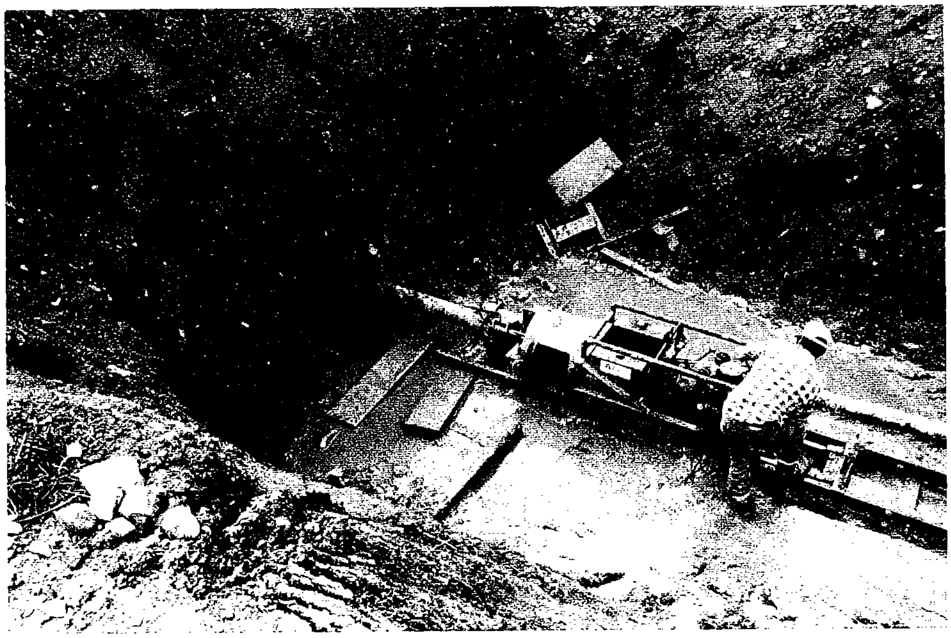


Figure 10. Drilling For The Horizontal Outlet Pipes

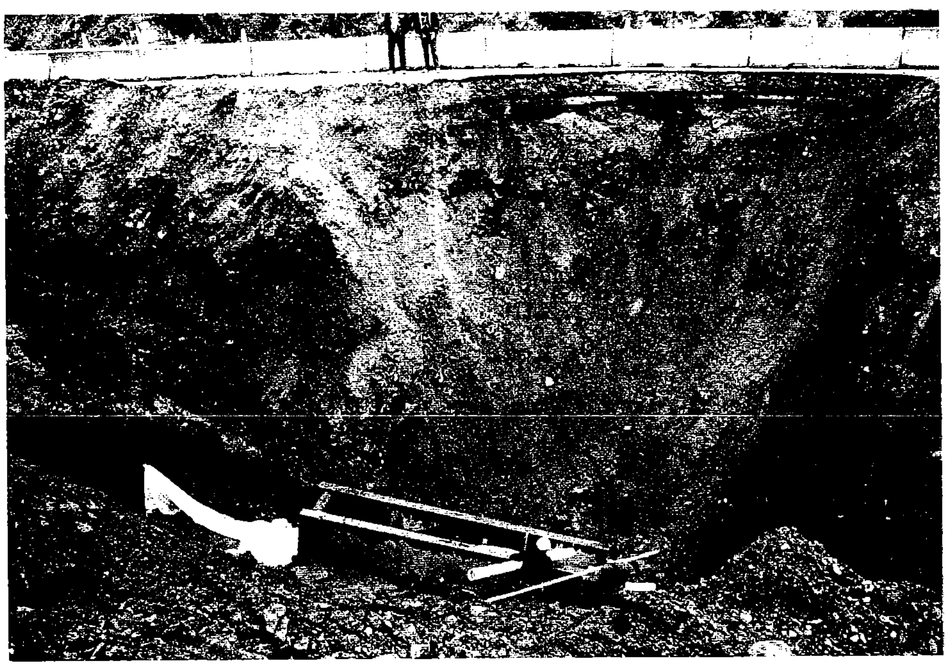


Figure 11. Caving Of The Roadway At The South Receiving Pit

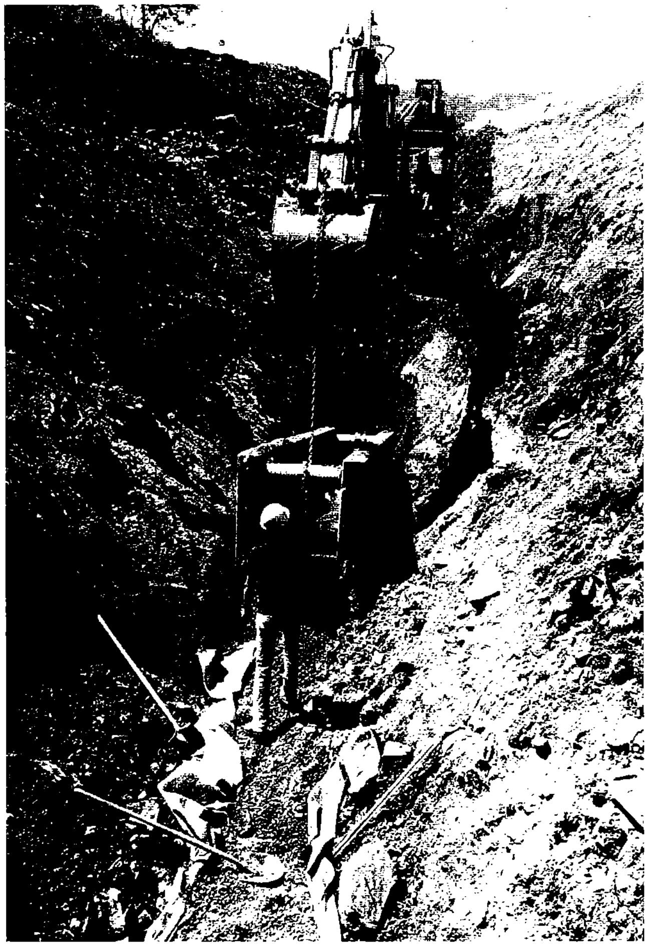


Figure 12. Raising The Steel Box To Add Another Lift Of Gravel And Fabric

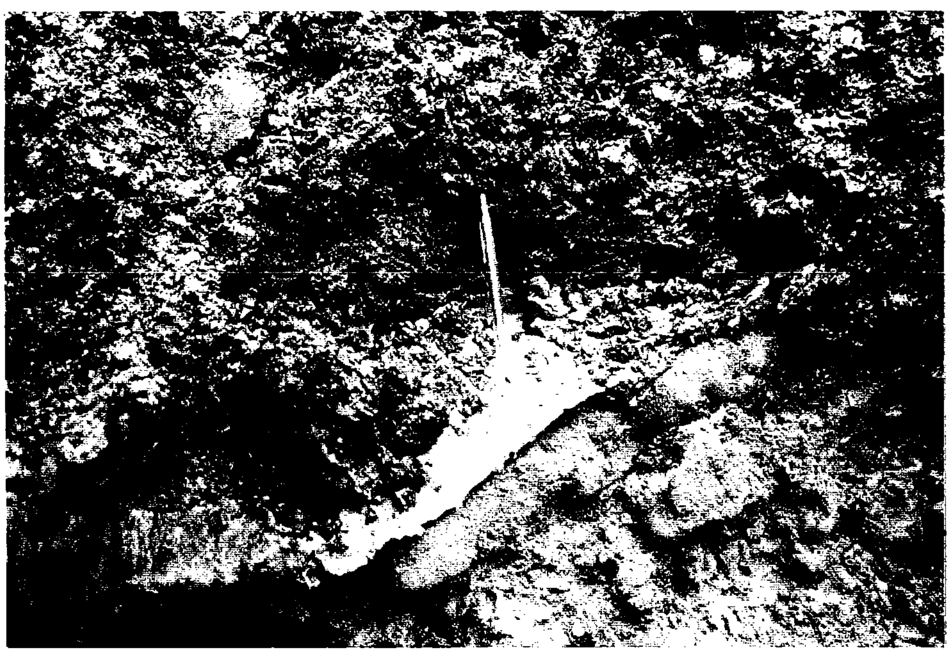


Figure 13. Bentonite Seams And Seepage Present Within The Backhoe Trench

Talus Slope Stabilization Using
Tie-Back Anchors in Provo Canyon, Utah

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ABSTRACT

The Utah Department of Transportation is widening U.S. Highway 189 from Murdock to Upper Falls in Provo Canyon. Due to the canyon's narrowness, scenic beauty, and other factors, developing the road with minimal impact was a major task. Between Stations 380+00 and 390+00 the rock cut for the new highway is up to 130 feet high and intersects an active talus slide extending approximately 500 feet up the valley side. Due to difficult access and environmental concerns, a detailed geotechnical investigation was not possible before construction. On the basis of surface geologic mapping a model was developed to estimate the depth, geometry, and strength of the talus slide. Several options for stabilizing the talus were studied before deciding upon using permanent anchors in conjunction with a cast-in-place concrete wall. The tie-backs were chosen for their amount of design flexibility which would allow for changes in subsurface conditions from those predicted. Installation of the anchors through the talus required the use of special drilling techniques. Anchors were fabricated on site and were essentially double-corrosion protected. All anchors were tested to meet performance specifications.

INTRODUCTION

The Utah Department of Transportation (UDOT) is widening U.S. Highway 189 in Provo Canyon. Because of the scenic beauty of the area, there was considerable public concern about the effect of the construction on the canyon. Between stations 380+00 and 390+00, the new alignment passes through a steep sided portion of the canyon. In order to minimize the height of the rock cuts the faces of the excavations were designed to be 0.25V:1H slopes, with the maximum cut being approximately 130 feet high. The cut intersects one long active talus slope and two shorter talus slopes as shown in Figure 1.

Because of the steepness of the canyon sides and environmental constraints, it was not possible to investigate the depth of the talus or the nature of the underlying rock prior to the start of construction. This paper describes the options examined for stabilizing the talus, the design of the tie-back wall for the largest talus run, and the installation of the anchors. The wall is now complete and the rock cut below the wall is currently being excavated. The design aspects of the rock cut have not been included in this paper.

GEOLOGY

The rocks in the project area form an interbedded sequence of limestones, sandy limestones, limey sandstones, and quartzitic sandstones and are part of the Pennsylvanian age Oquirrh Formation. By and large, the rocks are hard and generally siliceous.

Bedding attitudes throughout the project area are reasonably consistent with a NW-SE strike and a dip on the order of 30 degrees (SW). No direct evidence of faulting was noted within the area of the proposed rock cuts between Stations 380+00 and 390+00, but it is likely that the valleys that bound the area have formed along the trace of significant faults. This is evidenced by a significant change in bedding attitude across the southwestern bounding fault and a less apparent change across the northeastern bounding fault.

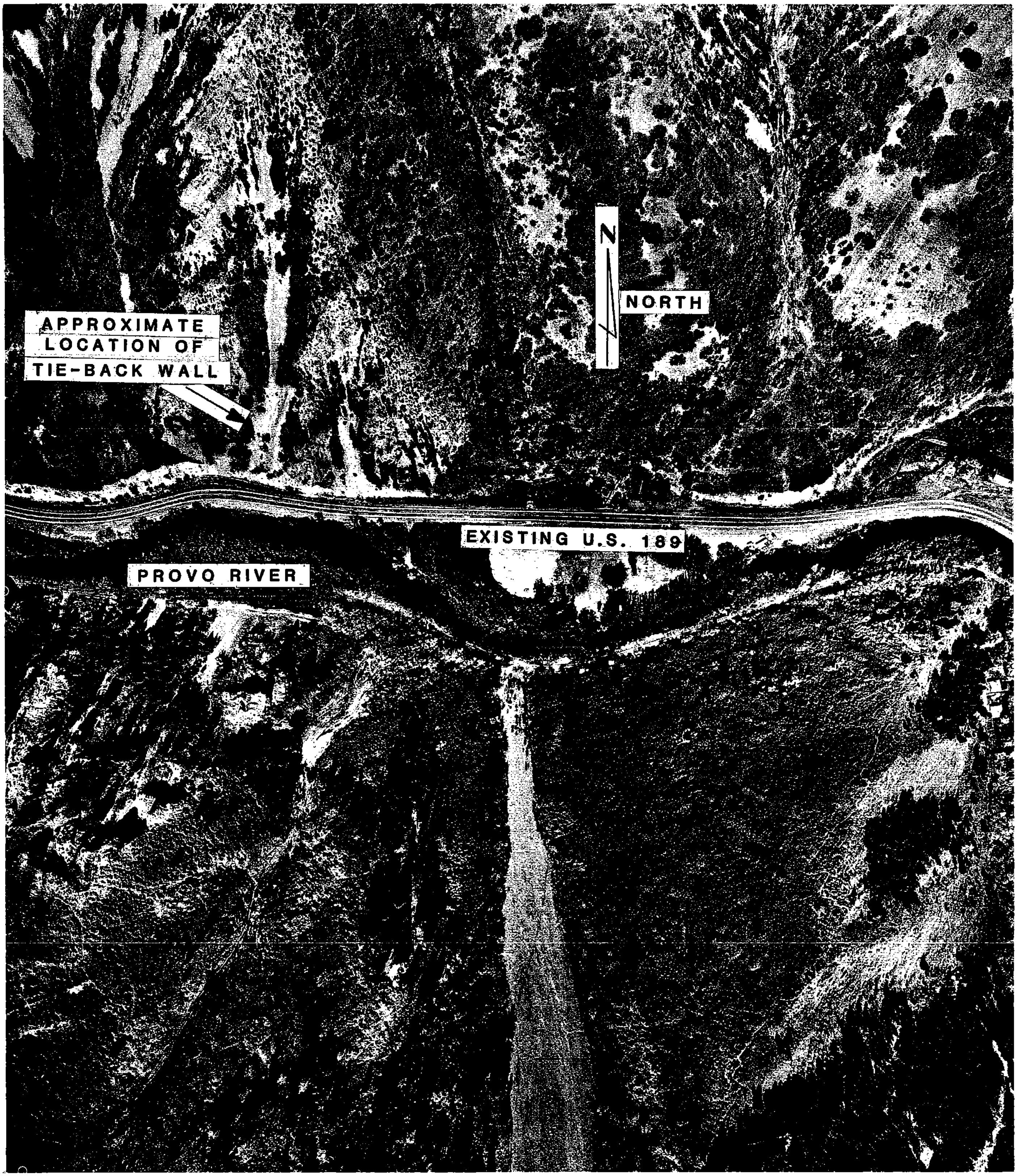
The talus is tabular and is generally less than 6 inches in maximum dimension. The talus is generated from outcrops of fractured rock which are isolated on the hillside as shown in Figure 1. The fracturing may be due to freeze-thaw action and/or shattering of the rock units during tectonic processes.

There are three significant accumulations of talus. Two of them appear to be active as evidenced by the lack of vegetation. The upper foot or so of these areas consists almost entirely of tabular material. Below this there is a varied percentage of fine-grained material contained as a matrix between the tabular shaped blocks.

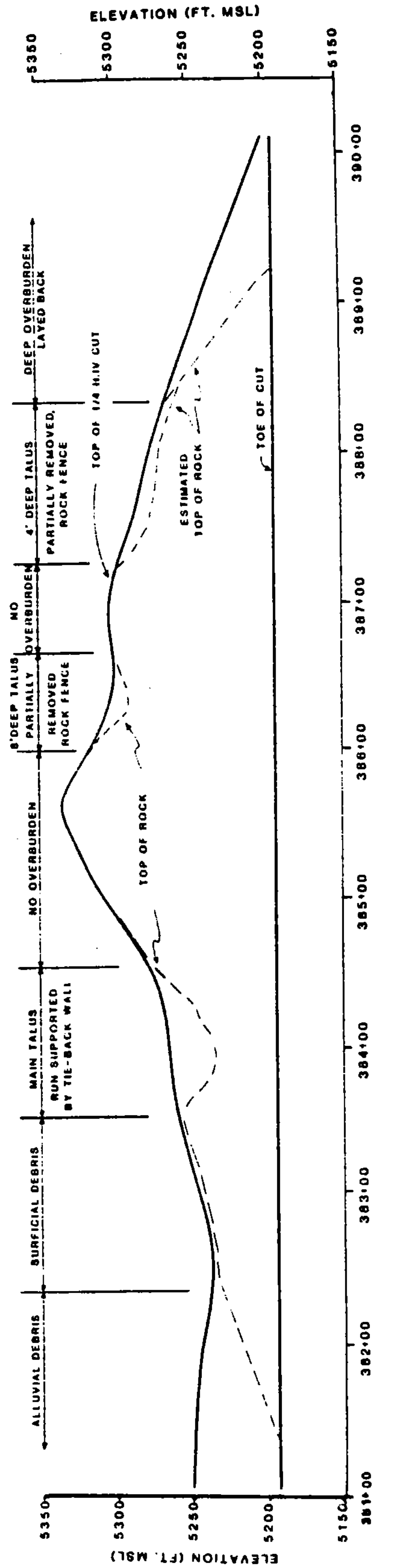
OPTIONS FOR TALUS STABILIZATION

No drilling was done in the talus prior to the start of excavation so the depth of the talus had to be estimated by projecting the bedding and joint surfaces into the hillside from the top of the exposed outcrop ridges on the sides of the talus slopes. The wedges formed by the intersection of these planes were assumed to be filled with talus. The maximum depth of the largest talus run was estimated using this method was to be 30 feet.

As shown on Figure 2 the three talus accumulations intersect the planned 0.25V:1H cut at Stations 384+15, 386+50, and 387+80. The talus run at Station 384+15 extends approximately 500 feet above the new cut and has a volume of about 30,000 cubic yards. The volumes, lengths, and depths of talus at Stations 386+50 and 387+80 are considerably smaller.



PROJECT LOCATION
FIGURE 1



PROFILE OF ROCK CUT WITH TALUS SLOPES

FIGURE 2

Several methods of stabilizing or retaining the talus were studied to arrive at a cost effective, environmentally acceptable, constructable, and stable solution. Some of the options studied included the following:

A. Drilling through the talus, then installing permanent tie-back anchors into the rock, and attaching the anchors to a reinforced concrete retaining wall.

B. Same as above, but instead of a reinforced concrete wall, the cut face would be covered with mesh and rebar and have shotcrete applied. This option was discarded due to aesthetics and because it would be difficult to cut the face uniformly enough to avoid a ragged appearance.

C. Installing soldier piles to form a cantilever retaining wall. Due to the excessively large size of piles required for talus depths greater than 10 feet and the difficulty in getting access for equipment large enough to drill large diameter holes, this option was ruled out as uneconomical.

D. Fully or partially removing the talus runs.

E. Suspending draped wire mesh above and over the rock cut to dissipate the energy of the blocks of talus that break loose from the upper slope.

Method A was selected for the largest volume of talus at Station 384+15, as shown in Figures 3 and 4, and a combination of Method D and Method E was selected for the two smaller volumes of talus, as shown in Figure 5.

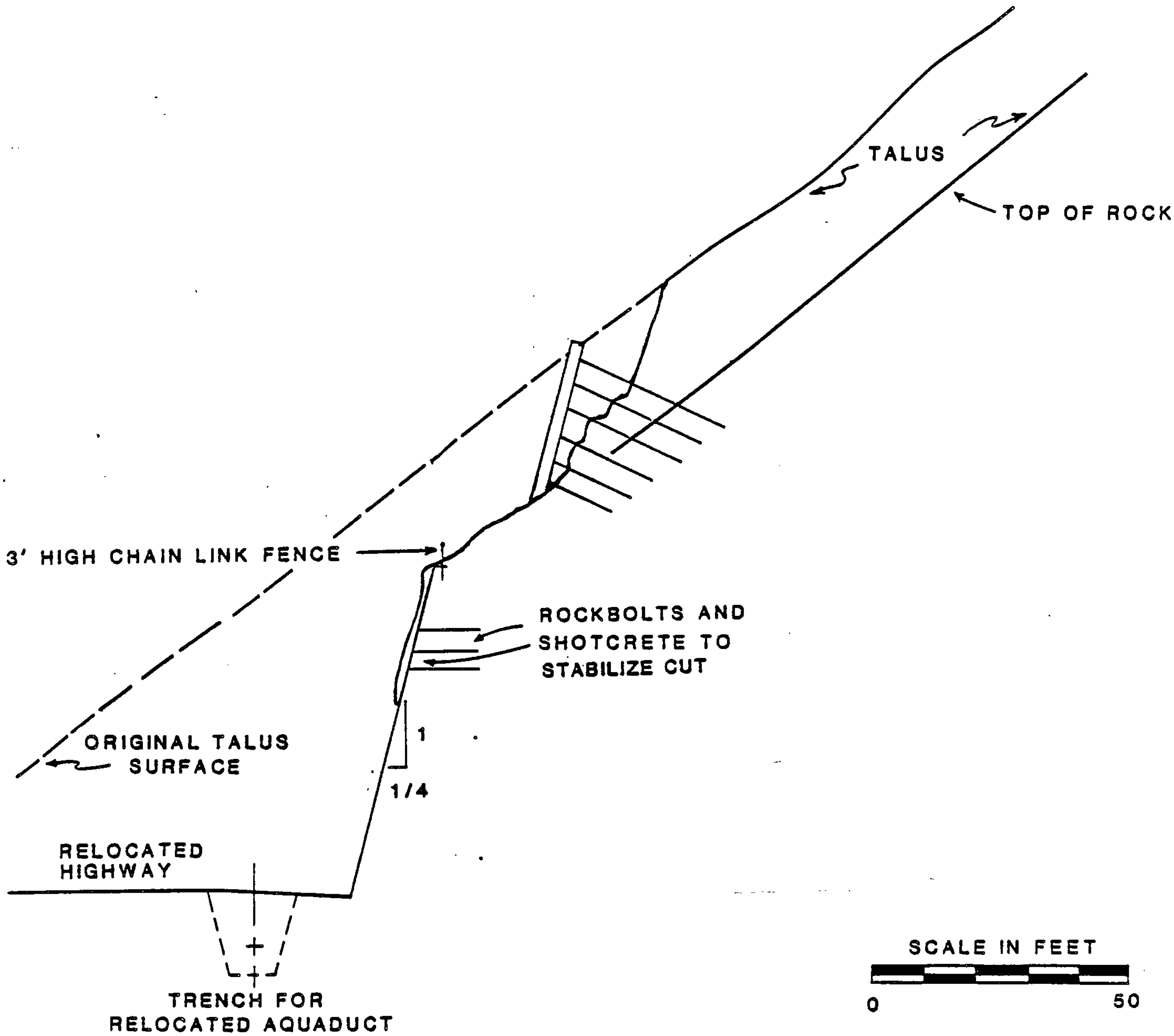
To study the effectiveness of the ditch width and geometry of the cut and wall, a series of rock bounce analyses were performed using a computer program developed by Hoek (1987), as shown in Figure 6.

TIE-BACK WALL DESIGN

The design used for the tie-back wall for the talus at Station 384+15 has two unusual aspects:

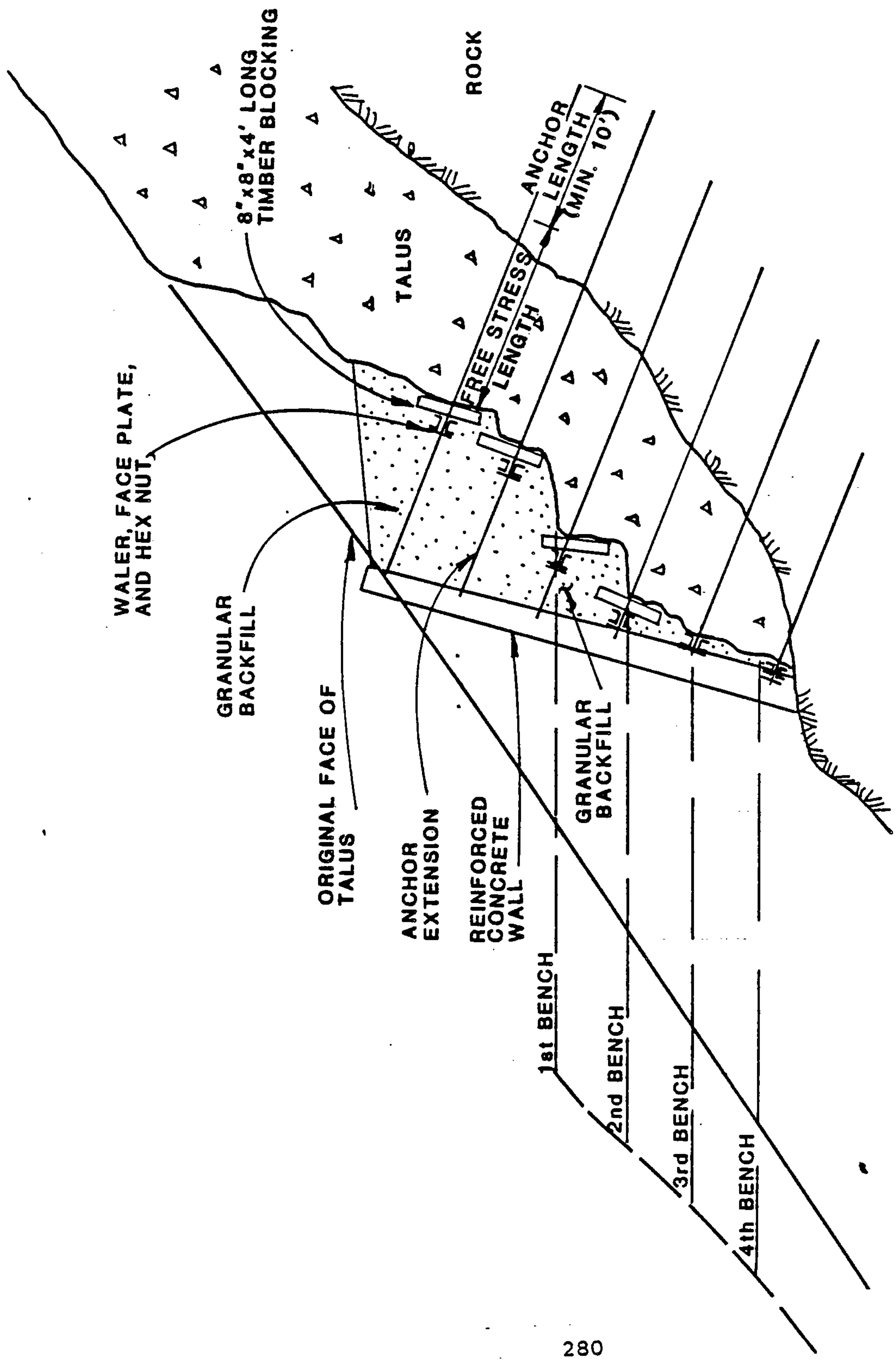
(1) - the capacity and spacing of the tie-backs had to be suitable for any depth of talus up to a maximum of 30 feet;

(2) - it was necessary to support the talus as the excavation proceeded because the talus would have become unstable if the full depth were excavated prior to constructing the wall.



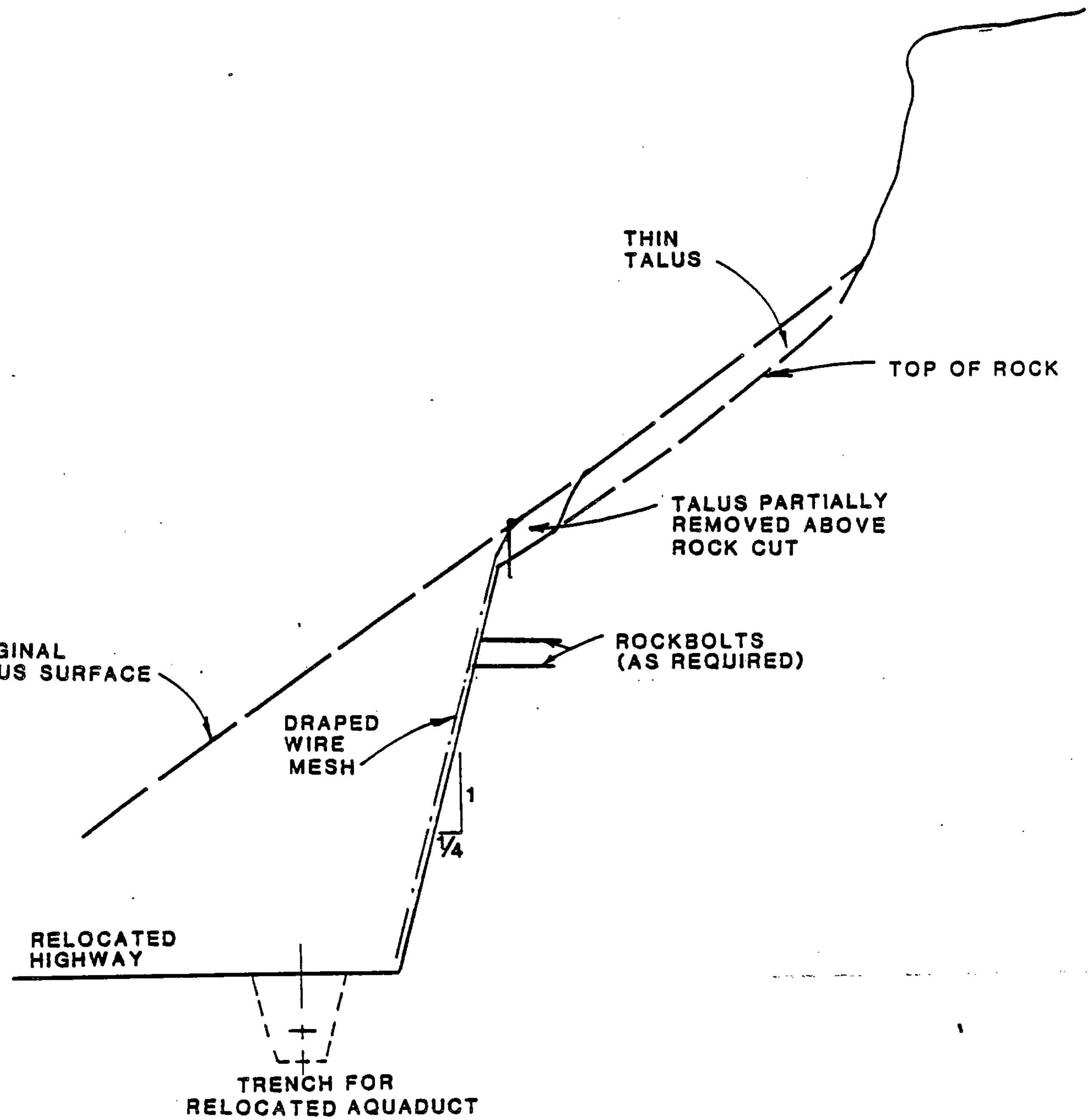
TYPICAL SECTION THROUGH TIE BACK WALL

FIGURE 3

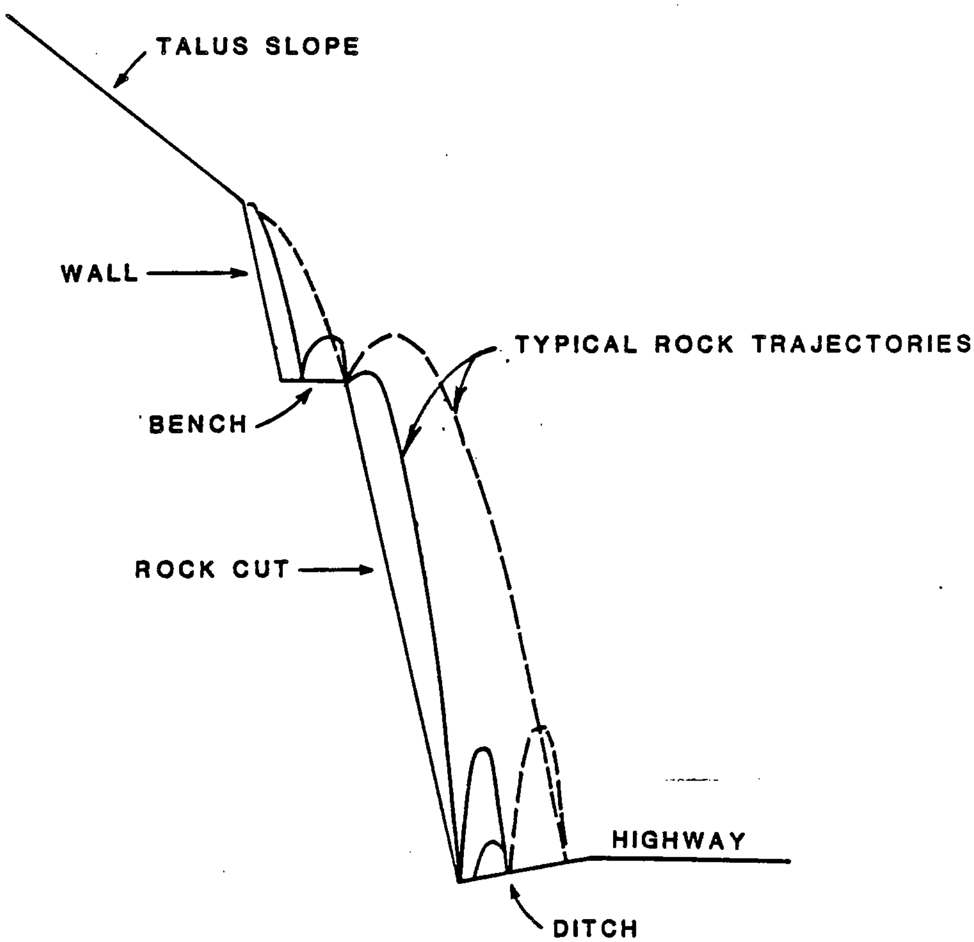


TYPICAL SECTION

FIGURE 4



THROUGH ROCK CUT
WITH SMALL VOLUMES OF TALUS
FIGURE 5



TYPICAL ROCK BOUNCE ANALYSES

FIGURE 6

The slope of the talus surface represents the angle of internal friction of the talus. Based on topographic maps, this angle varies between 36 and 39 degrees. The inclination of the underlying rock surface was unknown, but surface outcrops suggest that it is slightly steeper than the surface slope of the talus.

To evaluate the support required, several modes of failure were examined including sliding wedge and more complex limit equilibrium analyses (Sarma, 1974).

The following assumptions were used in the analyses;

- the angle of internal friction of the talus is 37 degrees;
- the factor of safety of the talus prior to excavation is 1.0;
- the support chosen will give an overall minimum factor of safety of 1.5 for the wall and a length of the driving mass upslope from the highway cut;
- the upslope extent of talus requiring support was limited to 120 feet, as the talus necks down considerably at this point; and
- a seismic coefficient of 0.15g was used in design (Algermissen, 1970).

For 30 feet deep talus the support pressures arrived at were approximately 3.23 kips/sq. ft. Parametric studies indicated that increasing the slope angle of the underlying rock surface, increasing the effective upslope extent of the talus, and decreasing the depth of the talus had little effect on the required resisting force per square foot of wall surface.

Using Terzaghi earth-pressure diagrams for cohesionless materials in braced excavations the stresses on the anchors and the face of the wall are approximately equal, top to bottom. This allowed anchor spacing and capacities to be uniform. Anchors were designed to be on 5 foot centers (vertical and horizontal) and to resist a total design load of 81 kips. Anchors selected were 1.25 inch diameter, 150 ksi., continuously threaded Dywidag bar.

CONSTRUCTION

Construction of the tie-back wall began in August 1987 and continued through October. Each step of the anchor installation procedure consisted of the following.

- Excavating 3 to 5 feet high benches.

- Locating anchor positions.

- Drilling and anchor installation.

- Grouting.

- Proof testing.

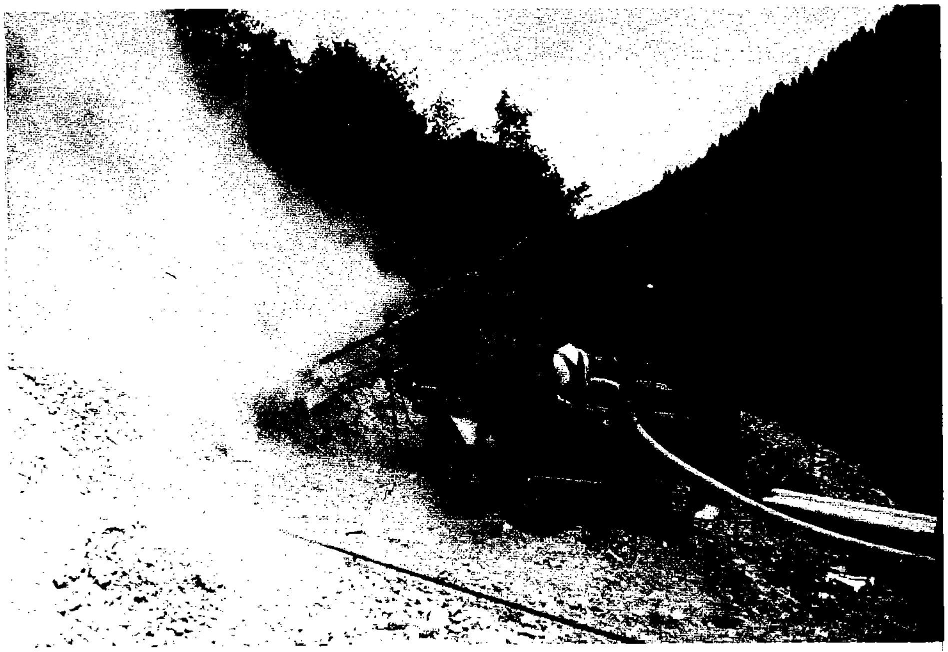
Excavation of the benches was performed with either a large backhoe or a bulldozer. Traffic below the cut was stopped during excavation since occasionally pieces of talus would roll down the slope and across the road.

Drilling was performed by Frontier Foundations of Salt Lake City. The drill rig used was an Acker air-track with the Odex drilling system (Figure 7). The Odex system uses an eccentric percussion bit which advances the 4.5 inch I.D. steel casing behind it through the talus and a minimum of 3 feet into rock. Once the casing is set into rock, the Odex bit was withdrawn and the rock (anchor zone) was drilled into a minimum of 10 feet deep with a standard 4.25 inch diameter carbide button bit. Before the steel casing was withdrawn a section of 4 inch diameter PVC pipe equal in length to the depth of the talus was inserted to insure that the hole would remain open through the talus.

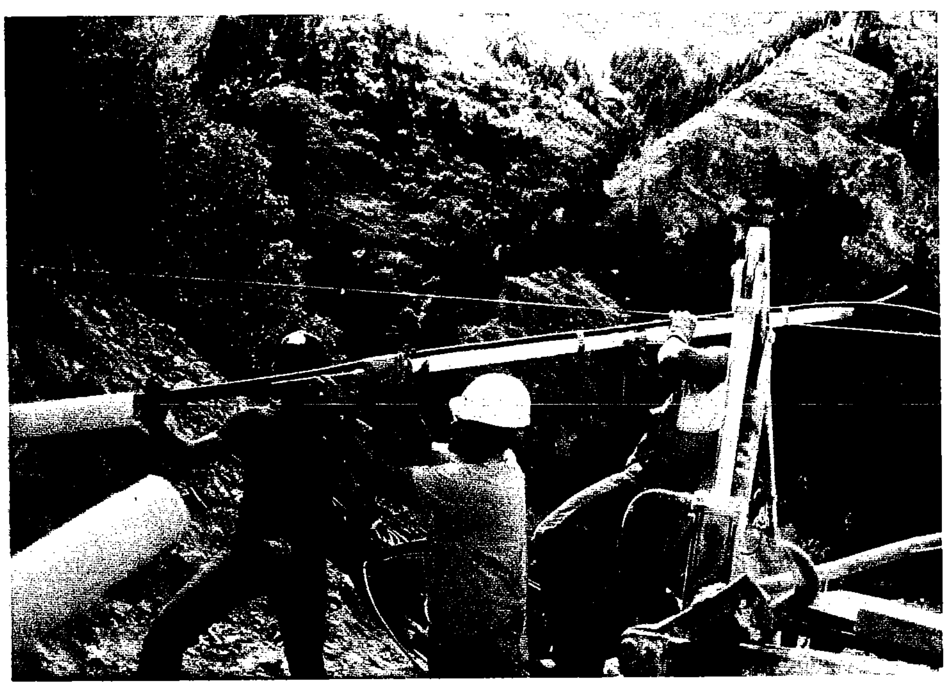
Anchors were fabricated on site and consisted of the Dywidag bar, centralizers, 2 inch diameter PVC in the free stress length, a grout tube extending to the end of the anchor, and a grease tube extending to the end of the free stress length (Figure 8). After the anchors were placed in the hole the anchor zone and the annular space between the 2-inch and 4-inch diameter PVC pipes was grouted in a single-stage process. Grout was a mix of water and Portland cement in a 0.67 : 1 ratio. Varying opinions on the use of Sika Interplast N as a fluidizer and admixture caused it's use to be discontinued after the first couple of anchors. These concerns were mainly about reduction of corrosion protection due to the entrained air. It was found that even without using the Interplast the grout was reaching it's required strength (3500 psi) in only 3 days.

Usually, the day after grouting, grease was pumped into the free-stress zone to add corrosion protection. This system effectively gave double corrosion protection in the most critical zone - through the talus.

Each anchor was proof-tested to 130% of it's design capacity (100 kips). In addition, several anchors were performance tested to a maximum load of 150 kips at which point the load was locked off and the anchor monitored for creep (Figure 9). After testing, the anchors were typically untensioned to a point where they would



DRILLING
FIGURE 7



TIE-BACK ANCHOR INSTALLATION
FIGURE 8

TIE-BACK ANCHOR PROOF TESTNG
FIGURE 9

just hold the wale and the timber blocking in place. Ravelling between timber blocking was never a severe problem, making lagging between wales unnecessary.

CONCLUDING REMARKS

The tie-back wall proved to be a satisfactory method of supporting a large talus run in steep terrain with difficult access. The behavior of the talus was much as expected and assumed during the design phase. In all, a total of 72 tie-back anchors were installed. The actual final height of the wall (29.3 feet) was very close to that predicted by the surface geologic mapping. If the ravelling of the material had been worse it is likely that it could have been handled using this same method, without a significant loss in production. The Odex drilling system and the anchors used performed very well, and the flexibility of the design satisfactorily allowed for the unknown irregularities in the subsurface conditions.

ACKNOWLEDGEMENTS

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CONSIDERATIONS AFFECTING THE CHOICE OF NAILED SLOPES AS A MEANS OF SOIL STABILIZATION

BY

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ABSTRACT

Although nailed slopes have been used in many instances for the short term stabilization of temporary excavations for basements in urban areas, their use for the long term stabilization of highway slopes is just beginning in the United States.

The authors have been involved in the recent study of permanent nailed slopes as an alternative to more conventional slope stabilization methods, for a new highway in mountainous terrain near Mount St. Helens, Washington. The severe climatic conditions at the site, as well as the close assessment of environmental considerations such as aspect, set new design considerations for the engineers and geologists.

The project required a site investigation adapted to the difficult terrain conditions, followed by an analysis to determine the most cost effective means of stabilizing cut and fill slopes. The study identified soil nailing as a recommended option for specific sites, after careful consideration of the geological conditions and constructability.

The paper describes the factors affecting the choice of soil nailing, including limitations set by the soil and rock conditions, the relative merits over other retaining systems, the environmental considerations, and the contractual format for installing the nailed walls. The paper provides useful information for geologists who may be considering the use of soil nailing, as the published data is limited for permanent nailed slopes.

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[2]Washington State Department of Transportation

INTRODUCTION

This paper presents a brief description of soil nailing and its potential applications in supporting both temporary and permanent cuts on highway projects. The paper is neither a state of the art review of the topic nor a detailed discussion of specific technical issues. Rather the purpose of the paper is to provide general background information to assist the practicing highway engineer in evaluating when soil nailing might be a viable alternative to conventional support systems. Soil nailing is a relatively new technique with a limited performance record, particularly as related to permanent support. Thus there are numerous technical issues relating to both design and construction standards that are still being developed. Research funded by FWHA is expected to provide the basis for future soil nailing standards for highway projects in the U.S.

Soil nailing has been shown be a cost effective and time saving solution to supporting both temporary and permanent excavations in soil. Several key factors need to be assessed in reaching a decision to use soil nailing as either the primary design or as an alternative. These factors are discussed below. As an example, the decision process leading to the choice by the Washington State Department of Transportation (WSDOT) to use soil nailing to support a permanent excavation for the new Spirit Lake Highway leading to Mt. St. Helens is discussed. This project represents an excellent example where soil nailing has several clear advantages over conventional support methods.

DESCRIPTION OF SOIL NAILING

Soil nailing is a method of slope stabilization whereby excavated vertical or near vertical slopes are strengthened by low pressure grouting in place of small diameter steel rods and placement of a shotcrete facing. The origins of soil nailing are linked to the development of the New Austrian Tunneling Method (NATM), rock bolting, and Reinforced Earth walls. The first nailed slope is generally considered to be the use of 12 foot long steel bars to reinforce a railroad cut near Versailles, France in 1972. The first nailed slope in the U.S. was for the Good Samaritan Hospital in Portland in 1976. Shen in the U.S., Gassler and Gudehus in Germany, and Schlosser in France published some of the early technical articles on the principles of soil nailing.

Nailed slopes have become common practice for supporting temporary excavations in both France and Germany. Nailed slope heights up to about 60 feet are frequently used in competent soils. The method is beginning to find acceptance in the U.S. particularly for the temporary shoring of building excavations in competent soils. Although less common than temporary support, nailed slopes have also been used to support permanent cuts. Permanent nailed slopes have been used in the French Alps since 1985. In 1985, a nailed slope was used to support cuts up to about 40 feet high in colluvium and weathered rock for tunnel portal construction at Cumberland Gap Kentucky. Several permanent nailed slopes have been designed at the same project and will be under construction this year.

The general procedure for soil nailing involves the following sequence as depicted on Figure 1:

- Excavate an unsupported cut approximately 4 to 6 feet high;

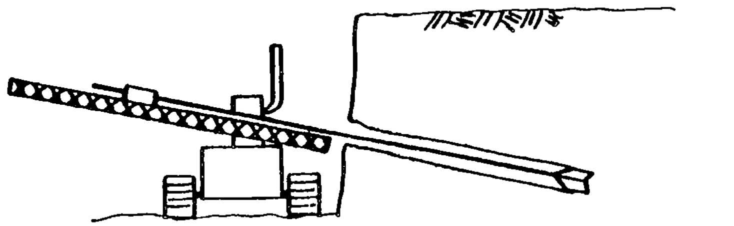
- Install nails inclined at about 15 degrees to the horizontal at approximately a 6 foot spacing in staggered rows such that each successive row is offset by half of the nail spacing as shown on Figure 1; the nails consist of small diameter steel rods (such as Dywidag bars) grouted into drilled holes generally 4 to 8 inches in diameter;

- After placement of a row of nails, cover the exposed soil face with reinforced shotcrete; the contractor is normally required to shotcrete within 24 hours of excavating each lift.

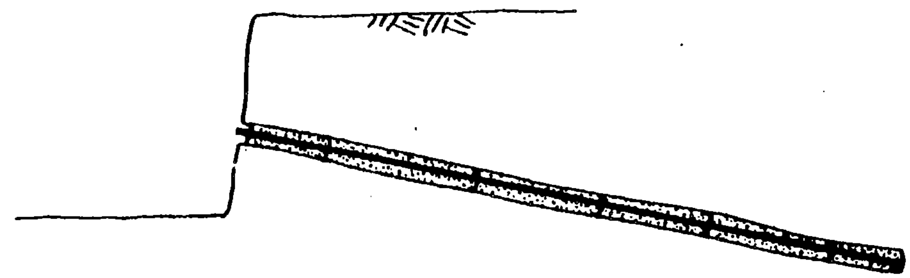
The contractor completes this procedure for successive excavation lifts until the excavation is completed to the required depth. Each excavation lift must be completely nailed and shotcreted before the next lift is excavated. A positive means of drainage is normally provided with either weepholes or fabric drains behind the shotcrete. Figure 2 shows a typical soil nailing project used to support a 55 foot deep excavation in dense glacial soils in Seattle.

A nailed slope does not behave in the same manner as a tieback wall. In a tieback wall, the tiebacks transfer the loads on the wall to the tieback anchor zone behind the so called "no-load" zone. Thus the tieback wall face itself must structurally resist the full design soil pressures. In a nailed slope where the nails are at a closer spacing, the soil and nails work in combination to form a reinforced soil mass similar to a reinforced earth wall. The shotcrete face acts to limit stress release and soil erosion but does not act as normal lagging to support the full earthpressures. In a tieback wall, there is a distinct "no-load" zone and a distinct anchor zone. In a nailed slope, the nails are designed to develop bond throughout their length.

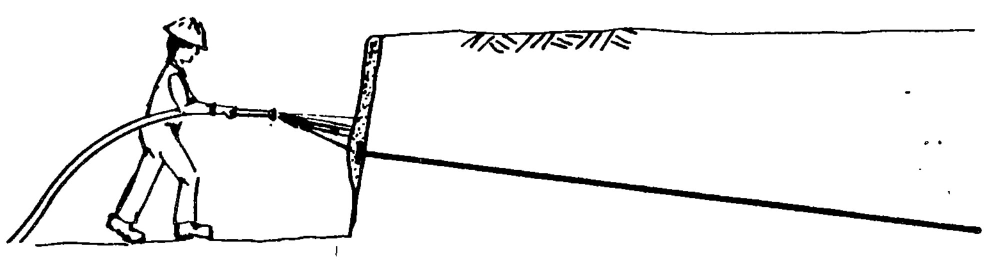
Design methods currently used consider the stability of the most critical soil shear plane behind the completed excavation. In addition, each successive excavation lift is analyzed to assess the stability during excavation. Different methods of calculation are available using various shear plane geometries and boundary assumptions. In the U.S. computer codes based on Shen's work are most commonly used. A typical design for a 40 foot deep temporary excavation in competent soil might involve 30 foot long equal spaced nails on a six foot staggered spacing with a nominal four inch thick shotcrete face with welded wire reinforcement.



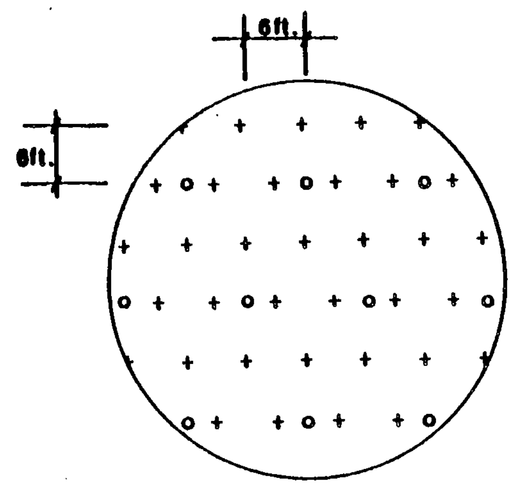
STEP 1 Excavate unsupported face to 6 feet high and drill nail.



STEP 2 Install nail with centralizers in hole and grout.



STEP 3 Shotcrete exposed soil face.



ELEVATION OF TYPICAL NAIL LAYOUT

+ NAILS

o WEEPHOLE

Figure 1. Sequence of Nail Installation

Figure 2. Nailed Slope Shoring Wall

FACTORS EFFECTING FEASIBILITY OF SOIL NAILING

In general, soil nailing is technically feasible in most of the conditions suitable for a tieback wall. Probably the major site condition limiting its use would be granular soils below the groundwater table or soft unstable cohesive soils. Specifically, the main technical factors affecting the feasibility of a nailed slope include:

- STAND-UP-TIME: For nailing to be feasible, the soil must be able to stand unsupported in 6 to 8 foot cuts for the time required to nail and shotcrete. Thus the soil must have some "apparent" cohesion. Most soils will exhibit adequate stand-up-time to allow nailing. Soils which are a problem include loose clean soils which have no "apparent" cohesion, granular soils which rapidly pipe below the water table, and soft cohesive soils which will squeeze into the excavation.

- REASONABLE SOIL BOND: The nail grout needs to develop a reasonable bond strength with the soil to minimize the design length of the nail. For ultimate bond values below about 750 psf, the nails become excessively long. Since the actual adhesion is sensitive to the drilling method used, anchor testing is essential to confirm design assumptions.

- CORROSIVE ENVIRONMENT: For permanent nailed slopes, long term corrosion effects could weaken the nails and the nail connections. Thus a corrosive soil environment could impact the feasibility of permanent soil nailing. Normal practice is to use permanent tieback technology to provide adequate double corrosion protection of the nails. Thus environments suitable for tie-back anchors would be suitable for nailing.

- ABILITY TO DRILL AND INSTALL NAILS: The ground must be suitable for drilling and installing the nails. In general, the same factors that affect the feasibility of tie-back installation will affect nail installation.

- PROXIMITY OF ADJACENT STRUCTURES: Performance of nailed slopes appear to result in similar deformations to soldier pile tie back walls. Thus nailed slopes have been successfully constructed adjacent to major existing structures. However, the extent of the performance record is limited as compared to tie back walls. Thus, the uncertainties in predicting deformations of nailed slopes should be considered if major structures are located near the excavation.

Several non-technical issues also need to be addressed when considering a nailed slope. These include:

- REGIONAL PRACTICES AND EXPERIENCE: Regional practices and experience need to be considered when evaluating the overall advantages/disadvantages of soil nailing. Proper shotcrete application by an experienced contractor is essential to the success of the nailed slope. There is a wide variation in the experience and use of shotcrete throughout the U.S. Limited regional experience might translate to poor construction and increased bid costs. Other regional practices and experience might also impact the choice of soil nailing.

- AESTHETIC CONSIDERATIONS FOR PERMANENT WALLS: The aesthetic considerations for permanent walls might impact costs and affect the acceptability of nailing. Temporary nailed slopes have rough finished shotcrete faces with weep holes and protruding nails. In addition, due to minor variations in the concrete mix, each shotcrete lift invariably has a slightly different shade resulting in a stratified appearance. Although increasing the costs, there is a wide range of techniques available for improving the aesthetics of a permanent facing including covering the protrusions, eliminating the weep holes, and coloring and texturing the shotcrete. However, depending on the overall architectural design concept, an exposed shotcrete finish may be unacceptable. As an alternative, the shotcrete could be covered with a variety of materials including conventional facia walls such as used to cover permanent tieback soldier pile and slurry walls. The main difficulty in the use of pre-cast panels is the low tolerance normally achieved in the nail locations, which makes attaching the panels difficult.

ADVANTAGES AND DISADVANTAGES OF SOIL NAILING

In general, soil nailing would be an alternative to a cantilever or tieback soldier pile wall. Normally these types of walls are used where either geometric or adjacent property constraints do not permit unsupported excavation of cuts. Probably the main advantage of soil nailing is cost. Although the total amount of steel required for the nails is normally similar or even greater than required for the tiebacks in a equivalent tieback wall, the nailed slope has no soldier piles. Based on experience with temporary shoring, a nailed slope can be expected to be 15 to 30 percent cheaper than a conventional soldier pile tieback wall. In general, the more competent the soil, the greater the probable savings. Another advantage is the potential for reducing construction time since nailed slopes can be constructed more quickly than soldier pile walls. In addition, any delays due to the delivery time on soldier pile steel are eliminated.

In addition to cost and schedule, there are certain conditions that are particularly well suited to nailed slope construction. Conditions such as soil over rock can often be difficult for soldier pile construction whereas these conditions are well suited for nailing. Installation of soldier piles in soils with a significant amount of cobbles and boulders can be very difficult, both practically and contractually. However, with a nailed slope there are no soldier piles and problems are relatively simple to resolve.

However, as discussed above, soil nailing cannot be used in a variety of conditions including clean sands below the groundwater table, soft clays, and other types of unusually loose/weak soils. Where feasible, the main disadvantage of nailing probably relates to the fact that it is a relatively new and evolving method. This might translate into higher design costs, higher bid costs, and uncertainties in workmanship and performance. In considering soil nailing, particularly for permanent support, the highway engineer should take this into account. Specific design and construction issues that are still evolving include the method of analyzing the nail loads and lengths, the loads on the shotcrete facing, the distribution of nail loads, and production nail testing requirements.

A further disadvantage of nailed slopes is related to the production testing of the nails. Normally production testing of tiebacks involves stressing all of the anchors to about 130% of the design load and then locking them off at some percentage of the design load. This procedure combined with the a limited number of pull out tests provides a high degree of confidence in the anchor capacity. Testing of tiebacks is facilitated by the no-load zone which allows the actual anchor zone to be readily stressed. Nails, on the other hand, are grouted for their full length and do not have a no-load zone. Thus to test the anchor capacity in the zone beyond the theoretical failure plane requires special nails to be installed and tested with an unbonded zone. This can be done by installing sacrificial nails or by installing selected production nails with an unbonded zone and grouting this zone after testing. The net result is that only a relatively small percentage of the production nails are tested.

EXAMPLE OF PROPOSED USE OF A PERMANENT NAILED SLOPE

BACKGROUND

On May 18, 1980, Mt. St. Helens, in southwestern Washington, erupted with a force equivalent to several hundred atom bombs destroying 60,000 acres of forest and killing 61 people. In addition, over 20 miles of State highway SR 504 leading to the mountain along the valley of the North Fork of the Toutle River was destroyed, buried under tons of debris and ash.

The proposed replacement highway for SR 504 will consist of about 25 miles of a new two-lane 32 foot wide roadway. The alignment will be located on the sides of the valley well above any future debris flows emanating from the Mountain. Figure 3 shows the general setting of the proposed alignment. Being a hillside alignment in mountainous terrain, the highway will involve major cuts and fills. In several areas due to steep existing soil slopes up to about 35 degrees, open cuts will not be feasible and retaining walls will be required. During the preliminary design phase, soil nailing was identified as a potential alternative for supporting required permanent soil cuts between Station 1439+00 and 1442+50. Figure 4 shows the area of the proposed walls.

Figure 3. Mount St. Helens Area

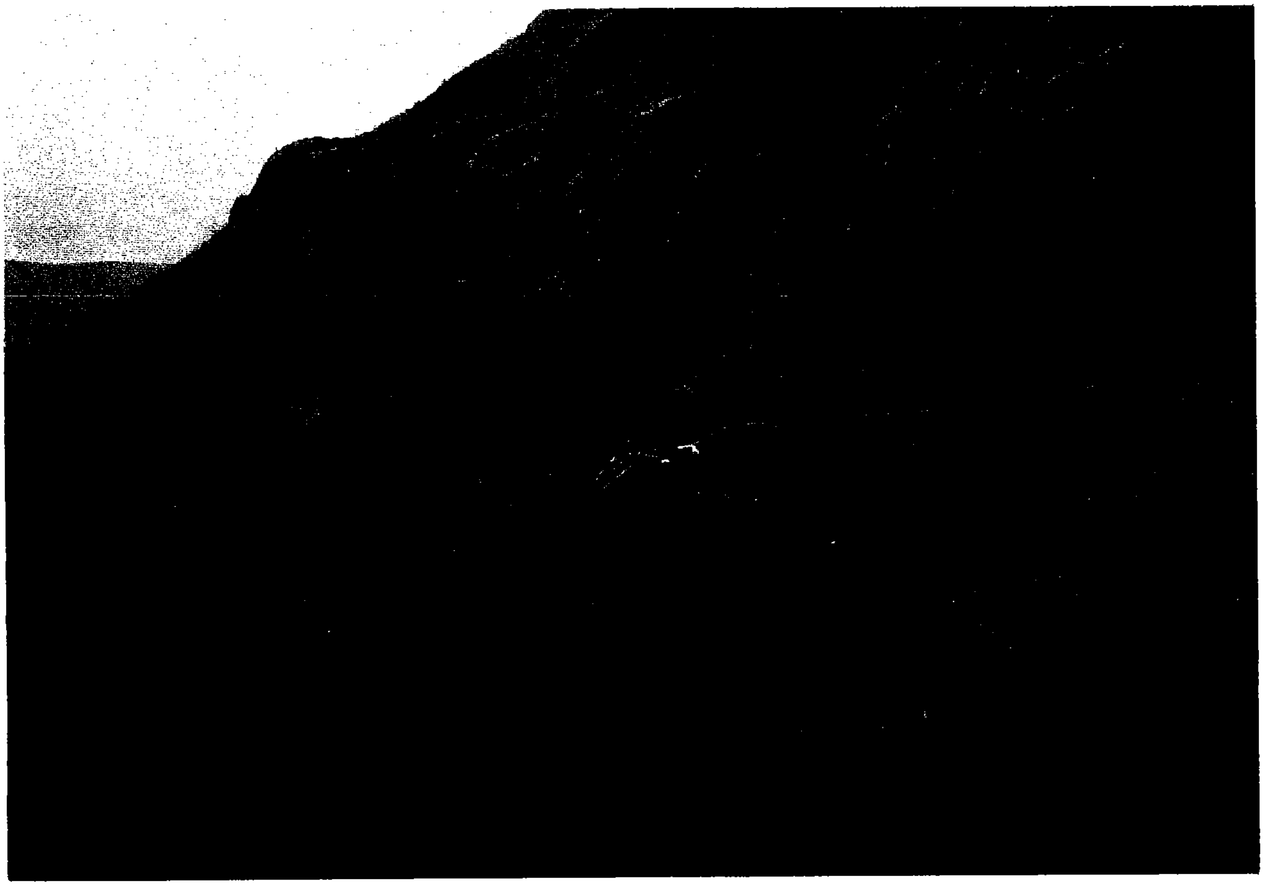


Figure 4. Alignment Area (wall near left side below cliffs)

ALTERNATIVES

The required cuts between Station 1439+00 and 1442+50 are on the order of 65 feet in an area where the natural slopes range up to about 35 degrees. The geologic conditions consist of compact colluvium over volcanic rock. The depth to rock is quite variable ranging from less than 20 feet to over 35 feet. Thus the required excavation will involve both soil and rock cuts. The colluvium is composed of a compact to dense silty sand and gravel with occasional cobbles and boulders and zones of clean open gravel size talus material just above the top of rock. The rock consists primarily of poor to good quality basalt. The soil-rock contact appears to roughly parallel the ground surface topography. The groundwater table is believed to be below the depth of cut although zones of seepage may occur in both the soil and the rock. Figure 5 shows the subsurface conditions as exposed in a 20 foot high vertical test cut.

Due to the steep existing slopes, it was not considered feasible to use unsupported permanent soil slopes or even to construct unsupported temporary cuts for construction of conventional retaining walls. Normal WSDOT design practice would have been to select a permanent tie-back soldier pile wall to support the soil, construct a bench, and then form an unsupported rock cut. Based on the subsurface conditions, soil nailing was judged to be technically feasible and was evaluated as a potential alternative to a tie back wall. Conditions suitable for soil nailing included a relatively deep groundwater table combined with compact to dense silty soils which were judged to have excellent stand-up properties. Figure 6 and 7 show schematics of the two alternative solutions.

A cost estimate was completed for the alternatives with a permanent nailed slope estimated to be about 20 percent less costly than the soldier pile and tie back alternative. In addition, the soldier pile solution was judged to have a greater potential for construction problems. This was based on several factors. The soldier pile design required that an assumption be made as to the top of competent rock in order to select the bench elevation and pile lengths. Due to the variability in the rock, there would be a risk that the top of rock would be deeper than assumed unless a very conservative assumption was made. A conservative assumption on the top of rock might result in excessive drilling into rock. Due to the occurrence of cobbles and boulders in the colluvium and zones of poor quality rock, drilling of the soldier piles might be very costly and involve extra expenses if normal equipment could not advance the hole. WSDOT had experienced problems in installing soldier piles in similar ground conditions on previous projects. The soil nailing procedures are well adopted to mixed ground conditions and variations in the top of rock elevation.

SOIL NAILING SELECTION AND DESIGN

Based on cost and considerations of potential construction problems with soldier pile installation, soil nailing was selected to support the permanent cut slope from 1439+00 to 1442+50. The design of the wall is currently being

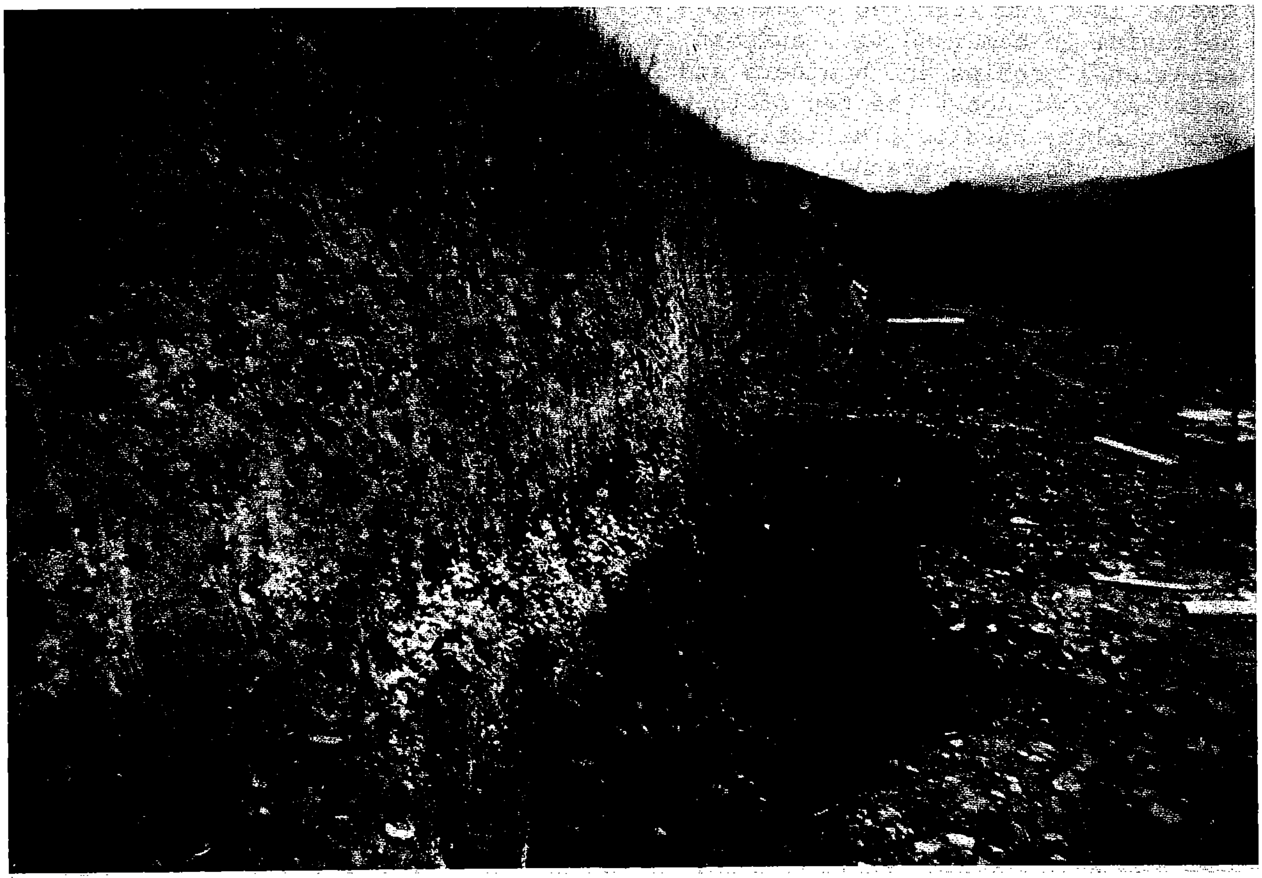
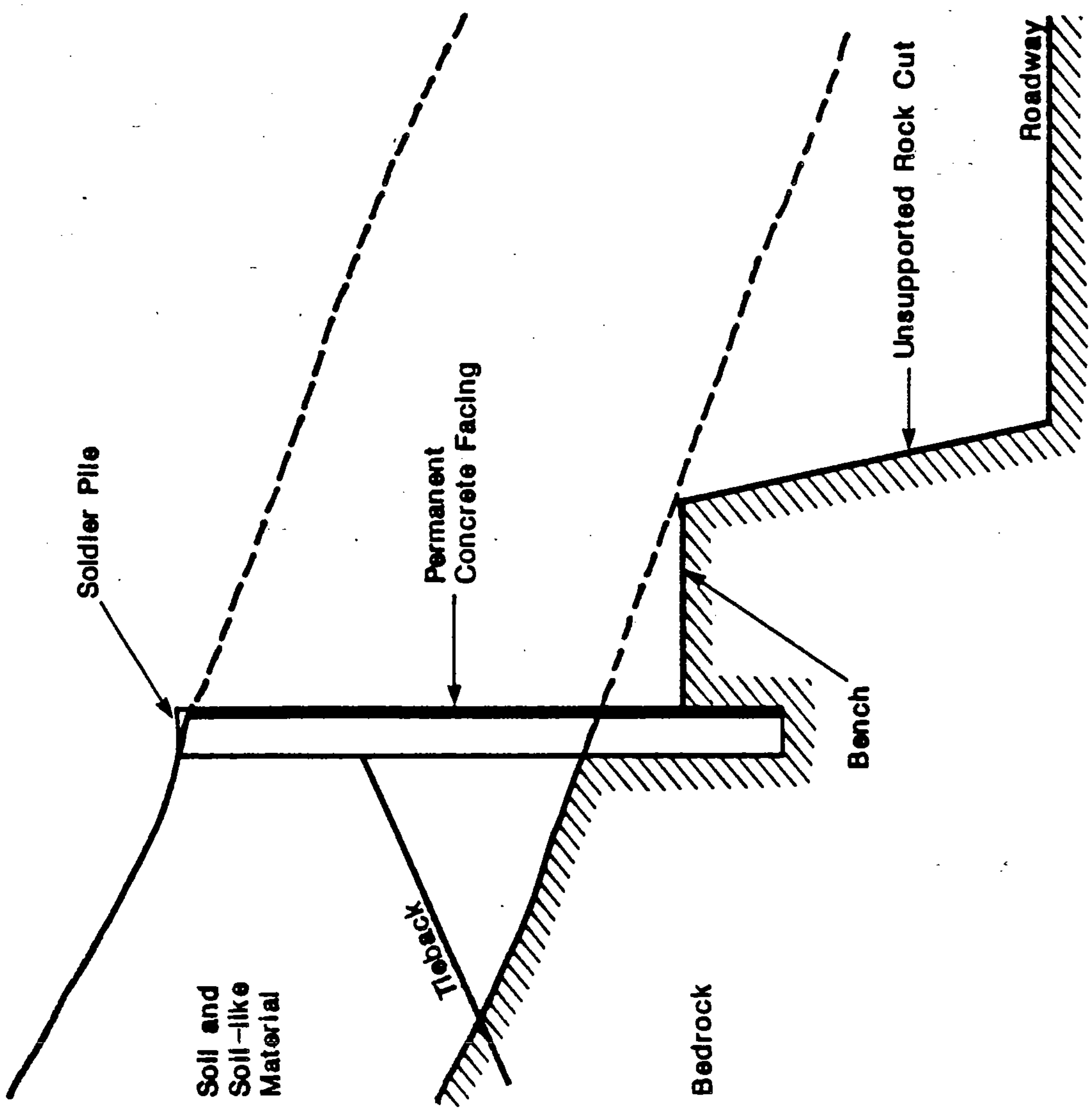
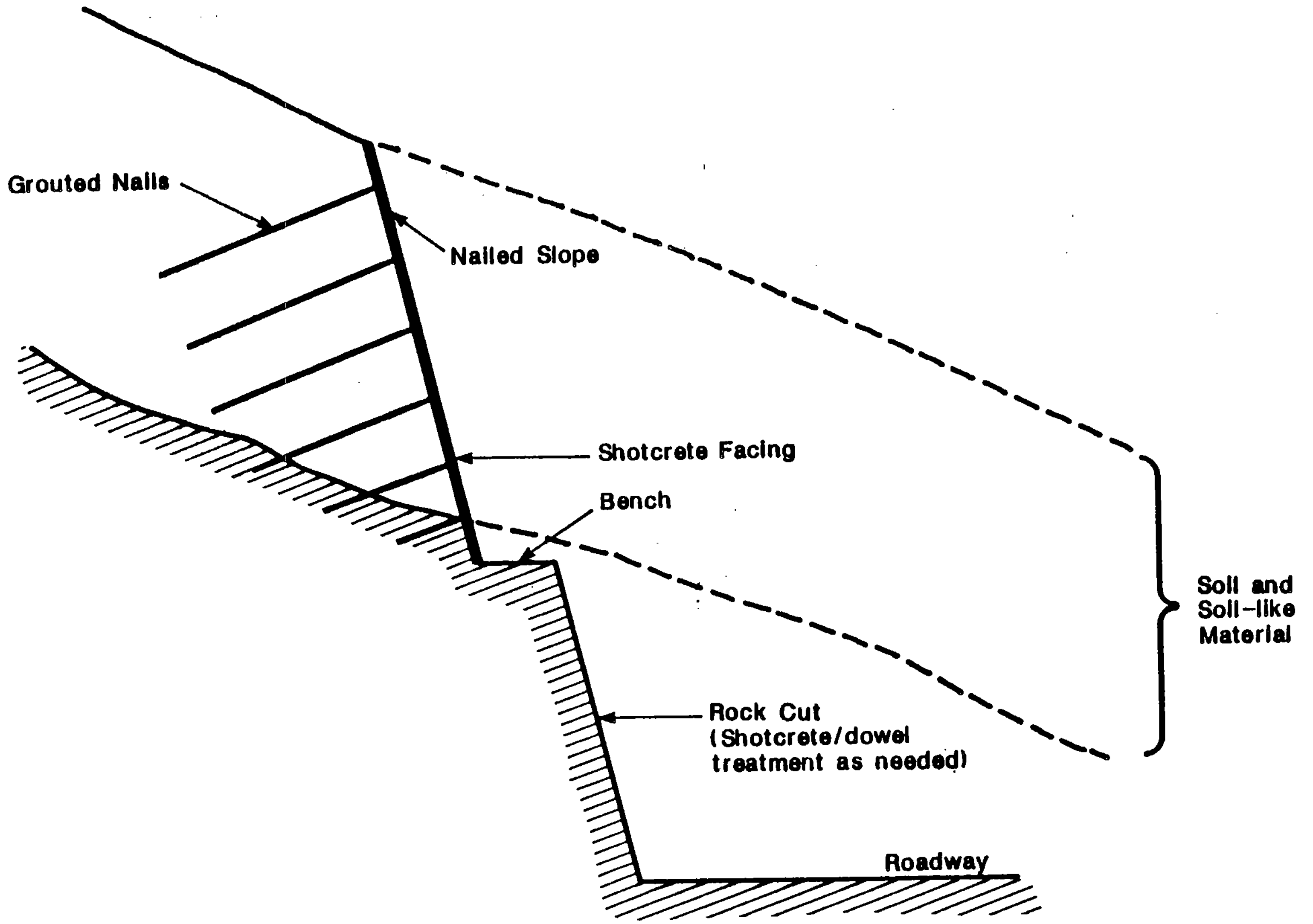


Figure 5. Typical Soil Conditions in Wall Area



No Scale

Figure 6. Soldier Pile Alternative



No Scale

Figure 7. Soil Nailing Alternative

completed based on the results of a field testing program that included nail pullout tests and stand-up tests. The stand-up tests consisted of excavation and monitoring of unsupported vertical and neat vertical cut slopes up to 20 feet high. Nail pullout tests were performed in both the colluvium and the underlying rock.

The final design of the wall is currently being completed. The general features of the wall will probably include:

- A uniform 1H:10V cut through both the soil and the underlying rock, for construction purposes a small bench may be required at the soil-rock interface.

- Soil nailing will be used to support the soil cut with limited localized support consisting of shotcrete and dowels in the rock.

- For aesthetic reasons, the entire slope will be covered with a concrete facia wall.

- Considerations are being given to strengthening the facia wall and using it to support the permanent design pressures between the nails.

- Nail spacing will be on the order of six feet with nail lengths on the order of 25 feet or shorter if they penetrate the rock.

- Due to a concern about frost jacking increasing the wall and nail loads, a conservative approach will be taken to drainage. Adequate drainage will be provided through a combination of weep holes and drainage fabric behind the wall possible supplemented with horizontal drains drilled into the ground in areas of heavy seepage.

The wall is scheduled for construction in 1989 with additional nailed slopes. WSDOT is currently considering the use of nailed slopes for other cut areas of the alignment.

CONCLUSIONS

Soil nailing is rapidly becoming an economical alternative to soldier pile walls for cuts up to sixty feet in medium dense to dense soils. Several factors need careful consideration prior to choosing nailing, including ground stand-up-time, soil-nail adhesion, and corrosion. It is anticipated that the nailed cuts on SR504 will provide one of the first examples of the successful use of nailing for permanent highway slopes in the western USA.

STEEL WIRE ROPE NET SYSTEM USED FOR PROTECTION AGAINST ROCKFALL AND DEBRIS FLOW AND FOR ALL OTHER MANNER OF PROTECTION

Robert A. Thommen, Jr.
Vice President & General Manager
Brugg Cable Products, Inc.

Rockfall and debris flow are most effectively prevented by using steel wire rope net systems. Emphasis is placed on the Company's engineering capability and testing as well as the use and importance of dampening devices that can absorb massive impact energies from within the net system. Outlining the planning criteria of such systems and briefly discussing our safety nets which can be used for prevention against other than rockfall, such as bridges and tunnels, highway and railway safety, etc. and all types of structures where additional protective measures, will ensure better safety records.

Introduction

Rockfall and debris flow are natural occurrences that in the wilderness can be brought about by erosion as well as animals and vegetation. Such incidents can also be caused by human interference. Today's expansion into previously uninhabited areas such as roads, housing, utility structures, recreational facilities, etc., have caused rockfall and debris flow occurrences to escalate. The result is an increase in danger zones and the risk of life and property. This in turn, increases our need for protective measures.

The types of protection which can be taken in order to prevent a disaster from occurring are as follows:

- Avoid any potential danger zones
- Areas which slope and where the forest has been destroyed by avalanches, recent increases in forest mortality and/or by cutting down timber for commercial use, have all reduced natural protection against such disasters. Ideally, these areas should be replanted with new trees which, in time, will act as a natural barrier.
- Installation of permanent rockfall and slope control defense systems.

Types of Defensive Systems

The following is a brief description of permanent preventative structures:

A.) Slope Erosion, Rock and Debris Flow Retaining Netting

Large wire rope nets placed and fastened on top of slopes, hillsides and mountains and/or covering overhanging rock formations in order to prevent loose rocks from tumbling down or breaking loose from the overhanging cliffs, and to prevent a potential hazard to the public and goods. (figure 1)

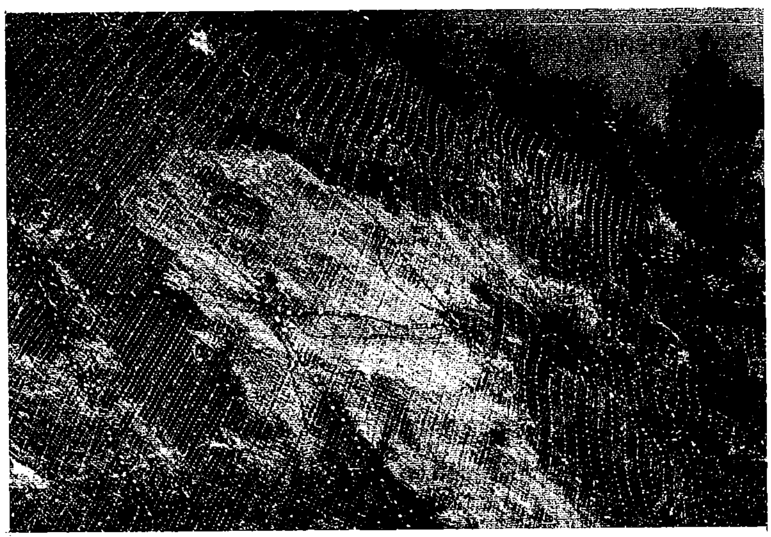


Figure 1 - Wire rope nets placed and fastened on top of slope

This system must be designed to withstand forces generated from the constantly sliding rocks within the permanently installed net system.

B.) Steel, Wooden and Concrete Type Support Structures for Retaining Rockfall and Debris Flow

Protective structures fabricated from heavy type steel, timber and made from concrete are called rigid type structures. If properly designed, they are able to withstand rockfall and debris flow effectively. By nature, rigid structures tend to absorb large kinetic impact forces poorly and therefore, in time, are most likely to require expensive repair in the field.

C.) <u>Wire Rope Nets Support Structures for the protection Against Rockfall and Debris Flows</u>

Made from lightweight, flexible wire rope net structures, capable of absorbing large kinetic energy, acting as a dampening device, rather than a hard stopping system. The net protective systems are labeled as being <u>flexible structures</u>. Characteristically, the net system reacts very favorably in stopping large and sporadically occurring impact energies (Figure 2) At this point in time, our attention is focused only on the importance of protective systems which use steel wire rope net systems.

Figure 2 - Wire rope net support structure

History

Nearly a half century ago, our Company recognized the advantages of wire rope netting for preventative structures and designed and installed the first wire rope safety system for prevention of snow avalanches. (Figure 3) Field testing of this net also indicated that the avalanche prevention net can be alternately used for protection against rockfall and debris flow.

The main objective of this development was to make full use of the flexibility of wire rope netting, its chief advantage over rigid type systems. Over the following years, numerous high impact tests have been performed in our factory and a new dampening device was developed and added to our rockfall protection systems.

Since then, the development of safety net systems continued rapidly. Today, there are a wide variety of sophisticated protective netting systems available for a multitude of applications.

Figure 3 - First wire rope safety net system made from wooden poles, using rectangular wire rope mesh.

Basic Design Criteria of Wire Rope Safety Net Protection Systems

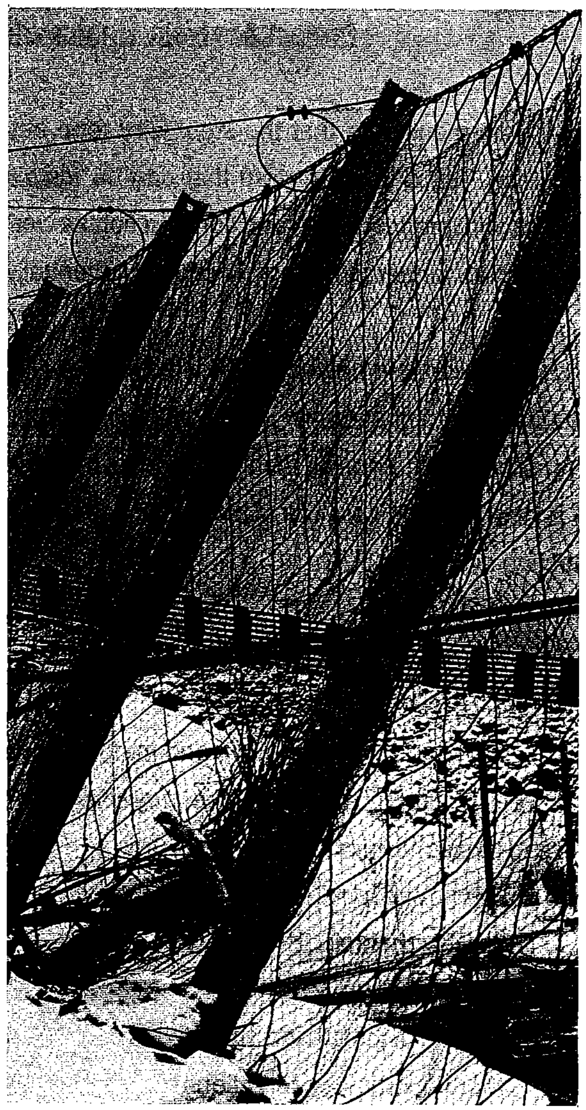
A.) Slope Erosion, Rock and Debris Flow Retaining Netting

As mentioned before, in order to prevent loose rocks from tumbling down or breaking loose from an overhanging cliff, etc., large wire rope nets are placed and fastened over these danger areas. At first, the wire rope is strung over the protective area, forming a grid and a net support system. The net support wire ropes must be anchored securely in the ground.

Once the wire rope grid is placed onsite, the individual nets are then placed and seamed with wire ropes to the net support ropes. Finally, commercial type cyclone fencing is spread and fastened to the wire rope netting in order to hold back smaller rubble, whereas the wire rope net, having a larger mesh size, is designed for stopping heavier sized rocks.

B.) Wire Rope Nets Support Structures for Protection Against Rockfall and Debris Flow

The purpose of these structures is to slow down and catch sporadically falling rocks and rubble. Enormous kinetic energy can be absorbed by utilizing wire rope nets that are equipped with dampening devices, and control the breaking forces and stopping distances under extreme load conditions. The breaking device is composed of a friction brake and wire rope loop and is activated only when the friction force reaches the preset value during heavier than normal rockfall conditions. Smaller rocks are stopped without operating the breaking element. The breaking device is built into the net support and suspension wire ropes and greatly increases their protective capacity at low cost. (Figure 4)


Picture 4 - Rockfall fence system with breaking devices

The dimension of the structures is determined by the maximum kinetic energy the system has to absorb and the height of the fence is dictated by the jumping height of the rocks.

Installation of these fences is rather simple due to the standardized design of the system. As many nets as necessary can be added in order to accommodate any particular length that the system is designed for. Foundation design data and detailed drawings are supplied by the manufacturer. Once the foundations have been poured, the setting of the net system support columns then occurs. The net support wire ropes are installed and the suspension wire ropes are securely anchored to the ground. Finally, the wire rope nets are placed and seamed with wire ropes to the net support ropes and cyclone fencing is spread and fastened to the wire rope netting.

The setting of the anchors into solid rock is a simple task. Although it must be mentioned that in less suitable terrain than that of bedrock, anchoring of the net system can create some problems.

For this reason, in 1980 an explosive wire rope anchor was developed and tested for breaking strengths of up to 70 Tons. The anchorage is made by inserting a pipe into a bore hole and detonating an explosive charge at the lower end of the pipe. A wire rope anchor is then inserted into the pipe and concreted into the pear-shaped cavity that was formed in the earth by the explosion.

Project Analysis, Engineering and Design Consideration

It should be mentioned that the steel wire rope net system like any other preventative measure, has its limitations. There is no system that can be installed that assures complete protection in all situations. However, with proper design and construction as well as an awareness of the surrounding conditions, a large portion of the danger can be avoided or substantially reduced.

Our construction and engineering teams will inspect each danger zone in order to analyze its needs. Any available document pertaining to the site will be studied in-depth. Gathering of information from people who are familiar with the area is also very helpful in providing the needed calculations for the network dynamics and anchorage and cost calculations. The following information is required to design a workable rockfall protection system:

1. Location and size of object to be protected
2. Slope profile
3. Terrain surface (vegetation, and material of top layer)
4. Terrain structure (rock, rubble, moraine)
5. Rock size, minimum and maximum
6. Frequency of rockfall
7. Trajectory (height, distance)
8. Velocity of falling rocks
9. Possibility of icefall and snowslides

*10. Other important observations

* NOTE: Rockfall fences could be exposed to snow avalanches and icefall and therefore to some extent, this has been taken into account when developing the layout.

More recently, computer programs are being used to calculate the velocity, jumping distance and the frequency of rotation of the rocks. This is necessary in order for the computer to simulate a rockfall trajectory and to analyze the complete behavior of each individual rock. After running as many as 500 or more rocks thru the program, precise information on rockfall behavior enables the engineer to develop efficient and cost-effective rock barriers.

Testing of Rockfall Protection Net Systems

Numerous high impact tests are being performed at our test facility by dropping heavy objects into wire rope nets from varying heights. Other tests were executed on site, in which cases heavy rocks were pushed down the slope into the nets.

These tests demonstrated that when using dampening elements, concrete blocks the size of a cubic yard and weighing in excess of 2,000 lbs. and falling from a height of 98 ft., have been totally stopped in the net system which remained undamaged.

Wire rope nets have been used for a number of applications, other than the prevention of rockfall and debris flow and have fulfilled their purpose with considerable success. Especially the following described avalanche prevention systems which have proven to be most effective in actuality.

Avalanche Prevention System

Avalanches are most effectively prevented when using support structures which are installed at the avalanche starting zone. Triangular shaped nets supported by flexible posts have proven to be most efficient and economical. As with the rockfall system, many nets as necessary can be added in order to accommodate any length which might be required. (Figure 5)

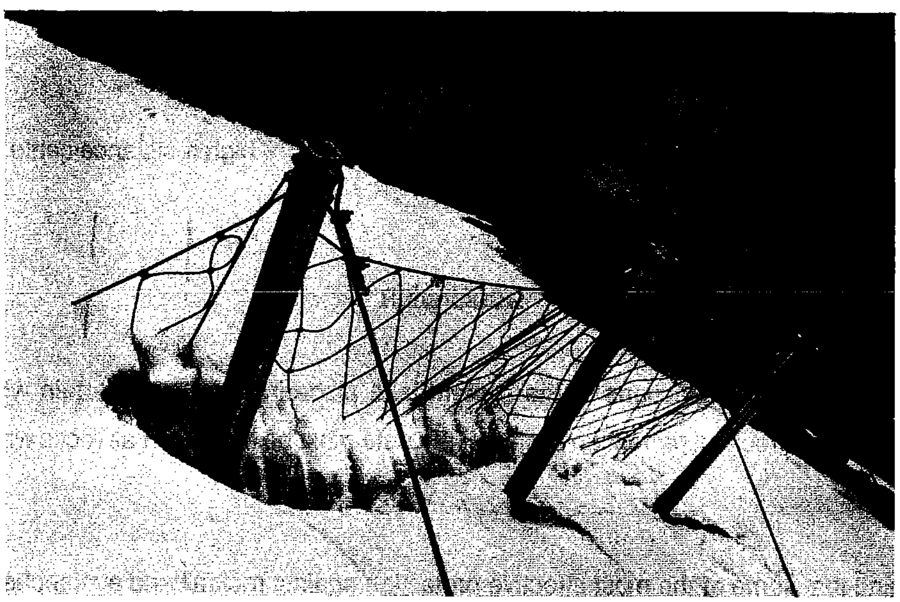


Figure 5 - Avalanche prevention system installed at the starting zone

length which might be required. (Figure 5)

The nets are securely anchored to the ground and are fastened to the posts. Suspension cables that are designed to carry large forces are secured to the posts and rigidly anchored to the terrain at the valley side of the slope. Constant forces are then diverted to the anchors of the nets, allowing the post to absorb most loads in its axial direction thereby preventing the post from bending and/or breaking.

Since 1951 when a patent was applied for by our Company for the snow avalanche prevention system, constant improvements and upgrading of our product was necessary in order to achieve a design which has proven itself in the field and which requires a minimum of maintenance.

All-Purpose Protection Systems

Wire rope high impact safety nets are used around buildings under construction, bridges and tunnels, offshore drilling platforms, helicopter landing pads and for highway and railway safety, etc., or any structure where additional protective measures will ensure a better safety record. The nets are being used for anti-attack measures in the protection of military, government, important infra-structures and industrial installations, against terrorist attack. Such net systems can be designed, using integrated electrical conductors which trigger an alarm if netting is cut and can be equipped with breaking elements to stop trucks and all types of vehicles in the net structure.

In some instances, specially engineered underwater nets composed of high resistance cable rings are being used in water to stop torpedoes, mines and frogmen.

Conclusion

It is important that periodic inspections of the system take place in order to ascertain what repairs are necessary and in case of rockfall protection systems, the nets must be cleared of all rocks which might have accumulated.

Also, keep in mind that some of the most effective protection against rockfall and avalanches is the natural forest and so therefore, if at all possible, in conjunction with the net barriers, new trees should be planted, thereby assuring a safer environment.

The wire rope nets available from our Company today have long passed the development stage and are of a proven design offering the following advantages:

- Made of lightweight construction and therefore easy to transport and install in difficult terrain
- Easy to repair on site due to its standardized design and ready availability of spare parts
- Environmentally pleasing and hardly visible from a distance
- Corrosion-resistant

In conclusion, we are pleased to report that over 1,000 systems have been installed by our Company on a worldwide basis.

4000 BRIDGE FOUNDATION INVESTIGATIONS
BY DAVID A. MITCHELL

"TOM MORELAND INTERCHANGE"
I-85/I-285 INTERCHANGE N.E. OF ATLANTA
ALL BRIDGE FOUNDATIONS ARE SPREAD FOOTINGS & "H" PILE FOOTINGS

INTRODUCTION

The author is the son of a construction worker and has lived in every state in the southeastern part of America. He received his B.S. Degree in Geology from the University of Southern Mississippi in 1958 and went to work for the Georgia Department of Transportation. He attended Georgia Tech and studied Soil Mechanics under Dr. George Sowers while with the Department of Transportation.

He is presently the Chief of the Geotechnical Engineering Bureau and is a Registered Geologist and a Registered Professional Engineer in the State of Georgia.

"4000 BRIDGE FOUNDATION INVESTIGATIONS
by David A. Mitchell

This is a story of my part in designing over 4,000 bridge foundations while working as a Geotechnical Engineer with Georgia Department of Transportation. Many of these bridges have mechanically stabilized embankments for abutments and range in length from 40 feet to 4 miles.

Our successful techniques in scour studies, subsurface exploration, and selecting proper foundations for certain geological conditions will be discussed. Not every state can use these techniques; however. It should be useful to know the basic successful bridge foundation investigation methods that have been employed in Georgia's Piedmont, Valley Ridge, and Coastal Plain Providences for the past 20 plus years.

A few foundation failures will be explained and costly corrective action taken will be emphasized which led to better designs at other similar sites. Typical maximum design loads for various pile types, spread footings, and drilled shafts will be given and typical geological conditions where one might limit these maximum design loads for economic reasons will be discussed such as a "Two Layered Site".

Main pier foundations will be installed for the New Talmadge Memorial Bridge this winter (1987-88) which is the largest bridge ever built in Georgia. Fresh slides will show foundations for the 450 feet high towers for this cable stayed structure. The writer will explain his thoughts in calling in a consultant for this project and what was gained by having a second opinion. Early results of the Pile Dynamic Analyzer's use at the site will be given and its importance discussed. Georgia Department of Transportation's test boring crews explored the site and drilled over 250 feet deep holes in the Miocene. Drilled shafts were designed for the anchor piers, which is a first in the Savannah area and was part of an extensive Test Pile/Test Load Project conducted in 1986.

Credit to Georgia Tech and Federal Highway Administration for their training, assistance and guidance is in order. Team type discussions lead to the highest loads ever placed on piling in the Georgia Coastal Plain. Detailed discussion of the basis for this dramatic increase in loading will be given along with the outcome of on site test loads during construction.

"4000 PLUS BRIDGE FOUNDATION INVESTIGATIONS"
METHODOLOGY

Georgia D.O.T. does 99% of its own Geotechnical work. This has been true since (1955-1960) when Dr. George Sowers helped D.O.T. establish a functional Soil Mechanics Laboratory and Soil Investigation Section. We were very fortunate that our office was located on Georgia Tech's Campus during these beginning years.

Orientation - Georgia D.O.T.'s Bridge Foundation Investigation Program calls for making an average of 200 Bridge Foundation Investigations per year. BFI's are performed by Crew Chiefs well trained in Geology and soil engineering by the professionals in the Bureau. At present 5 Graduate Engineers and two Geologists provide the training and direction required for our Crew Chiefs. Each Crew Chief of the six crews maintained has a minimum of 10 years exploration experience.

Spread Footings are usually the most economical foundation design and spread footings on soil require the most detailed B.F.I. Every site is drilled for spread footing initially and changed over to pile type borings when subsurface conditions are too poor. Close communication between the Engineer and the Crew Chief is vital on close decisions that often call for closer spacing between borings and samples.

Our Geologist makes scour studies at every stream crossing. To assist him he uses a Raytheon Model DE-719 Survey Fathometer. Utilizing field observations, fathometer profiles, and 50 and 100 year high water marks he assists our Engineers in placing foundations at proper elevations to discourage future scour problems. His data is also used to help design stone blanket protection and spur dikes when required. Aerial photographs are a must for every major stream crossing. The history of stream channel movement must be determined.

Economics have for a long time dictated our general pile foundation types for each major Geological Providence in the State. For example:

Prestressed Piles are used most often in the Coastal Plains and at other sites where they can be driven without damage. Prestressed piles have proven to be relatively maintenance free in our acidic and marine environments that are usually highly corrosive to steel piling.

Design methods for piles are based on standard penetration tests, trixial test, unconfined compression tests and experience. Load tests are used to calibrate hammers and are performed at all major stream crossings. Recently the pile driving analyzer has been used effectively to reduce some load tests and to better evaluate hammer performance. Jetting, predrilling and spudding are routine ways we allow to get the piles in when driving is tough.

Steel "H" Piles are used primarily in the Piedmont and Valley and Ridge Providence. Soils penetrated are chiefly weathered Shales, Cherty clays, Schist, and other Saprolites. Usually these piles are end bearing piles with tips seated on or in rock. Pile points are used to reduce skin friction and prevent pile damage when driving thru hard rock layers, boulders or debris.

Metal Shell Piles are used where pile lengths are expected to be erratic and where displacement type piles are desirable. Pilot holes are used occasionally at some sites to penetrate hard layers that can't be driven thru. Metal shell piles are used mainly in the lower Piedmont and in Karst Topography such as that found in the Coastal Plain (Ocala Limestone). "H" piles often are too long (over 100 feet) at these sites where metal shell piles often save at least 25% in pile length. Balken piles and Tapered Montotube Piles are also good piles for many of these Karst sites. All (MSP) pipe piles are filled with concrete.

Drilled Shafts provide excellent foundations at practically any site. However, they are used mainly in our Piedmont at present and are founded on or in rock. Most of our contractors are equipped to drive piles; consequently, they bid drill shaft jobs higher.

Alternates - Alternate foundation plans often keep prices down by introducing more bidders and material suppliers. Federal Highway deserves a lot of credit for insisting on alternate designs. I predict that eventually drill shafts will be used more as drilling and design technology expands to all areas of the Country. Drill shafts are excellent foundations for most all major stream crossing and offer the foundation engineer the best scour protection at many river crossings.

Fill Settlement - Most of our major foundation problems can be contributed to fill settlement. We routinely build all sizable end fills first and construct foundation after an estimated waiting period which is refined during construction by survey points placed on the fill and at the toe.

Downdrag protection is used when piles are driven thru consolidating fills and on piles driven first then backfilled around as is often true with mechanically stabilized earth walls. We use pilot holes, CMP "Cans", Bituminous coatings, polyurethane coatings and more recently plastic cardboard to help prevent downdrag. (See attached working specification)

We have excellent communication between ourselves and the Bridge Design Office and with construction. On critical wall and bridge construction problems we assist the Resident Engineer by furnishing an engineer on construction to help implement our more complicated wall and foundation designs.

Our files include "as built" data back to 1960 and are used when designing new structures routinely. Every bridge built in Georgia since 1960 has a foundation investigation on file.

MAXIMUM PILE BEARING

MS PILES

12" MS = 41 TONS
14" MS = 49 TONS
16" MS = 65 TONS
18" MS = 88 TONS

TIMBER PILES

TIMBER PILES = 24 TONS
LENGTHS > 40' - CLASS A
LENGTHS < 40' - CLASS B

SQUARE PSC PILES

12" PSC = 43 TONS
14" PSC = 64 TONS
16" PSC = 82 TONS
18" PSC = 86 TONS
20" PSC = 89 TONS
24" PSC = 95 TONS
30" PSC = 120 TONS (180)
36" PSC = 150 TONS (220)

ROUND PSC PILES

36" PSC = 134 TONS (176)
54" PSC = 197 TONS (260)

H PILES	9 KSI	12 KS
8 BP 36 =	32 TONS	
10 BP 42 =	55 TONS =	73
12 BP 53 =	70 TONS =	93
14 BP 73 =	96 TONS =	128

BALKEN PILES = 55 TON
45' MAX. LENGTH

THE B.F.I. REPORT

Definitions of Georgia's Terms Used in Foundation Design

By: David A. Mitchell

1. Minimum Tip Elevation: The highest permissable elevation at which a pile tip may be stopped.

   Minimum tip elevations are set during the bridge foundation investigation for the following reasons:

   a) To insure that piles have adequate lateral support.

   b) To insure that piles have adequate protection from stream scour and erosion.

   c) To insure that piles are not stopped above dangerously soft soil layers that may cause single or group settlement of piles.

2. Estimated Tip Elevation: This is the tip elevation that the Geotechnical Engineer has estimated for the type piles listed in the BFI driven to the maximum bearings given.

   The Geotechnical Engineer uses the boring logs, test results, occasionally computer programs, and past experience to arrive at this estimate.

   His estimate does not take into consideration the actual design loading of the pile or the type hammer that may be used to drive the pile. The quality of field supervision during the actual driving will also influence the actual tip elevation. Piles not reaching bearing by formula must be test loaded.

3. Maximum Design Loads: The attached table gives the maximum normal bearings that the pile can safely support taking into consideration the dynamic force needed to place it in the ground as well as its material strength.

   Guidelines for the material strength come from standards arrived at by AASHTO. Sometimes these maximum bearings cannot be achieved at a particular bridge site. If this is true, then the maximum bearing must be altered or lessened at some sites to achieve a safe pile foundation design with reasonable lengths.

Most of the time the Geotechnical Engineer is able to provide the designer this data in the BFI. Sometimes the Geotechnical Engineer has insufficient information or borings to set the maximum design loads; we then have problems during construction which we are forced to request additional information from the Project Engineer and work as a team to solve.

Additional information that may be needed is:

a) Test pile driving records

b) Test loads

c) Additional test borings

d) As built data at other bents

Many bridge foundation problems are complex; consequently, we must work as a team to safely and economically solve these problems.

4. Plan Driving Objective: The PDO given in the BFI assures the Geotechnical Engineer that the pile is placed in firm material capable of supporting the design load placed on the pile.

Most of PDO's will read as follows:

a) Driving resistance after a minimum tip elevation of __________ is achieved.

b) Driving resistance after at least 15 feet of penetration is achieved.

c) Practical Refusal after a minimum tip elevation is achieved.

5. Spread Footings: A spread footing is the usually square base at the bottom of a column which is used to distribute the column load to the foundation material.

Actually many of our footings are rectangular and in the case of drilled shafts they are round. Retaining walls have long strip footings that are sometimes stepped.

The maximum allowable bearing capacity given in the BFI report is used by the designer to determine the actual size of the footing. Normally, the maximum design loads recommended by the Geotechnical Engineer are as follows:

<u>Footings Founded in Hard Rock 4 to 15 Tons per Square Foot</u> = Square rectangular footings on rock. Often 5 tons is used since past experience shows that a sensible size footing usually results with this loading.

<u>16 to 50 Tons per Square Foot</u> = Round drilled shafts usually keyed into rock. Higher capacities are common for drilled shafts due to generally better inspection control which insures the bottom of the shaft is seated in hard fresh rock. In addition, tests show that often additional bearing is available from side friction.

<u>Footings Founded on Soft Rock 2 to 5 Tons per Square Foot</u> = Many footings are designed to be placed on soft rock materials that do not normally require blasting to excavate. Some of these materials are marls, kaolin, shale, schist, weathered granites, sandstone and limestone.

<u>Footings Founded on Dense Soil 1 to 4 Tons Per Square Foot</u> = Most footings placed on dense soil are design in this range. The standard penetration test is normally the main test used in designing most soil footings. A copy of one of the design charts we might use in designing a spread footing is attached.

In summation, spread footings are often our most econimical foundation design particularly at grade separation structures.

6. <u>Groundwater Level:</u> To us the ground water level is the highest measured water level in a drill hole measured after a period of at least 24 hours.

The BFI will always show the location of the water level at each site at the time it was explored. Ocassionally we install long term observation wells at critical bridge or retaining wall sites. Fluctuations in the GWL of 10 feet are not uncommon in holes checked through seasonal changes.

7. <u>End Fill Settlement Waiting Periods:</u> The BFI calls for allowing newly constructed end fills to be allowed to settle for an estimated time period before driving piles or constructing foundations in the end bents and sometimes adjacent intermediate bents.

The estimated waiting period is determined by the Geotechnical Engineer after studying the test boring and making calculations using laboratory consolidation tests. Many caps have been broken by fill settlement and even some columns have had to be torn out and replaced due to fill settlement.

Fill Settlement may be divided into categories as follows:

<u>Primary Settlement:</u> This is the settlement that occurs rapidly most of which takes place during the actual construction of the embankment or end roll.

<u>Secondary Settlement:</u> This is the very slow settlement that takes place months or sometimes years after the embankment has been placed. The actual time that it is safe to construct bridge foundations at sites where long term settlement will occur is hard to predict accurately and usually requires settlement gauges or points that can be monitored by project personnel.

Waiting periods given in the BFI will normally vary from 30 days to 24 months depending on the Geotechnical Engineer's estimate. Usually a site having a 30 or 45 day waiting period is not very critical and the anticipated settlement will be mostly primary settlement.

On waiting periods longer than 45 days the proper time to allow foundation construction can be determined more accurately by installing and reading with a level TBM's placed on the endfill. Three feet long 1/2-inch diameter steel rods driven into the soil at least 30 inches are most often used for this purpose. These are placed at the top of the slope and read weekly to determine measurable settlement. When we are asked to review these readings for a possible early release, we like to see at least 3 weeks of readings with no measurable settlement.

Occasionally settlement is still occurring and the waiting period has expired. It is very important that a good instrument man is taking these readings, since a lot of plans hinge on the accuracy of these readings. The Geotechnical Engineer may allow piles to be driven even if minor settlement is still occurring. Two methods we have used in protecting piles from fill downdrag are as follows:

1. <u>Provide Pilot Holes</u> to reduce the fill's grab or adhesion to the pile. Bentonite is often necessary to keep holes open in sandy soil.

2. Drive the piles but <u>withhold construction of the pile cap</u> until at least 3 weeks of no measurable settlement.

There is another method that we have developed about two years ago See attached copy of the specification covering this method. This method is our preferred treatment for downdrag protection in the Piedmont.

8. <u>Corrosion Protection:</u> A study is made at each bridge site to evaluate the problems that may be encountered with the corrosion of steel piles or concrete foundations.

   Chemical analyses are made of ground or surface water present and of the soil that may come in contact with the foundations. Our field engineers try to inspect existing structures located over the same streams to see how corrosion has affected the substructure materials. Sometimes we also contact the Resident Engineer and secure his experience with corrosion.

   Corrosion recommendations consist of Epoxy Coatings, Concrete Encasement, and Cathodic Protection. All "H" piles used as pile bent are now encased in concrete on stream crossings.

LOGGING THE TEST BORING

SOIL STRENGTH DETERMINATION

I. When standard penetration "blow" counts are available use the following:

GRANULAR SOIL		CLAY	
BLOWS	DENSITY	BLOWS	CONSISTENCY
0 - 4	Very loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 24	Medium Dense	5 - 8	Medium Stiff
25 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		31 - 60	Hard
		Over 60	Very Hard

II. When shear strength is known from unconfined compression, triaxials or vane shear tests the consistency of cohesive (clayey) soil may be determined using the following:

CONSISTENCY OF CLAY	SHEAR STRENGTH (½COMPRESSIVE STRENGTH IN LB/SQ FT)
Very Soft	<250
Soft	250 - 500
Medium	500 - 1000
Stiff	1000 - 2000
Very Stiff	2000 - 4000
Hard	>4000

III. When no tests are available use judgement or estimate strength. For example, a sandy clay that is difficult to auger through maybe "very stiff." A sand that appears to be stream deposited and washbores, very easily may be estimated as "loose." A sandy silt that is very difficult to penetrate washboring may be estimated as "dense." When drilling with truck mounted power augers or rotary drills such remarks as medium hard drilling, hard drilling, very hard drilling and etc. are also very usefull in estimating soil strength. (See AASHO Manual on Foundation Investigations for Additional Information)

SOIL CLASSIFICATION IN GENERAL

Logging or classifying soil should be performed using the following sequence of discription terms:

1 2 3 4
Strength - Color - Minor Soil Content - Major Soil Content -

5
Geological or Local Name (if any)

For Example:

1 2 3 4
Very Dense Gray Silty Sand

1 2 3 4 5
Dense White Sandy Silt (Kaolin)

1 2 3 4 5
Stiff Gray Silty Clay (Gumbo)

1 2 3 4 5
Loose Brown Silty Sand (Stream Deposits)

To add the field unified classification system is very easy if you remember the soil initial is reversed. For example:

SM = Silty Sand
SC = Clayey Sand

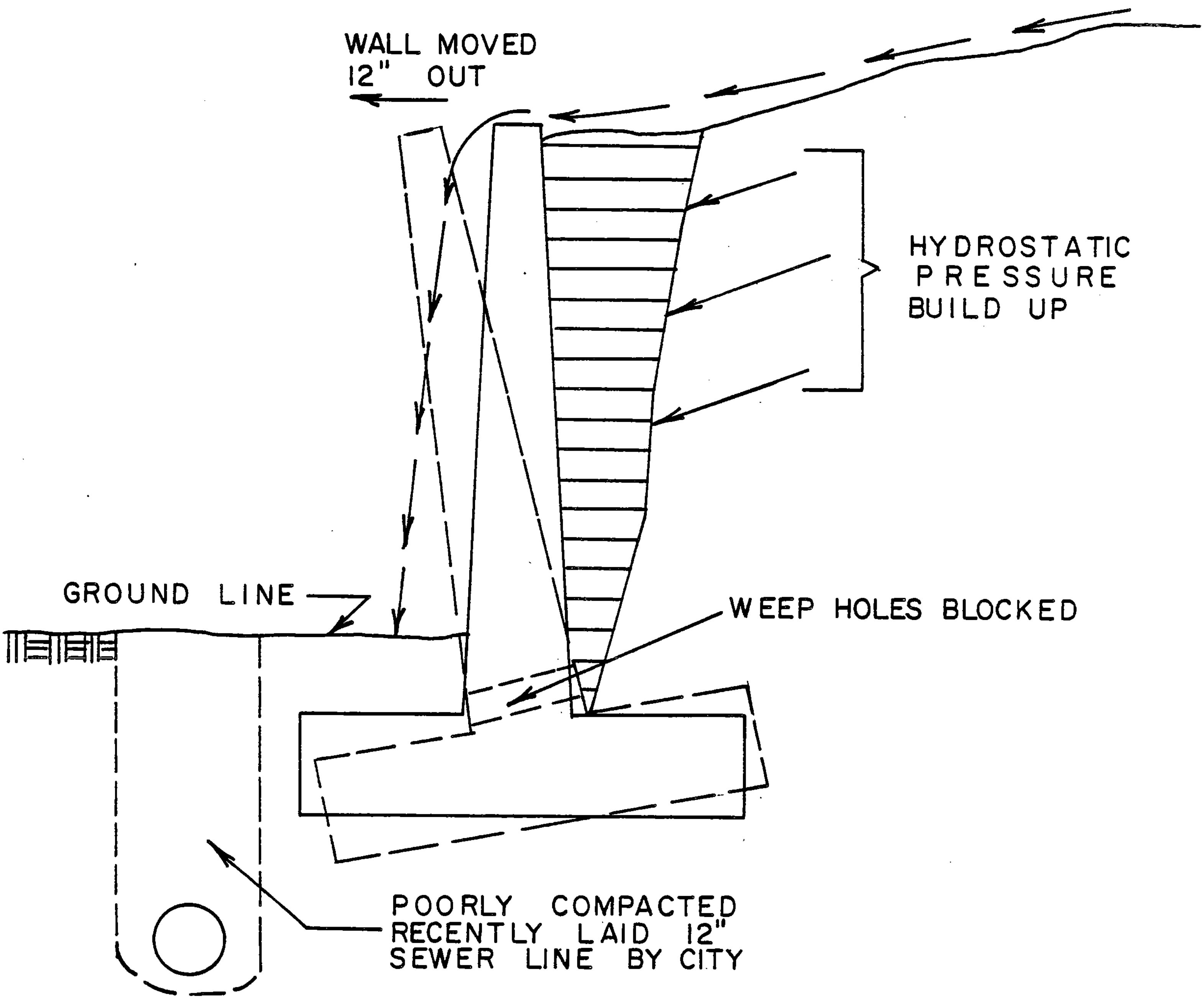
When referring to a soil's plasticity characteristics such as lowly plastic or highly plastic this may be placed in between the color and minor soil content if desired. For example:

1 2 3 4
Very Soft Gray Highly Plastic Silty Clay
(CH)

NOTE: The plasticity characteristics could be omitted since the unified letters CH also mean highly plastic clay.

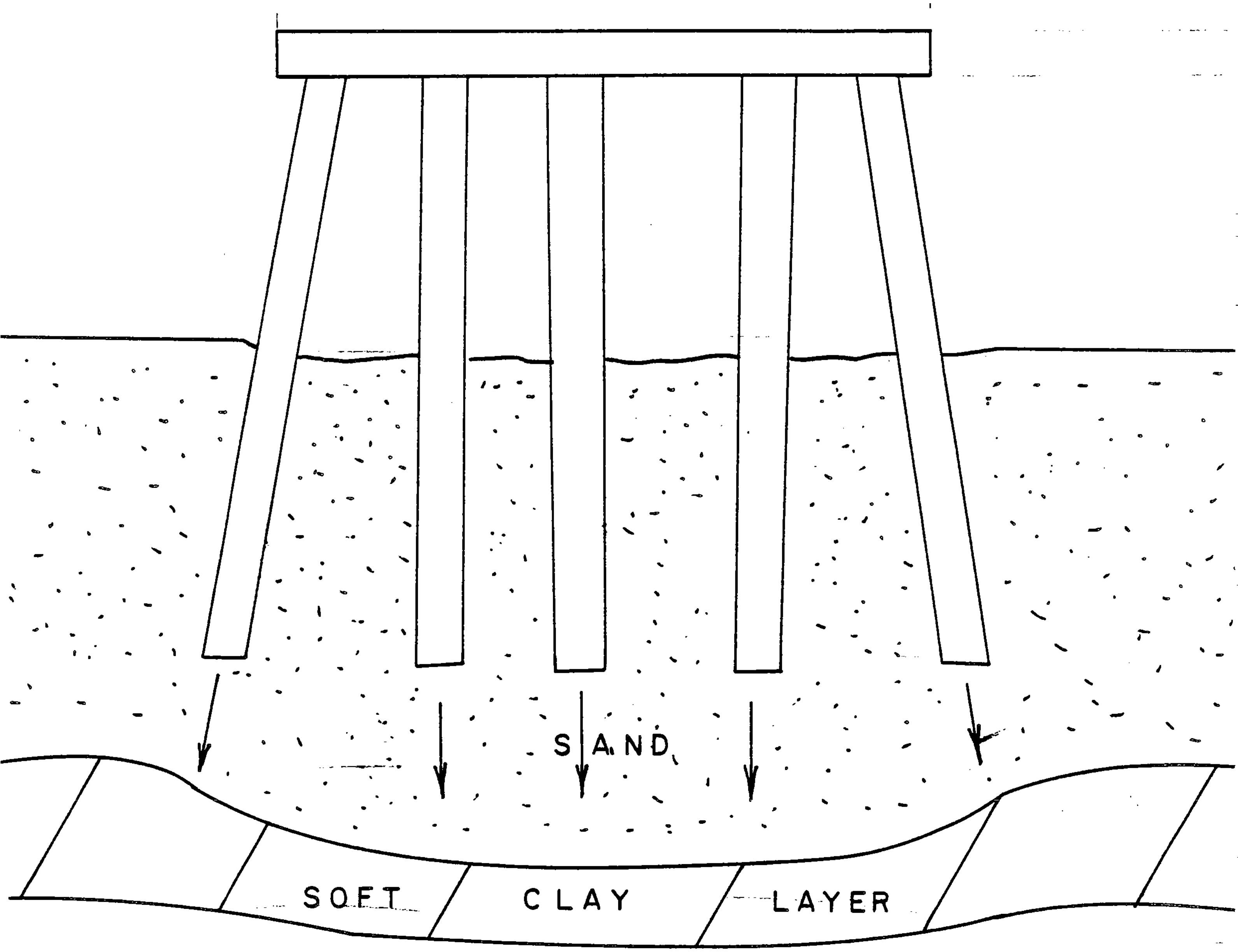
SKETCH NO. A

MOVEMENT OF RETAINING WALL DUE TO UNDERMINING OF TOE AND EXCESSIVE HYDROSTATIC PRESSURE



SKETCH NO. B

GROUP SETTLEMENT OF PILE BENT



November 21, 1983
Rev October 15, 1986
REV FEBRUARY 11, 1987

DEPARTMENT OF TRANSPORTATION

STATE OF GEORGIA

SPECIAL PROVISION

Addition to Standard Specifications

SECTION 551 - PILE PROTECTION IN EARTH WALLS

551.01 GENERAL: This Specification covers the material requirements and options available to protect bridge end bent piles which are located in the stabilized backfill of earth retaining walls.

551.02 CONSTRUCTION: When the Plans require protection of end bent piles from negative skin friction the Contractor shall provide such protection by utilizing one of the following methods:

A. After the end bent piles have been driven and before installation of the earth reinforcing elements, the Contractor shall place over each pile a cylindrical can which shall prevent the earth wall backfill material from coming into contact with the pile. The can shall be large enough in diameter to give 1" minimum clearance from the pile to the inside of the can. Spacers shall be placed between the pile and the can to prevent the can from coming into contact with the pile during backfilling of the wall. Cans shall extend from the bottom of the earth stabilized backfill to the bottom of the bridge end bent cap. After the cans are positioned, they shall be effectively sealed at the top during the period of time of placement of backfill to prevent rubbish or aggregate accumulation in the can, and shall remain sealed until fill settlement time has expired. When the backfill for the wall has come to the level of the bottom of the cap, and all fill settlement time has expired, the cans are to be filled with aggregate.

B. The second option involves applying an asphaltic layer to the piles. The end bent piles shall be primed, if necessary, and uniformly coated with an asphaltic material. If piles are delivered well in advance of placement, they may be primed to retard rusting thus avoiding cleaning and priming operations prior to coating with asphaltic material. Priming before coating will depend upon the condition of the piles and will be performed as deemed necessary by the Engineer to obtain bonding of asphaltic material to the piles. No separation of asphaltic material from the piles will be allowed prior to driving of the piles. If more than 10% of the asphalt separates from the piling as judged by the Engineer, the coating will be replaced while the piling are being driven.

Application of the prime coat shall be with a brush or other approved means and in a manner to thoroughly coat the surface of the piling within the limits designated with a uniform continuous film. Primer may be applied without heating and when deemed necessary may be thinned to a suitable brushing consistency with a volatile solvent. The primer shall be permitted to dry thoroughly before the asphaltic slip layer is applied

Asphalt for the slip layer shall be heated to 200 to 300 degrees Fahrenheit and applied by one or more mop coats or other approved means as necessary to obtain an average coating of 10 mm (0.4"). The piling shall be completely coated at least for the entire length exposed to the earth stabilized backfill after driving.

C. The third option involves mopping the piles with a corrosion-inhibiting grease with coverage as follows:

Steel Piling = 1/16 Inch Minimum
Concrete Piling = 1/4 Inch Minimum

No grease shall be applied until after piles are driven and only that portion of the pile to be in contact with wall backfill shall be treated.

In addition to the grease a urethane OR PROPYLENE sleeve shall be installed to protect the GREASE coating from the backfill.

Sleeves may be sprayed on or preformed. Portions of THE SLEEVE damaged or removed by construction activities during backfill placement shall be replaced.

551.03 MATERIAL

A. Cans placed over piling may be smooth or corrugated steel pipe of sufficient thickness to prevent buckling during the placement and compaction of the earth stabilized embankment. Cans shall be coated both inside and outside with 2P coating as per Section 535.04.D of the Standard Specifications or may be galvanized as per ASTM A-123.

B. Primer used for coating piles shall conform to AASHTO M-116 (ASTM D-41).

C. Asphaltic material for the slip layer shall conform to AASHTO M-115 Type II (ASTM D-449).

D. Aggregate for the backfilling of the cans shall conform to Section 806 or 801 of the Standard Specifications.

E. Corrosion INHIBITOR (GREASE) shall conform to the following test requirements:

1. Drop point 350 degrees F Min By ASTM D-566,
2. Flash point 350 degrees F Min By ASTM D-92,
3. Water content 0.1% Max By ASTM D-95,
4. Rust test MEET ASTM D-1743, and
5. Water soluble ions:

Chlorides	10 PPM Max	By ASTM B-512
Nitrates	10 PPM Max	By ASTM D-992
Sulfides	10 PPM Max	By APHA 427D(15th ED)

F. POLYURETHANE FOAM:

Min. Density 1.5 P.C.F. (ASTM D-1622)
Compressive Strength Perpendicular 16 @ 6% (ASTM D-1621)
Only foam approved for commercial use in this state shall be allowed.

G. POLYPROPYLENE FLUTED SHEETS:

"PLASTIC CARDBOARD" ULTRA-VIOLET STABILIZED. SCORED OR GREASED TO FOLD AROUND PILING AND INTO "H" PILE WEB MINIMUM LENGTH 48 INCHES. SECTIONS TO BE ADDED WITH MINIMUM 3-INCH (SHINGLE STYLE) OVERLAP.

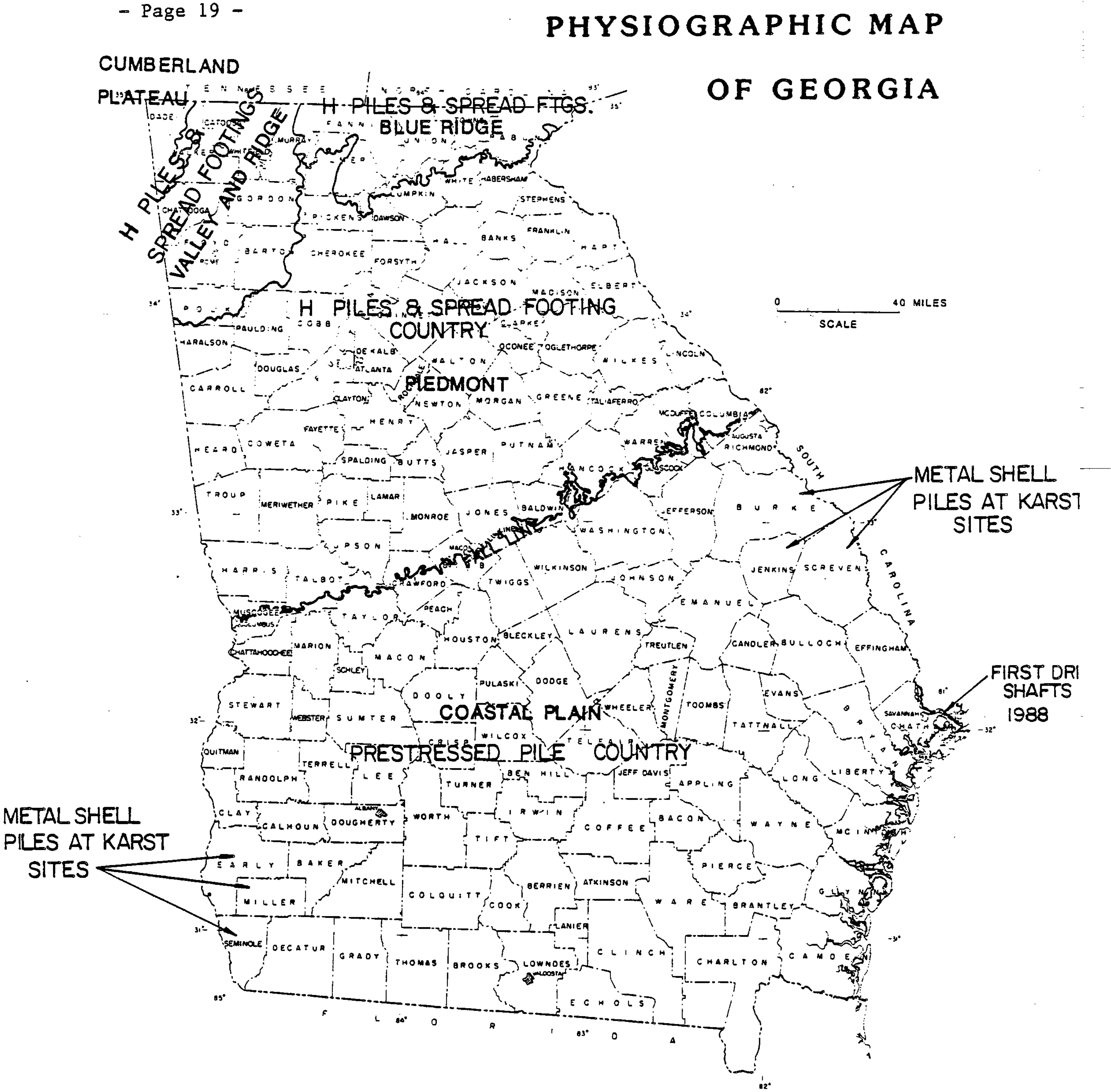
H. DUCT TAPE:

SHALL BE USED FOR PATCHING AND SECURING OF BOTH PLASTIC CARDBOARD AND POLYURETHANE. NO DUCT TAPE SHALL BE IN DIRECT CONTACT WITH THE GREASE OR PILE. DUCT TAPE SHALL BE USED IN SANDY BACKFILL TO SEAL OVERLAPS AND PREVENT SAND INFILTRATION.

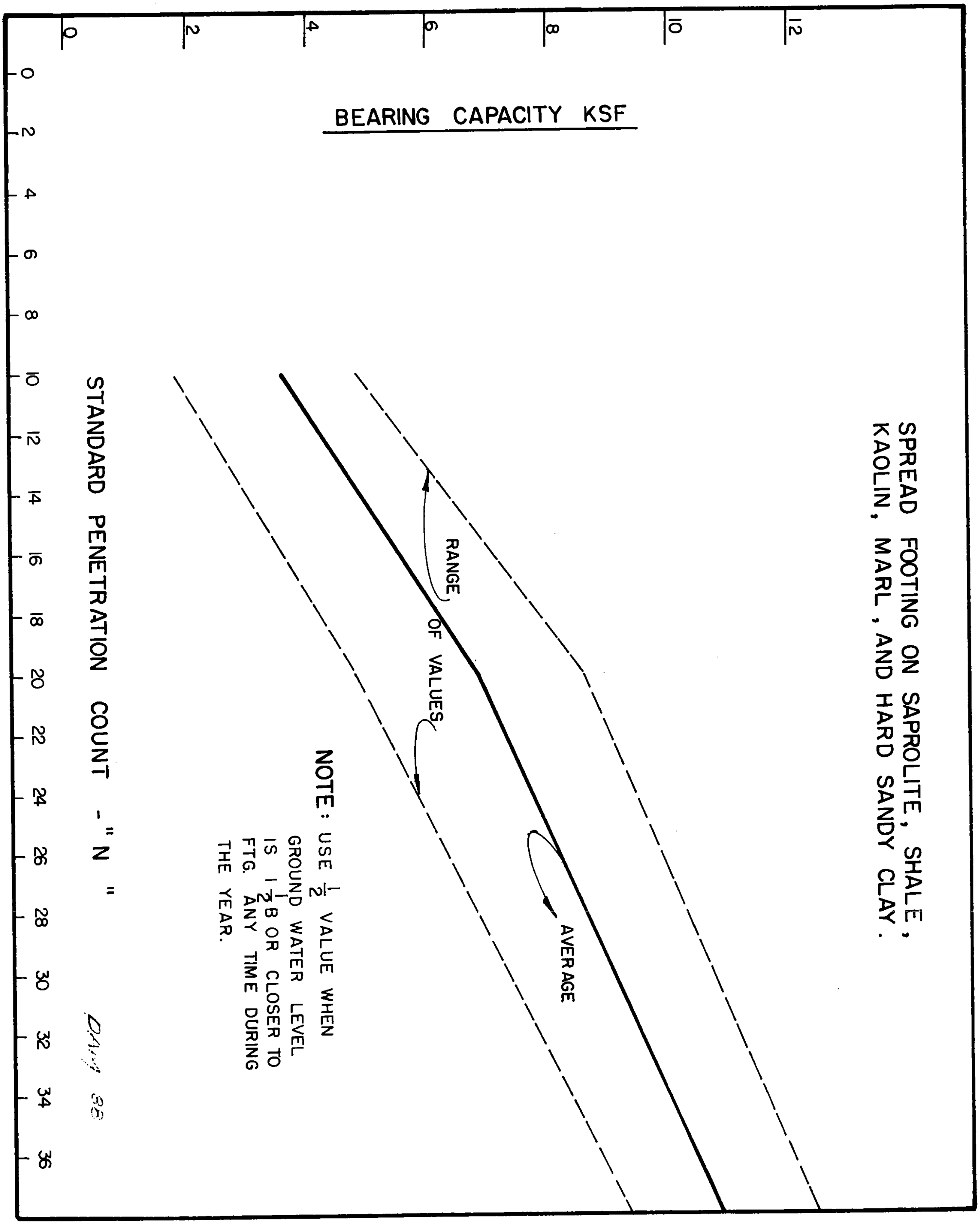
551.04 MEASUREMENT AND PAYMENT: No separate measurement for payment purposes will be made of the materials and labor required to conform with this Specification. All costs incurred in complying with this Specification shall be included in the bid price for the piling.

BDM/BRIDGE

PREDOMINANT BRIDGE FOUNDATION DESIGNS ACCORDING TO PHYSIOGRAPHIC LOCATION



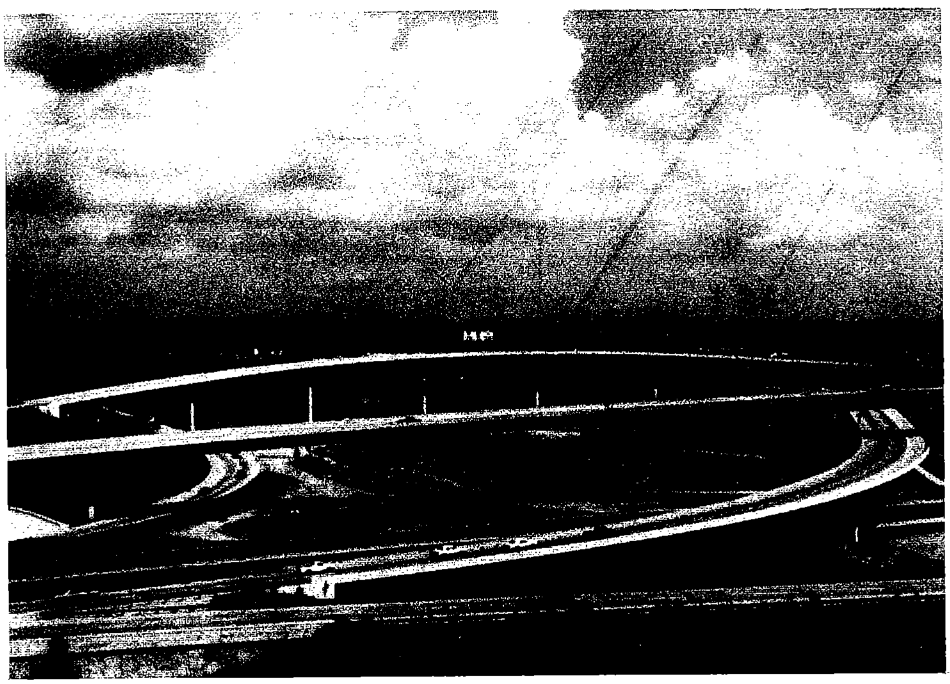
NOTE: MAJOR RIVER PIERS OFTEN FOUNDED ON "H" PILE FOOTINGS IN COASTAL PLAIN


SPREAD FOOTING ON SAPROLITE, SHALE, KAOLIN, MARL, AND HARD SANDY CLAY.
BEARING CAPACITY KSF
0 2 4 6 8 10 12
STANDARD PENETRATION COUNT - "N"
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36
RANGE OF VALUES
AVERAGE
NOTE: USE 1/2 VALUE WHEN GROUND WATER LEVEL IS 1 1/2 B OR CLOSER TO FTG. ANY TIME DURING THE YEAR.

"TOM MORELAND INTERCHANGE"

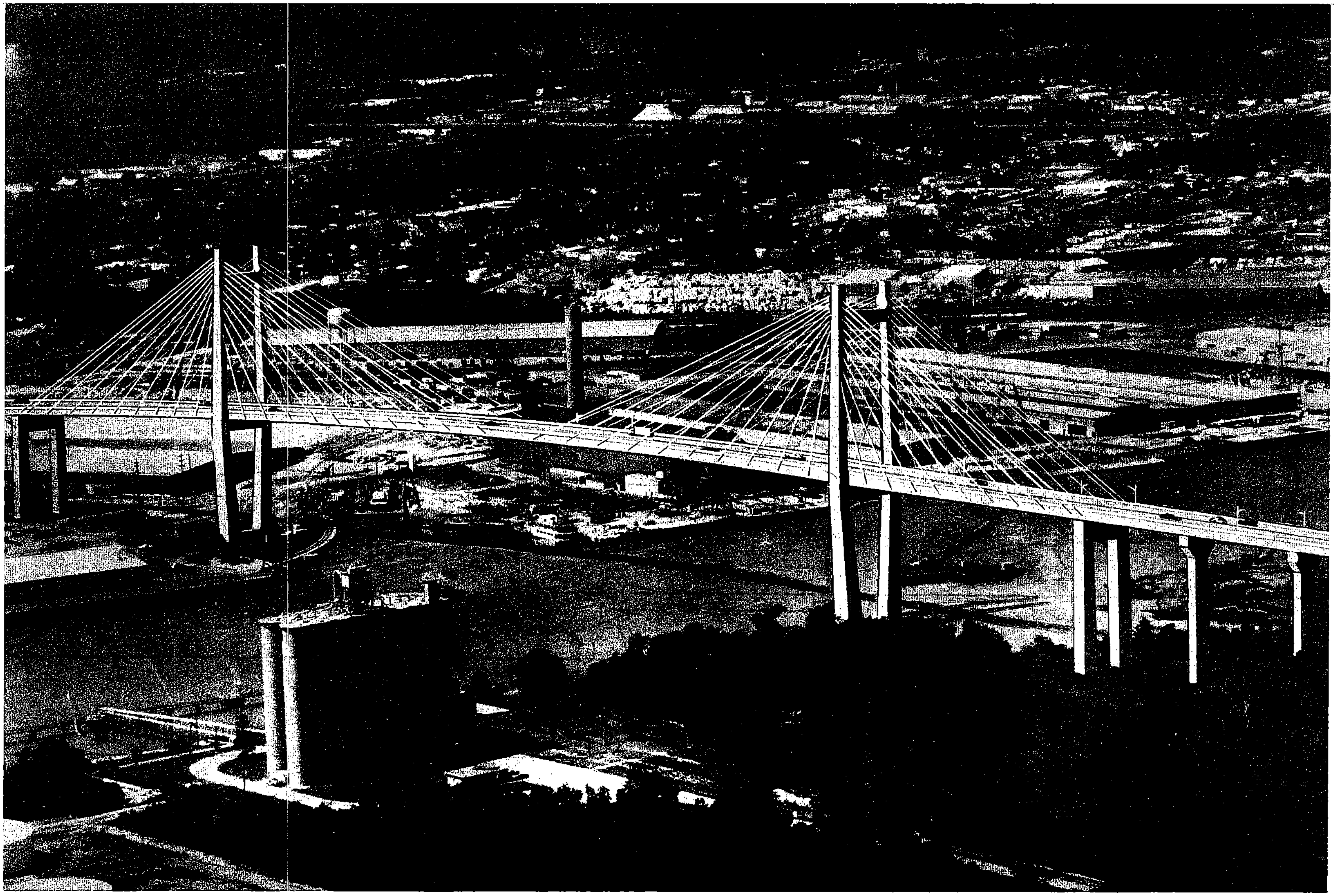
I-85/I-285 INTERCHANGE N.E. OF ATLANTA

ALL BRIDGE FOUNDATIONS ARE SPREAD FOOTINGS & "H" PILE FOOTINGS



NOTE: GEORGIA STABILIZED EARTH WALLS IN FOREGROUND
TWO LARGE TIEBACK WALLS NOT SHOWN

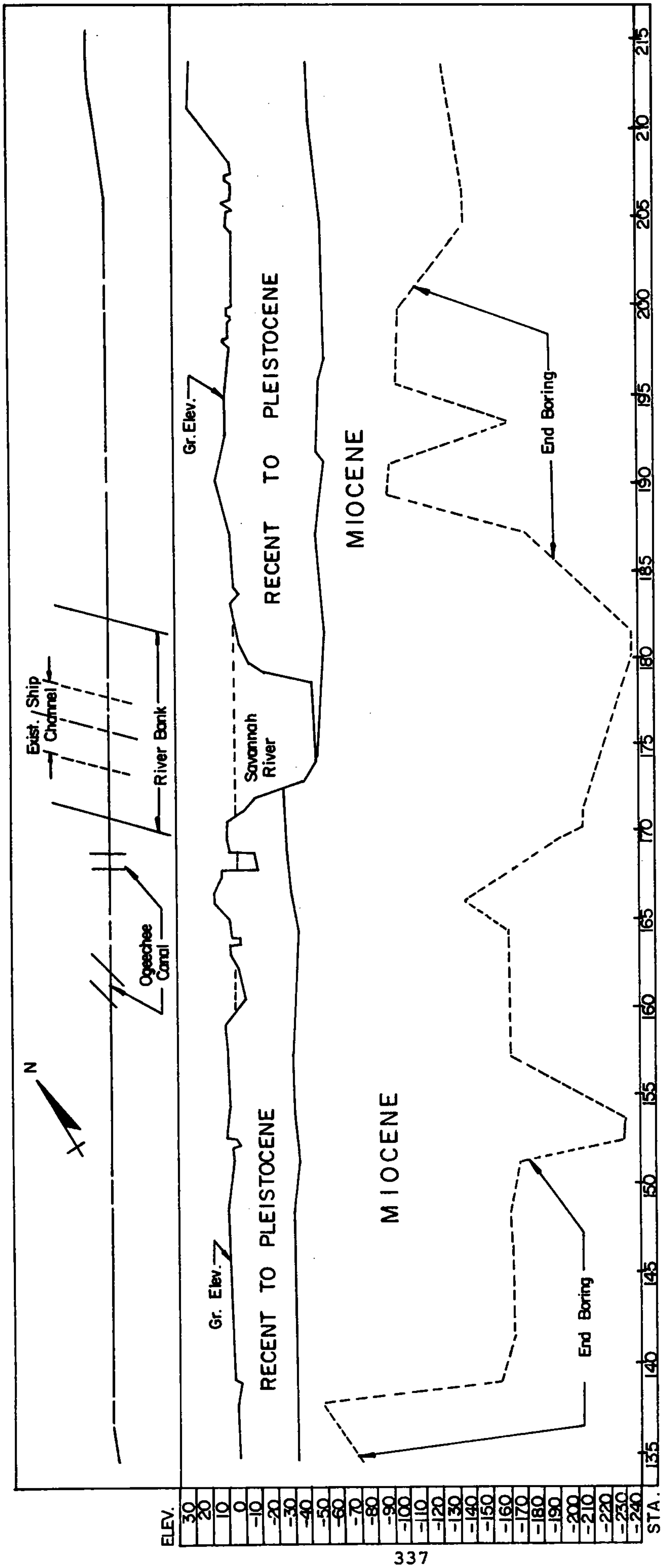
NEW TALMADGE MEMORIAL BRIDGE OVER SAVANNAH RIVER AT SAVANNAH GA.
THIS IS AN ARTIST CONCEPT - ACTUAL BRIDGE UNDER CONSTRUCTION
ESTIMATED COMPLETION DATE 1990



MAIN TOWERS = "H" PILE FOOTINGS
ANCHOR BENTS = DRILLED SHAFTS
APPROACH BENTS = PRESTRESSED PILES

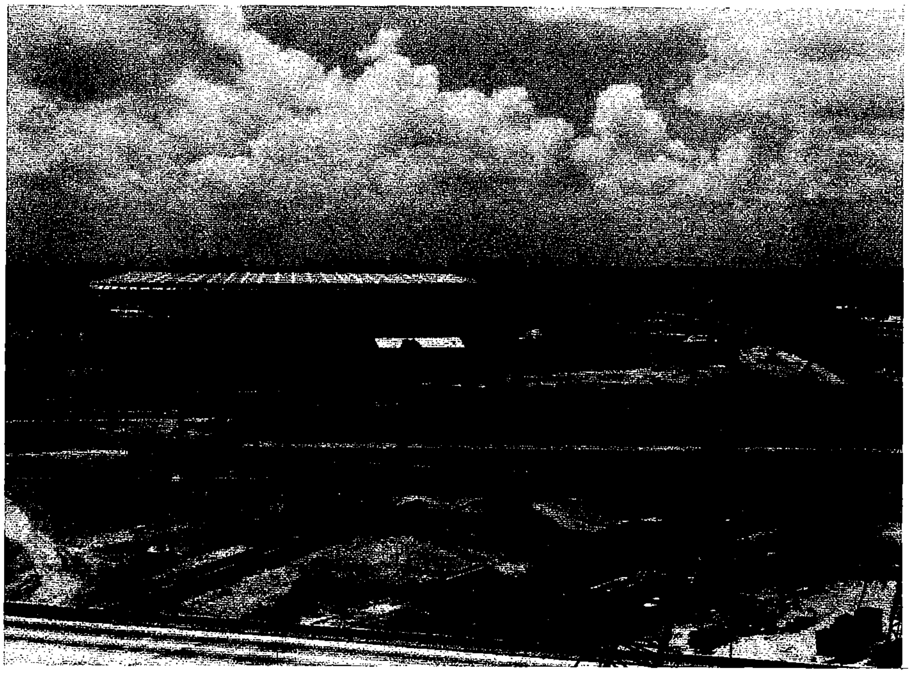
David Mitchell
- Page 23 -

GEOLOGIC PROFILE

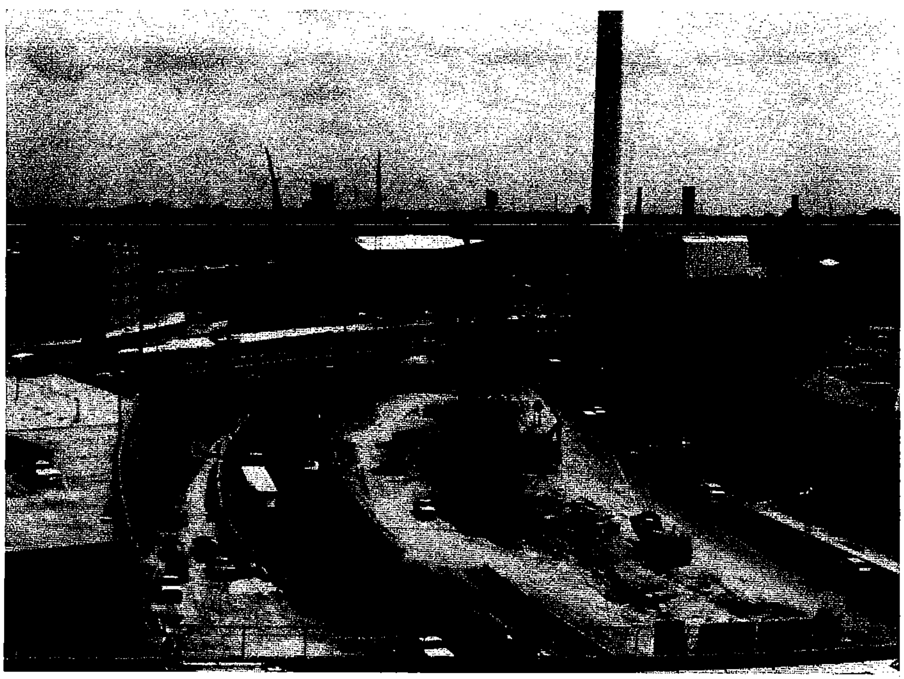


NEW US 17 BRIDGE O/ SAVANNAH RIVER
SAVANNAH, GA.

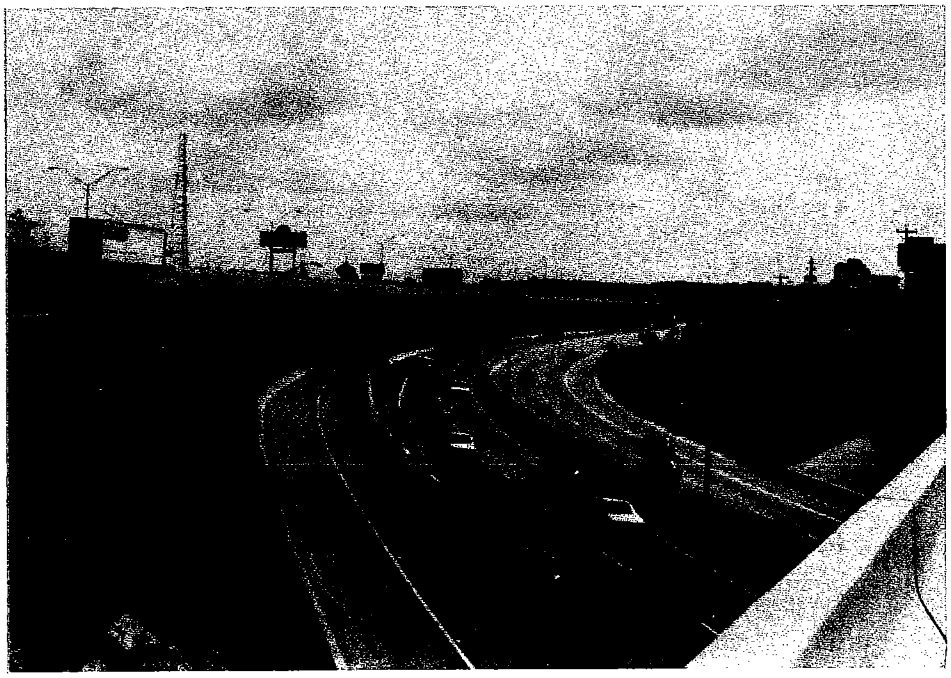
I-75 SOUTH ATLANTA AT I-20 INTERCHANGE
NEAR FULTON CO. STADIUM - ALL BRIDGES ON
STEEL "H" PILE FOOTINGS AND SPREAD FOOTINGS



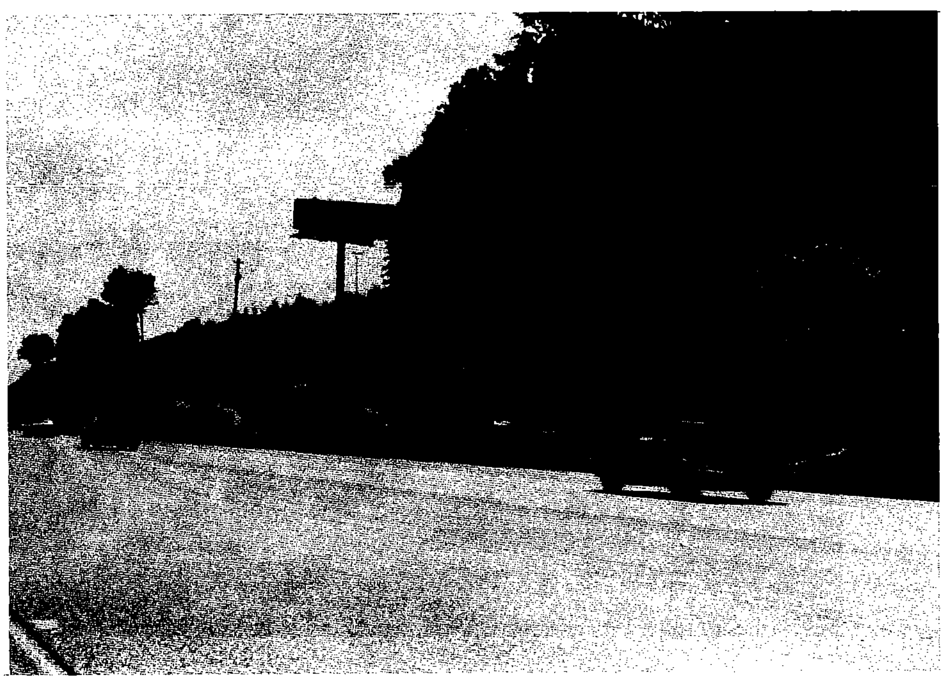
I-75 ADJACENT TO CITY OF ATLANTA
ALL BRIDGES ON "H" PILES & SPREAD FTGS.
WALL AT RIGHT IS LONG TIED BACK WALL



I-85 NORTH OF DOWNTOWN ATLANTA
MILE LONG VIADUCT BRIDGE
SUPPORTED ON SPREAD FOOTINGS & SHORT "H" PILES

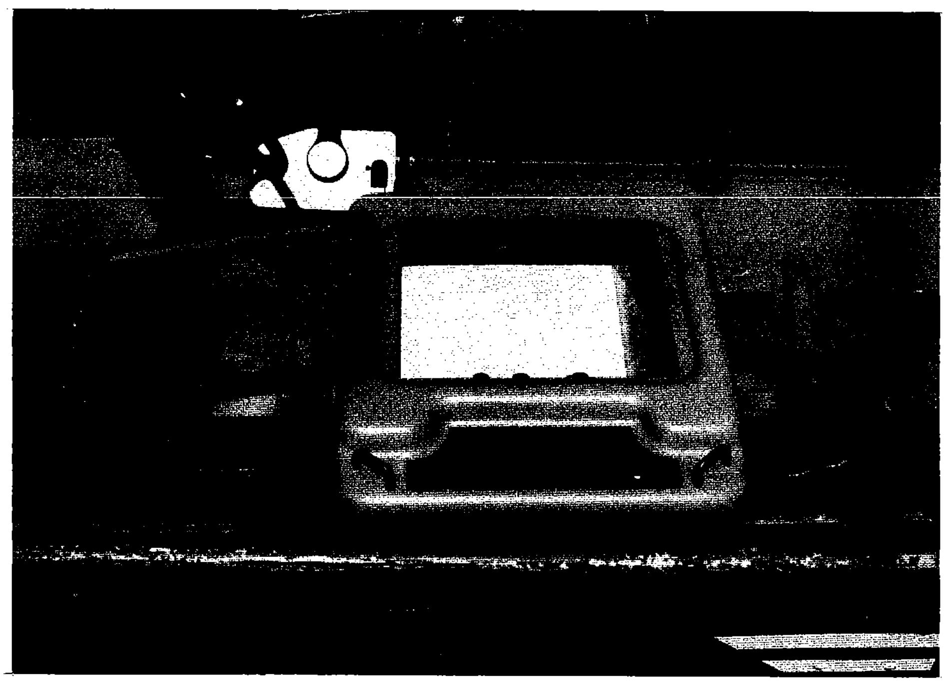


I-85 NORTH OF DOWNTOWN ATLANTA
WALL PLACED ON SPREAD FOOTINGS ON
IRREGULAR ROCK SURFACE



GENE BURDEN GEOLOGIST IN CHARGE
OF FATHOMETER PROFILES & SCOUR STUDIES
FOR ALL MAJOR STREAM CROSSINGS

PORTABLE RAYTHEON MODEL DE-719
SURVEY FATHOMETER - CHART RECORDER WITH
MARKER BUTTON AND OPEN FACE FOR PENCIL NOTES



REFERENCES

1) Dr. George F. Sowers, Georgia Institute of Technology. Direct contact for many years. Textbooks, various publications.

2) Mr. Thomas D. Moreland, Former Commissioner of Georgia D.O.T. primarily responsible for establishing Georgia D.O.T. Geotechnical Group. Former Chief Geotechnical Engineer.

3) Dr. Lymon C. Reese, University of Texas
August, 1977 Drill Shaft School for Georgia D.O.T. various publications.

4) Deep Foundation Institute, various publications
Atlanta Seminars.

5) Federal Highway Administration, Region Four, Georgia Division, various schools, contact with Bridge and Geotechnical Engineers.

CONSTRUCTION OF TIED BACK SOLDIER PILE ROCK RETAINING WALL ALONG I-90 IN NORTHER IDAHO, A CASE HISTORY

W. Capaul, D. Wyllie*, R. Dunsmore, R. Smith, J. Winger,
A. Paroni, J. Draeger, J. Perfect;
Idaho Transportation Department
*Golder Associates, Inc.

In May, 1987, an $8 million dollar contract was awarded by the Idaho Transportation Department to Harcon-Reid Burton Construction, a joint venture, to construct 3 tied back soldier pile retaining walls, 8 mechanically stabilized embankment (MSE) retaining walls, and to complete miscellaneous grading. This work would create a corridor sufficient to allow construction of an elevated viaduct structure which would carry Interstate 90 traffic through the narrow Silver Valley of North Idaho.

The restricted area into which the viaduct could be constructed led to evaluating several alternate rock slope stabilization methods which would lessen the impact on the City of Wallace and the adjacent rocky hillsides.

Early in the design phase, ITD retained the services of Golder Associates, Inc. to provide various designs for stabilizing the adjacent rock slopes and to assist in problem solving during construction.

A tied back soldier pile wall design was chosen to be the most effective method to minimize the areal extent and lessen the visual impact of the rock cuts while producing confidently stable slopes. This design would also minimize the possibility of rock fall damage to the $30 million dollar I-90 viaduct.

To date, all three soldier pile walls are inplace and excavated to bench grade. The walls vary in height from a few feet to over 50 feet and are as long as 765 feet. The three walls will contain approximately 24,000 lineal feet of 120 and 250 kip design load strand anchors; 11,300 lineal feet of 25 kip design load rock dowels and anchors; 5872 lineal feet of C15X33.9 double C-channel soldier piles; and 43,000 square feet of untreated 4"x10" fir lagging. All anchors are double corrosion protected. The lagging was designed to provide only temporary support until a 10" cast in-place concrete facing wall could be constructed.

The holes for the piles were drilled using a 24" diameter Ingersoll Rand Super Drill. This down-the-hole style drill was suspended from a Manitowoc 3900W, 140 ton boom crane.

The anchor holes were drilled by a variety of methods including airtrack, Klemm, modified ODEX, and a 6" DHH swung from a Manitowoc 4000, 150 ton boom crane, the latter proved to be the most efficient anchor drilling method on this site. Due to the variably weathered and highly fractured nature of the rock, the smaller drills had considerable difficulty maintaining a clean, straight, open hole for anchor insertion.

A performance specification required the bidding contractors to utilize their expertise to determine the anchor style, bond length, drilling method, and their own anchor placement and grouting methods. Of the first 29 anchors installed, 16 failed to achieve the design capacity. Subsequent modification of the contractors drilling and grouting methods has yielded all passing anchors.

The contract called for the construction of 8 MSE walls totalling over 82,000 square feet of face. The purpose of the retained earth wall is largely to limit the areal impact of the interstate on the already spacially limited valley thus providing area for future growth of the City of Wallace. They also were utilized to maintain both channel widths in areas near the South Fork of the Coeur d' Alene River and easement widths adjacent to the Union Pacific Rail branch line. The contractor chose to use the VSL Corporations's option for the MSE wall construction.

Construction on this project continued during the winter months and is moving towards a late summer completion date.

All phases of the construction of the soldier pile walls and the related learning curve for the ITD will be discussed in detail.

CONSTRUCTION OF A NEW INTERCHANGE FOR THE INDIANA TOLL ROAD, COMPLICATED BY POOR SOIL CONDITIONS AND PRESENCE OF SANITARY LANDFILLS, GARY, INDIANA

Terry R. West
Department of Earth & Atmospheric Sciences
Purdue University

New interchanges have been added to the Indiana Toll Road in the heavily urbanized area of northwest Indiana. The Cline Avenue interchange at milepost 10 on the Toll Road was opened to traffic in October, 1986. Located on the west side of Gary, in Lake County, the original toll road was completed in 1954.

A toll plaza and six exit/entrance ramps were added at Cline Avenue adjacent to the Gary Airport and the Grand Calumet River. Several landfills adjacent to the Toll Road complicated the new construction. Originally the toll plaza was to be located directly over an existing landfill owned by Gary Development Company. Following initial soil borings, the toll plaza was relocated to the east, beyond the landfill, requiring longer ramps and a greater fill thickness.

In undisturbed areas the subsurface consists of about 15 feet of wind deposited fine sand, 35 feet of gray beach sand, 35 to 55 feet of glacial clay, 2 to 10 feet of stiff glacial till, with Racine Dolomite of Silurian age as the bedrock. In some locations the sand had been removed for borrow during the 1954 toll road construction, and has since been backfilled with industrial waste and blast furnace slag.

The 1954 toll road was built over the previous channel of the Grand Calumet River, with the River relocated directly to the north. Muck was removed from the previous river channel and replaced with sand fill. Adjacent to the existing toll road, areas of muck about 10 feet thick had to be removed during construction of the new ramp system.

Settlement of compressible layers below the new embankments was another construction problem. Excavation of muck, placement of bridge supports on piles and construction of retaining structures also complicated road construction. Contaminated soil from the industrial landfills also had to be removed during construction of the new interchange.

DESIGN AND CONSTRUCTION METHODOLOGY FOR ROCK CUTS IN GLENWOOD CANYON

Roger Pihl, Engineering Geologist, Colorado Geological Survey
Tim Bowen, Senior Engineering Geologist, Colorado Geological Survey

The Colorado Geological Survey and Colorado Department of Highways have evolved an informal methodology for designing and constructing safe, visually pleasing rock cuts on the Glenwood Canyon Interstate 70 project in Western Colorado. Philosophically, the goal is to maintain involvement of project engineer, geologist, and landscape architect on a regular basis.

The methodology considers site specific geological conditions and their relationships to construction and performance parameters such as mass stability, rockfall potential, aesthetic values, and economics. Also considered ar landscaping concerns, which impose restrictive limitations on ditch configuration, backslope, access, rock reinforcement, and excavation methods. These limitations often conflict with the goal of achieving a stable, economical cut. Methods of preliminary investigation and planning which minimize the visual impact of the cut while producing a safe backslope are discussed. These methods include field mapping of rock structure, assessment of rock quality, and computer simulation of rockfall.

Contractor operations are significantly affected by owner directives. Preparation of plans and specifications must address such practical considerations as access to difficult locations, protection of nearby structures, job safety, and maintaining traffic flow. These consideration may influence the final cut configuration.

The methodology adopted by the Glenwood Canyon geotechnical staff has been implemented on four major rock excavations with progressively improving success. These excavations, in quartzite and granitic rock, are presented as separate case histories with emphasis on application of the methodology to site specific problems.

MODELING SHEAR STRENGTH AT LOW NORMAL STRESSES FOR ENHANCED ROCK SLOPE ENGINEERING

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University of Idaho, Moscow, Idaho 83843

ABSTRACT

The engineering design of slope configurations and slope reinforcement systems for highway rock-slope cuts requires estimation of the shear strength along geologic discontinuities, which will control potential failure paths and mechanisms. Because most highway cuts in rock are rarely higher than about 60 m and because geologic structures generally lack continuous lengths on this scale, the effective normal stresses acting on potential sliding surfaces will be less than about 40 tsm (8200 psf). In fact, for most cases these normal stresses are less than 10 tsm (2050 psf). Natural roughness and non-planarity of rock discontinuities induce a nonlinear shear strength envelope that can have a pronounced curvature at low normal stresses. This nonlinear character of rock shear strength has a major influence on slope stability investigations, especially when slope cuts are not very high and the resulting failure modes are small.

Several methods for modeling the nonlinear shear strength of rock discontinuities and for incorporating it into limit-equilibrium slope stability analyses are discussed. These generally can be divided into two groups: 1) those based on the JRC (joint roughness coefficient) used in conjunction with the basic friction angle and 2) those based on least-squares regression models of laboratory direct-shear results for natural discontinuities used in conjunction with a waviness angle measured in the field. Examples of slope stability computations for plane-shear and for three-dimensional wedge failure modes suggest that the safety factor based on JRC methods is very sensitive to the JRC rating. Also, a power regression model generally is preferable to a linear model for describing the shear strength of rock discontinuities. If not used with care when analyzing small potential failure masses, a linear model can result in calculated safety factor values that are unrealistically high.

INTRODUCTION

The concept of a nonlinear shear failure envelope for rock discontinuities, such as joints, bedding, and foliation, is not a new idea in the field of rock mechanics. Curved relationships between the shear strength and normal stress were noted some 20 years ago by a number of investigators, including Lane and Heck (1964), Patton (1966), and Jaeger (1971). Many rockfill materials also tend to have nonlinear shear failure envelopes (Barton and Kjaernsli, 1981). A curved model for rock discontinuity shear strength can predict values of shear strength that are much different than those predicted by a traditional linear model consisting of c (cohesion) and ϕ (friction angle), particularly for small values of normal stress on the order of 0.1 tsm to 20 tsm (20 psf to 4100 psf). Such values of applied normal stress are common to those experienced in highway rock-slope cuts, both for dry failure masses less than 10 m high found near the slope crest or in benches and for larger wet (i.e., groundwater pore pressures are present) failure masses that may involve an entire slope.

Most of the public-domain and commercially available computer software for conducting limit-equilibrium rock-slope stability analyses rely on a linear c, ϕ model of shear strength. Estimation of the input values for c and ϕ typically relies on laboratory direct-shear tests of 10 cm to 30 cm blocks that contain discontinuities or on pull/tilt tests conducted on larger blocks of rock in the field. Shear strength models based on laboratory tests are quite sensitive to the range of normal loads specified by the responsible technician or engineer. Ideally, these loads should subject the test specimens to normal stresses similar to those expected in the rock slope under study. If a fairly large range of stresses is anticipated, then the selected model for the shear failure envelope should be appropriate over the entire range.

Because limit-equilibrium stability analyses are used to compute factors of safety that lead to design recommendations, the effective normal stresses on the potential failure planes as computed via these stability analyses are really the key levels of normal stress for which the shear strength must be estimated. Thus, a preliminary stability assessment aimed at the identified failure modes (such as plane-shears, 3-D wedges, and step-paths) in the rock slope can provide valuable guidance to the design engineer for the shear-strength testing program. This means that computer programs used for slope stability evaluations should provide output of calculated values of "intermediate" variables, such as the weight and volume of the potential failure mass, the pore pressure, the effective normal stress acting on the sliding plane(s), and the corresponding shear strength.

THE JRC MODEL OF SHEAR STRENGTH

A curved shear strength model for rock discontinuities initially proposed by Barton (1973) relies on three terms besides the effective normal stress: 1) the joint roughness coefficient (JRC), 2) the joint-wall compressive strength (JCS), and 3) the basic friction angle. The JRC is a descriptive measure of surface roughness over lengths of 0.1 m to 10 m and has unitless values that range from 0 to 20. It can be estimated by visual matching of roughness profiles (ISRM, 1978, p. 345) or by measuring the amplitude of asperities on the joint surface (Barton, 1982). The JCS typically is estimated from the measured rebound of a standard Schmidt-L Hammer as prescribed by ISRM (1978, p. 347-350). The most reliable means to obtain a value for the basic friction angle is to conduct direct-shear tests of saw-cut planar surfaces made in rock specimens collected from the field site. Engineers sometimes will rely on a generic, "textbook" value of, say, 30 degrees and will not bother to conduct direct-shear tests. However, basic friction angles can vary widely among even similar rock types, so at least a minimal direct-shear testing program is advisable (results of recent rock shear testing conducted by the author yielded basic friction angles of 22, 29, and 34 degrees for quartzite, gneiss, and granite, respectively).

Barton (1973) relied upon a wide variety of experimental results to develop a formula for describing a curved shear-strength envelope that predicts discontinuity shear strength as a function of effective normal stress, JRC, JCS, and basic friction angle. This empirical model, which has been accepted widely in the rock mechanics community, can be expressed in the following form:

$$\tau = \sigma_n' \cdot \tan\left[(JRC) \cdot \log_{10}\left(\frac{JCS}{\sigma_n'}\right) + \phi_b \right] \quad (1)$$

where: τ = shear strength, σ'_n = effective normal stress, JRC = joint roughness coefficient, JCS = joint-wall compressive strength, and ϕ_b = basic friction angle.

Of the input terms, JRC generally has the greatest influence on shear strength at low normal stress values. The basic friction angle also is important, but its potential variability (i.e., range of values) is considerably smaller than that of the measured JRC. Figure 1 illustrates the sensitivity of predicted shear-strength envelopes to the JRC value. Therefore, care must be taken to obtain representative, unbiased JRC data/information.

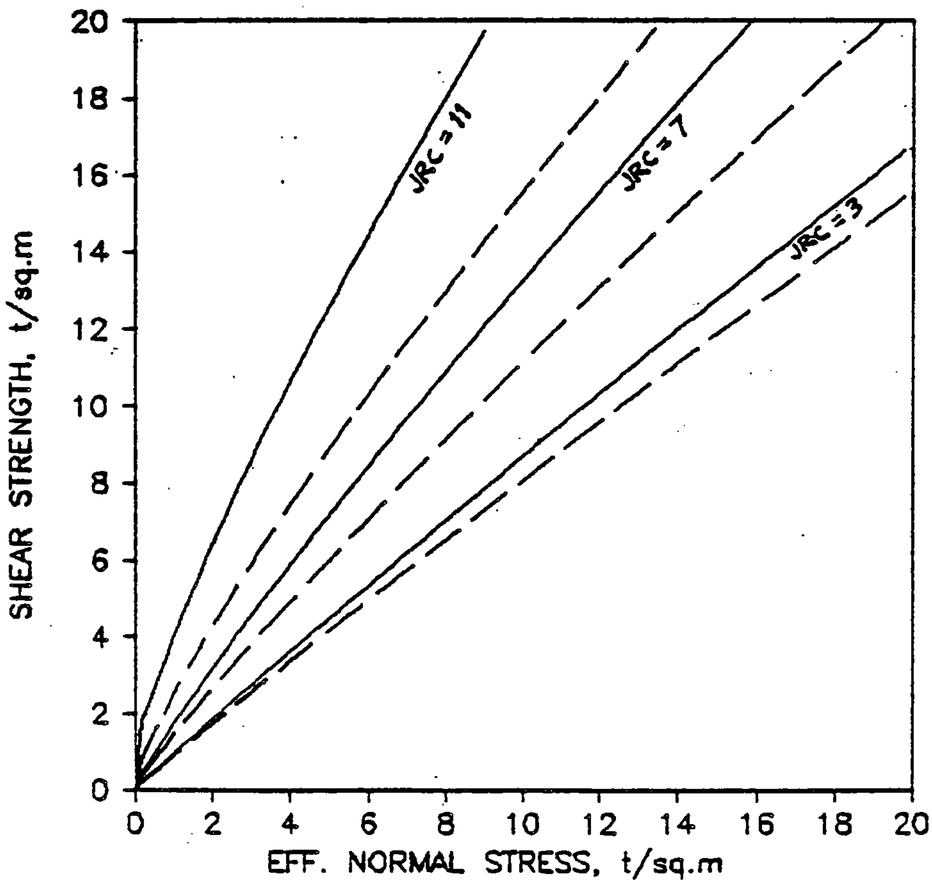

Figure 1. Examples of discontinuity shear-strength envelopes predicted by the JRC model for a given set of granite joints having JCS = 10000 tsm (as derived from a Schmidt-hammer rebound value of 42 and a rock density of 2.66 tcm) and ϕ_b = 32° (as derived from direct-shear tests of saw-cut surfaces); the dashed envelopes are for a JCS value of 2000 tsm.

In limit-equilibrium slope stability computations, once the effective normal stress acting on the sliding surface has been calculated, the corresponding shear strength (which contributes to the resisting force) can be obtained directly by using equation (1). Unfortunately, most of the slope-stability computer programs in use today require a c, ϕ linear model of shear strength. A simple modification to such computer codes will allow them to handle the nonlinear JRC shear-strength model via the following expressions that relate c and ϕ to the JRC-predicted shear strength (Kirsten and Moss, 1985, p. 117):

$$c = \frac{\pi}{180} \cdot \sigma_n' \cdot \mathrm{JRC} \cdot \left[1 + \left(\frac{\tau}{\sigma_n'} \right)^2 \right] \cdot \log_{10} e \tag{2}$$

$$\phi = \arctan\left(\frac{\tau}{\sigma_n'} - \frac{c}{\sigma_n'} \right) \tag{3}$$

where: c = cohesion, ϕ = friction angle, and the other terms are as previously defined. Thus, a computer subroutine based on equations (1), (2), and (3) can be incorporated into traditional rock-slope stability programs/codes to produce the necessary c and ϕ values.

SHEAR-STRENGTH MODEL BASED ON PSUEDO-RESIDUAL SHEAR TESTS

Laboratory direct-shear tests of natural rock discontinuities often will provide curved shear-strength envelopes for both the peak strengths and for the psuedo-residual strengths. The term "psuedo-residual" is appropriate in this case, because most investigators doubt whether a true residual strength can be obtained during small-scale shear tests due to dilation that persists even for quite large displacements. The data obtained from such shear tests practically always can be fitted by a power model better than by a linear model when least-squares regression is applied (see the example given in Figure 2). A general power-curve model for the shear-strength envelope is given as: $\tau = a\,(\sigma_n)^b + c$ (Miller and Borgman, 1984), which provides adequate flexibility to handle cohesionless behavior (c = 0) and a linear failure envelope (for b = 1, then a = tanϕ and c = cohesion). Knowledge of scale effects and of pertinent results of back-analyses of rock-slope failures makes it prudent to use the psuedo-residual shear data rather than the peak shear data from small-scale laboratory direct-shear tests.

The power-curve failure envelope obtained from such laboratory shear tests of natural joints is an appropriate model of shear strength on the testing scale (typically 10 to 30 cm) and will incorporate the effects of asperity roughness and joint-wall compressive strength. The effects of undulations/waviness observed over distances of about 1 to 10 m will not be taken into account by the power-curve model. One way to remedy this shortcoming when conducting a limit-equilibrium slope stability analysis is to incorporate this larger-scale waviness into the calculated resisting force (i.e., as an additive term to the shear strength in the numerator of the safety factor ratio: FS = Resisting Force / Driving Force). This additional resistance should depend on the severity of the waviness and on the effective normal stress acting on the overall sliding surface.

One approach to quantifying this waviness is to measure the average and minimum dips along the rock discontinuity(ies) of interest as part of the field data collection exercise

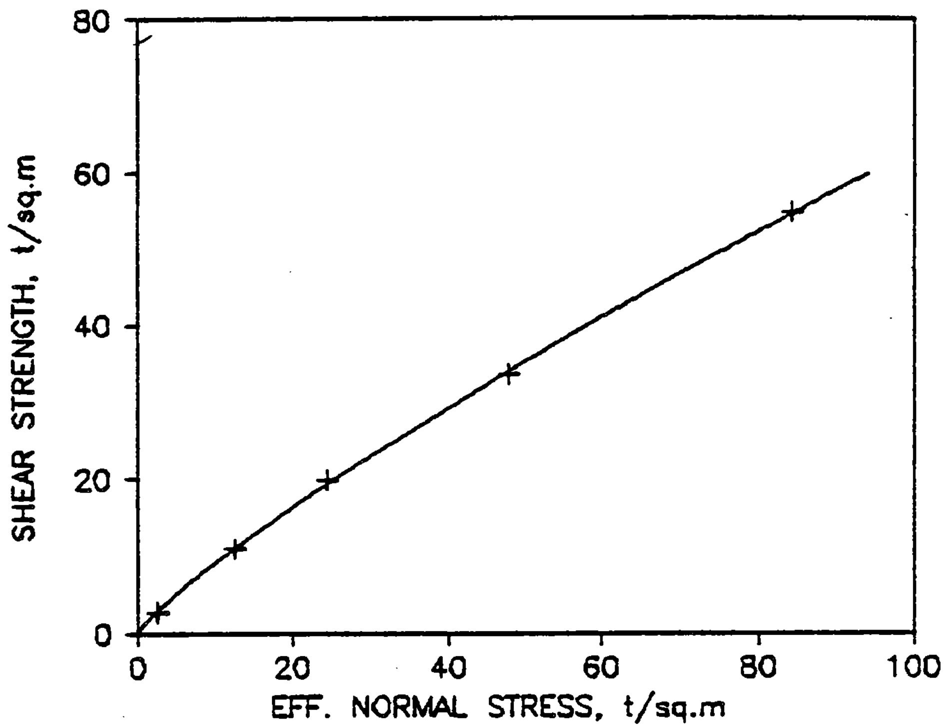

Figure 2. Example of a power-curve failure envelope fitted to direct-shear test results for a natural joint in granite; $\tau = 0.017 + 1.340\ \sigma_n^{(0.836)}$ (in tsm units).

(Call and others, 1976). An estimate of the waviness then is given by: waviness = [average dip] - [minimum dip], and is expressed in units of degrees. The average dip of the sliding surface is used in slope stability analyses to calculate the volume of the potential failure mass (which leads to subsequent determinations of the weight and effective normal stress) and to resolve forces that act on the failure mass. If the discontinuous rock mass were to slide along the identified failure surface, then individual rock blocks with sizes defined by the joint spacings would have to slide along portions of the failure surface having a dip equal to the average dip minus the waviness. Thus, the greater the waviness, the greater the resistance to sliding that is experienced by the potential failure mass.

Based on these considerations, some engineers (among the first was R.D. Call, now with Call & Nicholas, Inc., Tucson, Arizona) rely on a simple means to incorporate waviness into slope stability calculations, as illustrated in Figure 3. The contribution of discontinuity waviness to the resisting force is equal to the effective normal force times the tangent of the waviness angle. This waviness term adds a "large-scale" component to the shearing resistance along a potential failure surface, and when used in conjunction with a curved shear-failure envelope obtained from direct-shear tests of natural discontinuities (which consider small-scale roughness and joint-wall strength), a reasonable mechanism for

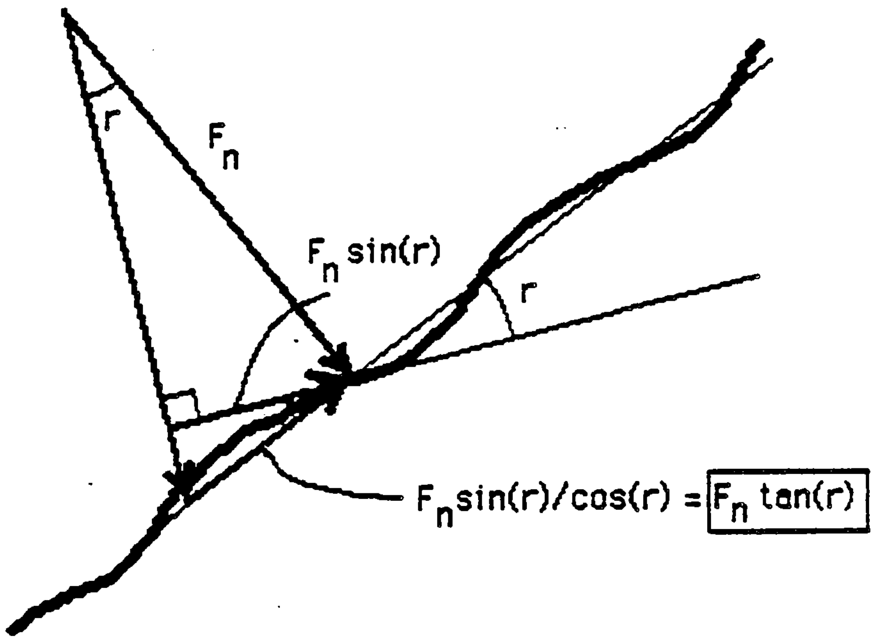

Figure 3. Contribution of the waviness to the resisting force in a rock-slope stability analysis.

incorporating discontinuity shear strength into rock-slope design is possible. One advantage of this method is that waviness is much easier and faster to measure in the field than are the types of data associated with JRC.

When using this model of discontinuity shear-strength, the design engineer should beware of applying a linear c, ϕ failure envelope to the psuedo-residual shear data provided by a laboratory testing program. A linear model may seem appropriate for a large range of normal stresses (and may suffice for values exceeding 30 tsm for most rock types), but such is not the case for many natural discontinuity surfaces subjected to low values of normal stress. As an example, the five shear data presented in Figure 2 can be fitted by a linear model to yield the following expression: $\tau = 2.978 + 0.624\sigma_n$ (envelope 3 in Figure 4), which provides a considerable overestimation of shear strength at normal stresses less than 7 tsm. A more appropriate linear model for slope stability investigations would be based on the three data points having normal stresses less than 30 tsm; this gives $\tau = .938 + 0.783\sigma_n$ (envelope 2 in Figure 4).

EXAMPLES OF SHEAR-STRENGTH INFLUENCE ON CALCULATED SAFETY FACTOR

To illustrate the influence that the selected shear-strength model exerts on the slope design process, safety-factors have been computed for realistic plane-shear and wedge failure modes using the shear strength of granite joints as presented in Figures 1 and 2. The

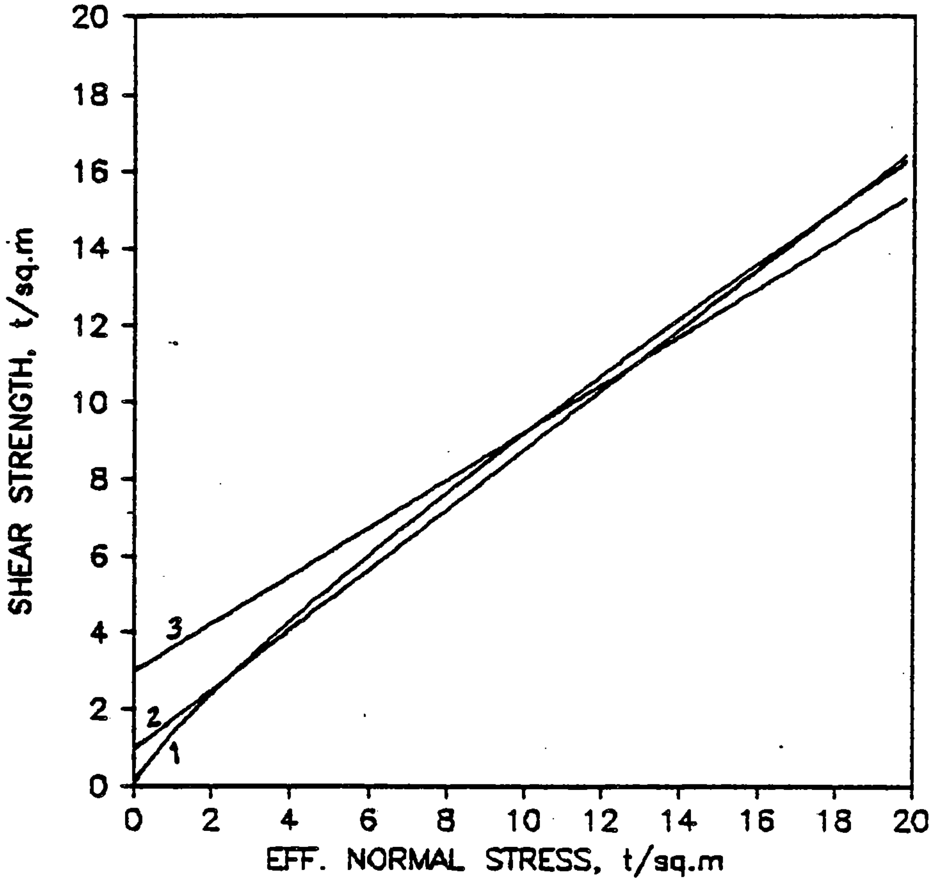

Figure 4. Comparisons of shear-strength envelopes (fitted to data given in Fig. 2) at low values of normal stress; curve 1 is general power model, curve 2 is linear model fitted to three smallest data, curve 3 is linear model fitted to all five data.

results are based on dry conditions and slope cuts of 0.5:1 (i.e., 64°) made in a natural hillslope with a gradient of 4:1 (i.e., a dip of 14°). The failure height in all cases is defined as the vertical distance from the slope crest to the daylight point of the sliding surface in the slope-cut face. Tables 1 and 2 summarize the results of the safety factor calculations. Case B for the plane-shear modes is plotted in Figure 5, and Case A for the 3-D wedge modes is plotted in Figure 6.

These example calculations illustrate the importance of adequately modeling discontinuity shear strength during a rock-slope design project. For the low normal stresses typical to such investigations a simple reliance on traditional c, ϕ shear-strength models can produce unrealistically high safety factor values in a limit-equilibrium slope stability analysis. The problem is even more apparent when groundwater-induced pore pressures are present in the rock slope, because they will reduce further the effective normal stress acting on potential sliding surfaces.

Table 1. Safety factor values computed for example plane-shear failure modes with potential sliding planes dipping at 35° and 50°.

	Failure Height (m)	1. Power		2. Linear2		3. Linear3		4. JRC-Model	
Case A:	30	1.27	0.87	1.27	0.82	1.21	0.93	1.21	0.74
JRC = 3	15	1.42	0.97	1.35	0.93	1.45	1.29	1.25	0.76
Wav. = 3°	6	1.64	1.12	1.57	1.27	2.17	2.38	1.30	0.80
	3	1.83	1.26	1.95	1.84	3.38	4.19	1.34	0.82
Case B:	30	1.47	0.98	1.47	0.93	1.41	1.05	1.78	1.16
JRC = 7	15	1.62	1.09	1.55	1.05	1.65	1.41	1.92	1.26
Wav. = 11°	6	1.84	1.24	1.78	1.39	2.38	2.50	2.13	1.40
	3	2.04	1.38	2.16	1.96	3.58	4.31	2.31	1.52
Case C:	30	1.72	1.13	1.71	1.08	1.65	1.19	2.72	1.96
JRC = 11	15	1.86	1.23	1.79	1.19	1.89	1.55	3.15	2.32
Wav. = 20°	6	2.08	1.38	2.02	1.53	2.62	2.64	3.92	3.02
	3	2.28	1.52	2.40	2.10	3.82	4.45	4.76	3.87

(Safety Factor Values)

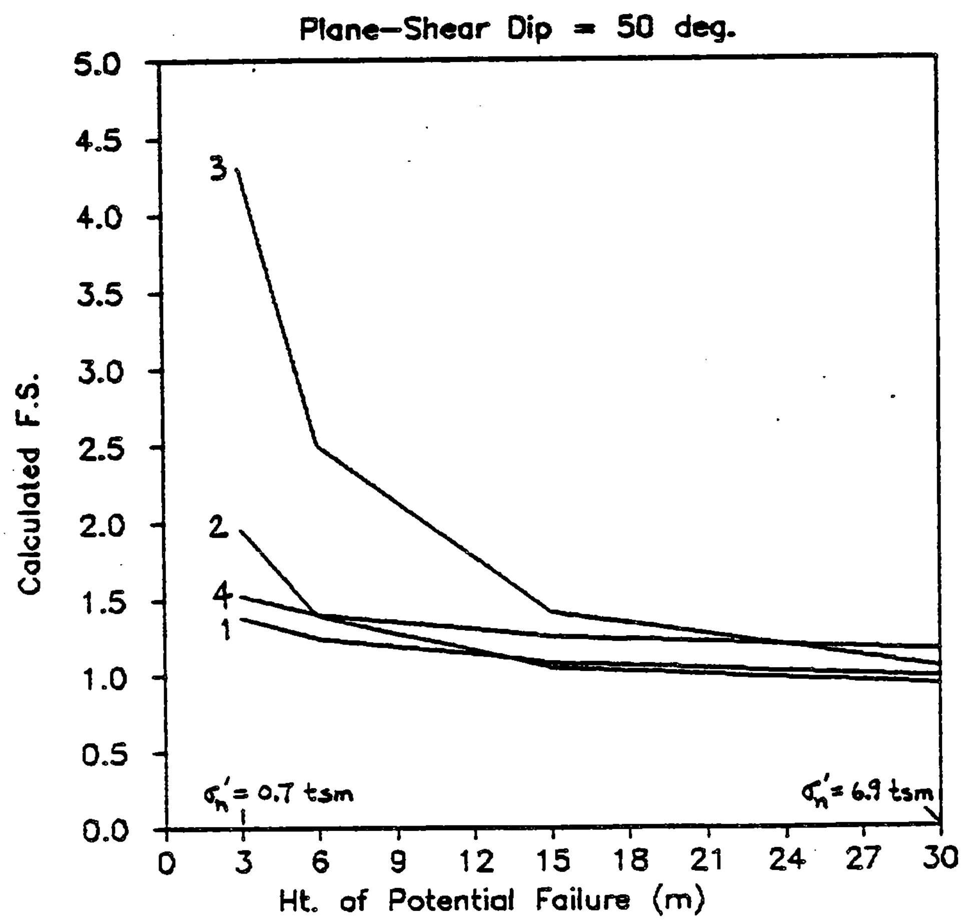

Figure 5. Factor of safety values for 50° plane-shear Case B.

Table 2. Safety factor values computed for example 3-D wedge failure modes for a slope cut with dip dir. = 300°, dip = 64°; left plane dip dir. = 255°, dip = 57°; right plane dip dir. = 330°, dip = 48°. The bearing and plunge of the wedge intersection are 304° and 45°, respectively.

	Failure Height (m)	Safety Factor Values: 1. Power	2. Linear2	3. Linear3	4. JRC-Model
Case A:	30	1.164	1.097	1.257	0.995
JRC = 3	15	1.299	1.253	1.753	1.027
Wav. = 3°	6	1.508	1.721	3.240	1.071
	3	1.695	2.501	5.718	1.105
Case B:	30	1.321	1.253	1.414	1.558
JRC = 7	15	1.456	1.409	1.910	1.685
Wav. = 11°	6	1.665	1.878	3.396	1.877
	3	1.851	2.658	5.874	2.044
Case C:	30	1.497	1.429	1.590	2.632
JRC = 11	15	1.632	1.585	2.085	3.119
Wav. = 20°	6	1.840	2.053	3.572	4.070
	3	2.027	2.834	6.050	5.231

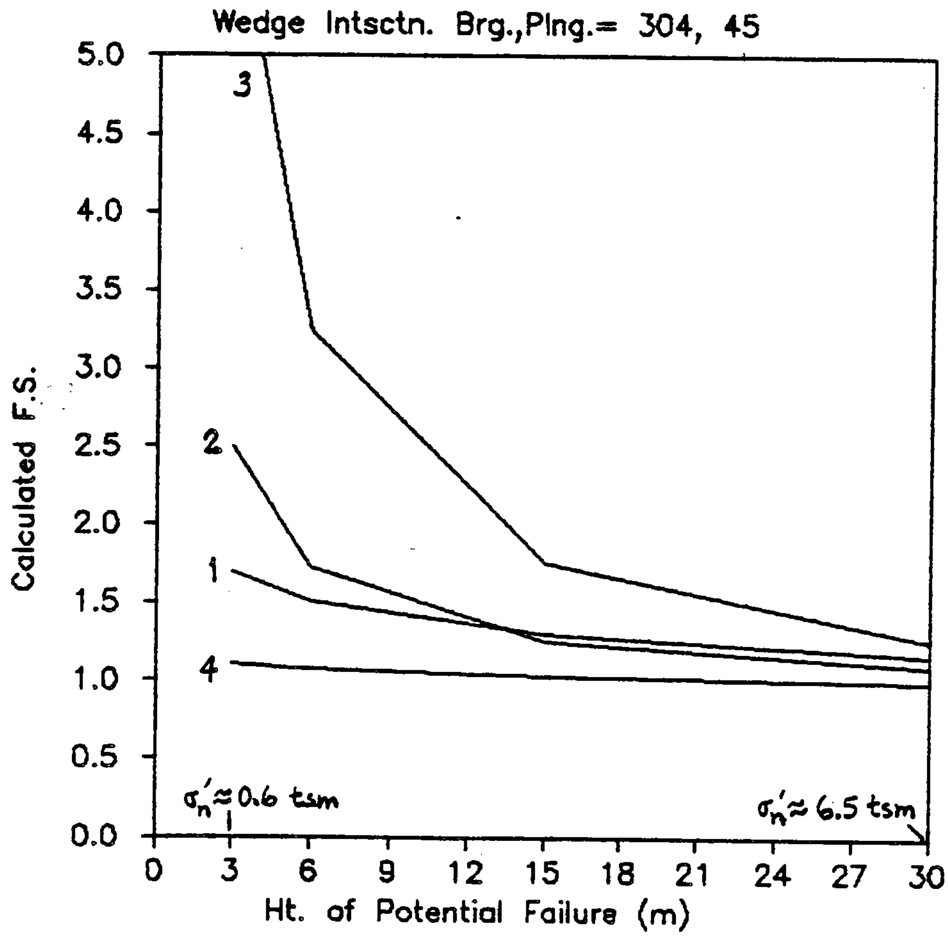

Figure 6. Factor of safety values for 3-D wedge Case A.

CONCLUSIONS

When limit-equilibrium slope stability analyses are used in the design of highway rock-slope configurations or support systems (such as rock bolts, cable tendons, tie-back walls, etc.), the geotechnical engineer must exercise care in estimating the shear strength along rock discontinuities. It is especially critical to model the shear-strength envelope at very low normal stresses (less than 20 tsm), because potential sliding surfaces in road cuts will be subjected to such stresses. The nonlinear JRC Model of shear strength generally is preferred to a traditional c, ϕ linear model, but it predicts shear strengths that are highly sensitive to the estimated value of JRC (joint roughness coefficient). This sensitivity may lead to overly conservative slope designs if the JRC is underestimated and to undesirable nonconservative designs if the JRC is overestimated.

An alternate shear-strength model is based on psuedo-residual shear strengths obtained from laboratory direct-shear tests of natural discontinuities and a waviness angle measured in the field. A general power-curve failure envelope can be fitted to the laboratory shear data and almost always provides a better estimator of shear strength at low normal stresses than does a traditional linear model. The larger-scale waviness effects are added in to the resisting force during the safety-factor calculations in the stability analysis. Experience with this design approach and the results of example calculations of safety factors indicate that a linear model can be appropriate over small ranges of normal stress, but that a power-curve model will provide more reasonable estimates of shear strength in the general case.

A preliminary slope stability assessment of potential failure modes (such as plane-shears, 3-D wedges, and step-paths) can provide valuable guidance to the design engineer for the shear-strength evaluation program. This assessment can be enhanced significantly by computer programs that provide output of calculated values of "intermediate" variables, such as the weight and volume of the potential failure mass, the pore pressure, the effective normal stress acting on the sliding plane(s), and the corresponding shear strength. In particular, knowledge of the pertinent levels of normal stress anticipated in the rock cut is essential to a valid estimation of discontinuity shear strength, and thus, to a prudent rock slope design.

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SPIRIT LAKE MEMORIAL HIGHWAY - GEOLOGIC INVESTIGATIONS IN A ZONE OF NATURAL AESTHETIC CHANGE

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ABSTRACT

On May 18, 1980 mass wasting events related to the eruption of Mount St. Helens destroyed over 20 miles of Washington State Route 504 located in the valley bottom of the North Fork Toutle River. Reconstruction of the highway is proposed along the north slope of the valley, well above the level of the 1980 debris avalanche and mudflows. The lateral blast associated with the eruption transformed a heavily forested area into a zone of barren slopes with accumulations of blown-down timber and tephra. This natural change in aesthetic character has changed the nature of the geologic hazards to the highway in this area. Increased susceptibility to landslides, snow avalanches, and erosion resulted from the devastation of an area of over 230 square miles. Surficial deposits where the alignment crosses the blast zone consist of tephra, glacial, and various types of mass wasting deposits. Bedrock units include andesite, basalt, gabbro, and a variety of pyroclastic rocks.

INTRODUCTION

About 20 seconds after 8:32 on the morning of May 18, 1980, apparently in response to a magnitude 5.1 earthquake, the bulging north side of Mount St. Helens collapsed, beginning a train of events which killed 60 people and left an area of 230 square miles with blown-down timber. Debris avalanche, mudflow, and tephra deposits which accumulated in the North Fork Toutle River valley covered over 20 miles of Washington State Route 504 with up to 640 feet of debris.

On behalf of Washington State Department of Transportation (WSDOT), Golder Associates Inc. conducted a geotechnical engineering investigation of a 17-mile portion of the new Spirit Lake Memorial Highway (SR-504) from Hoffstadt Mountain to Coldwater Lake (Figure 1). Investigations included drilling 382 boreholes totaling 17,194 feet and excavating 205 test pits along

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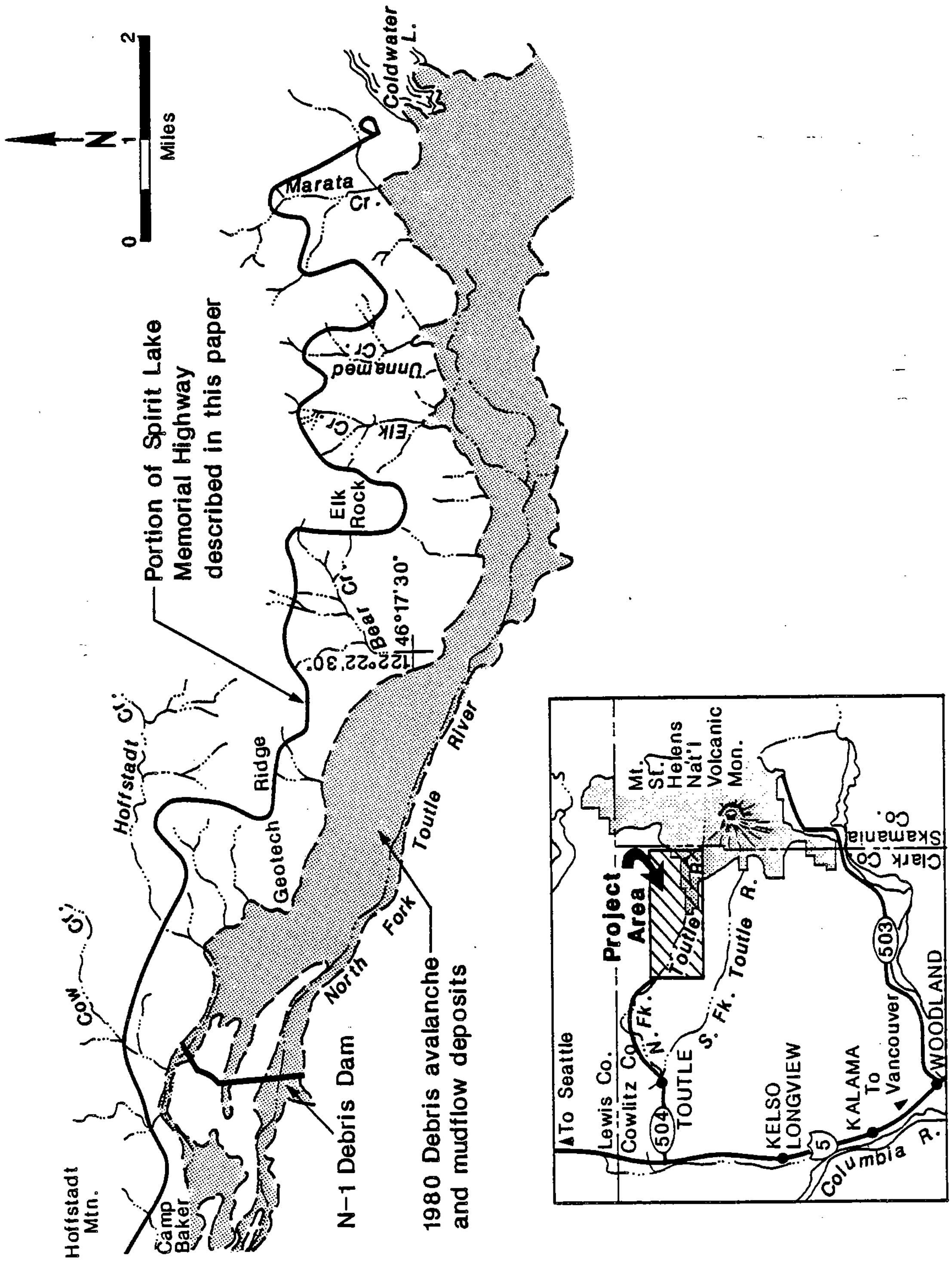


Figure 1
Location Map

the alignment and at seven bridge locations and two borrow areas during a 108-day period in the winter and spring of 1987. Reconstruction of the highway is proposed along the north slope of the valley, well above the level of the 1980 debris avalanche. This investigation was intended to provide sufficient information on which to base plans, specifications, and estimates for final roadway design.

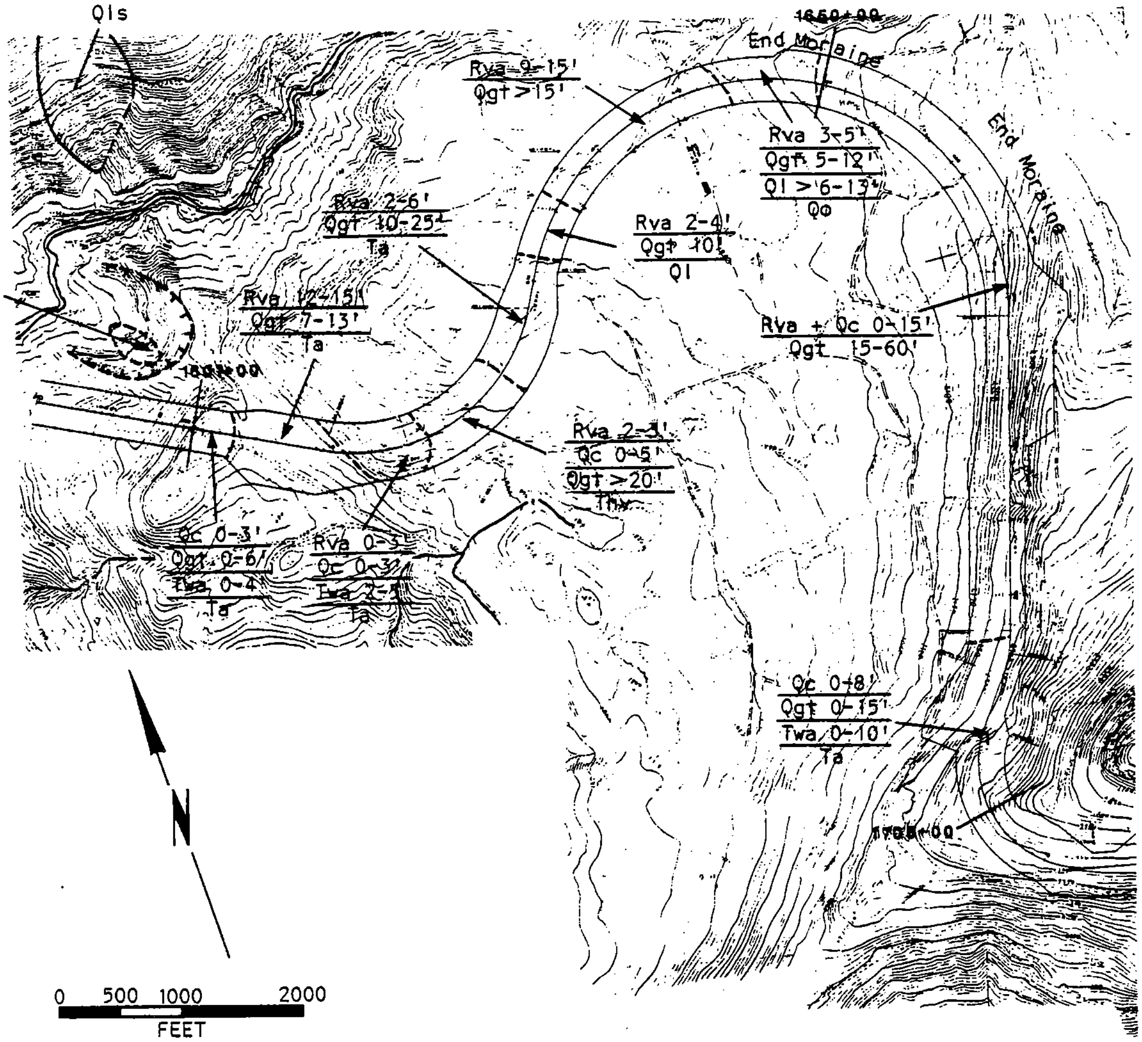
Highway alignment investigations frequently call for designs and construction techniques intended to minimize environmental impact. This project is no exception, however, the reason for building this highway is to access an area of major natural environmental impact. Without the deforestation caused by the eruption, and the logging roads built to remove the downed timber, geologic investigations in this area would have been substantially more difficult. This paper focuses on the general geology of the Mount St. Helens area and specific issues relating to engineering geologic work in a zone of major natural aesthetic change.

METHODOLOGY

The engineering geology investigation for this project consisted of review of previous reports, maps, and technical publications, interpretation of aerial photographs and images, reconnaissance geologic mapping, a primary drilling phase, seismic refraction work, a secondary drilling and test pit excavation phase, and additional geologic investigations at selected sites. The alignment was divided into six segments based on WSDOT construction requirements, and separate reports were issued for each segment. In addition, separate investigation reports were prepared for seven bridges along the route.

Interpretation of vertical aerial photographs was conducted using color photographs, at a scale of approximately 1:24,000, taken May 17, 1985 by WSDOT. Image analysis was conducted using false-color infrared images, at a scale of approximately 1:63,000, taken August 6, 1981 as part of the U.S. Geological Survey's National High Altitude Program (NHAP). The results of these analyses were plotted on mylar overlays to the photographs and used as a starting point for field mapping. Black and white vertical aerial photographs, at a scale of approximately 1:45,000, taken July 28, 1952 by the U.S. Geological Survey, were used for comparison purposes to evaluate any changes in the distribution of surficial deposits over the last 35 years.

Geologic mapping was conducted with the WSDOT color aerial photographs, enlarged to a scale of approximately 1:12,000. Selected terrain-unit mapping techniques (Kreig and Reger, 1976; Vita, 1984) were used for all geologic mapping. These techniques were chosen because they emphasize the stratigraphy of the surficial geologic units and, unlike traditional surficial and bedrock geologic mapping, permitted inclusion of the borehole data on the alignment plans (Figure 2).



Rva	Recent Volcanic Ash	Qo	Quaternary Glacial Outwash
Qc	Quaternary Colluvium	Twa	Tertiary Weathered Andesite
Qls	Quaternary Landslide Debris	Ta	Tertiary Andesite
Qgt	Quaternary Glacial Till	Thv	Tertiary Hydrothermally Altered Volcanic Rock
Ql	Quaternary Lacustrine Deposits		

Contact (Dashed where approximate)

Landslide Scar

Rva 2-4' / Qgt 10-25' / Ta — Geologic terrain units along the alignment

Read as: Recent volcanic ash approximately 2' to 4' thick overlies Quaternary glacial till approximately 10' to 25' thick which overlies Tertiary andesite.

Figure 2
Geologic Map of a Portion of the SR-504 Alignment

Single standpipe piezometers were installed in approximately 250 boreholes and nested piezometers were installed at six locations to more definitively monitor groundwater conditions. Drilling was conducted in two phases so that additional holes could be drilled if the results of the initial subsurface investigations suggested that further exploration was warranted. To investigate subsurface conditions in cut areas, and to better define the landslide deposits at the base of Hoffstadt Mountain, 18,704 feet of seismic refraction data were collected.

Subsurface investigations were undertaken using four track-mounted drill rigs and one excavator. Drilling and sampling techniques were consistent with American Society for Testing and Materials (ASTM) standards D1452, D1586, D1587, D2113, and D3550 (ASTM, 1985). Sampling in the unconsolidated deposits was accomplished by drilling casing, washing, and then driving either standard split-spoon samplers or larger diameter California samplers using both standard penetration test (SPT) methods and modified heavy hammer methods alternately. In rock and indurated deposits, diamond core drilling using a double-tube system with a split inner barrel was used to obtain core samples. Test pits were dug with a track-mounted excavator. Rig geologists or engineers were present to log all boreholes and test pits. Logging was accomplished using Golder Associates standard procedures for soil descriptions (Golder Associates, 1987) and a standard method for rock descriptions, which is based on the International Society for Rock Mechanics (ISRM) Suggested Methods (Brown, 1981). All soil samples and core were independently reviewed in a field laboratory for consistency of description prior to production of final borehole logs. Because of the large number of boreholes and the necessity of on-site engineering analysis concurrent with the investigation program, final borehole logs were produced at the project site using Golder Associates Inc. microcomputer borehole logging program. Data entered into the logging program was linked to a data base management program to facilitate data retrieval and analysis.

Golder personnel and subcontractors had offices and living facilities in a trailer camp established for this project at the former site of Weyerhaeuser's Camp Baker (Figure 1).

REGIONAL GEOLOGY

During the Quaternary this area was subjected to extensive alpine glaciation, as well as volcanic activity, and glacial, periglacial, and tephra deposits left by these geologic processes have mantled the older volcanic rocks. After deglaciation, hillslope development has occurred by fluvial, colluvial, and mass movement processes. On the steeper slopes, mass movement processes have been particularly important.

The Pacific Northwest is a zone of convergence between the Juan de Fuca and the North American tectonic plates. Subduction of sea-floor materials beneath the North American continental plate and subsequent partial melting of these rocks at depth, forms the magma necessary for the eruptions that have produced the volcanic peaks in the Cascade Range. Igneous activity in the southern Washington Cascades spans at least the last 35 million years (Fiske et al, 1963), and recent activity is exemplified by the 1980 eruptions of Mount St. Helens.

Surficial Geology

Limited investigations of the glacial and volcanic history of this area have been undertaken to describe the distribution and stratigraphy of the surficial deposits. Based on an analysis of the geomorphology and the physical properties of the surficial deposits in the Mount St. Helens area, three major glacial advances are recognized. During the last major glacial advance (Wisconsin - Evans Creek), ice apparently did not emerge from the valley now occupied by Coldwater Lake (Figure 1), and hence, all glacial drift within the project area is considered to be pre-Wisconsin in age.

Glacial deposits in the project area have not been dated, however, based on weathering criteria and stratigraphic position, they are considered to be correlative with the Hayden Creek and Wingate Hill drifts described by Crandell and Miller (1974) in the Mount Rainier area. These two drifts probably represent two major pre-Wisconsin glacial advances. Clasts within the Wingate Hill drift show weathering rinds and are generally more weathered than the clasts ascribed to the Hayden Creek drift. In both cases, the tills are considerably more weathered than is characteristic of alpine glacial deposits of Wisconsin age in this area.

Ice apparently flowed down what is today the North Fork Toutle River and invaded tributary valleys, in some cases forming proglacial lakes. Lacustrine and fluvial deposits in Maratta Creek, Cow Creek, and the Hoffstadt Creek area (Figure 1) form the evidence for these lakes. Remnants of a delta that was formed by the deposition of sediments from Hoffstadt Creek into one of the proglacial lakes were found at an elevation of approximately 2,000 feet, adjacent to Hoffstadt Creek. Other lacustrine deposits and peat have been found between Geotech Ridge and Cow Creek. The proglacial fluvial and lacustrine sediments were subsequently overridden by glacial ice and the upper portions of these deposits were deformed. The evidence for this deformation is especially well developed near the mouth of Maratta Creek.

Two prominent end moraines are present in the upper portion of the Cow Creek drainage, and a major end moraine is present in the Hoffstadt Creek drainage. In the Cow Creek drainage, proglacial fluvial sediments were deposited against one of the moraines on the Toutle valley side and lacustrine sediments were deposited on the north side.

The downvalley limit of ice was not mapped, however Hayden Creek till is present on the hill adjacent to the U.S. Army Corps of Engineers N-1 debris dam (Figure 1). Brief reconnaissance efforts did not delineate till further downvalley. The terminal moraine for that glacier may be buried under the valley fill. Seismic refraction data indicates approximately 200 feet of valley fill on the north side of the valley near the N-1 debris dam.

Prior to glaciation, the volcanic and plutonic rocks in the project area were more directly exposed to the weathering environment. Much of the weathered rock has been stripped away by glacial erosion, however, remnants of the weathered horizons are present locally.

Tephra was nearly ubiquitous in the project area. Tephra layers have been recognized above, below, and within the glacial deposits and formed a significant portion of the fines found in the colluvium. Quaternary lahars were not mapped within the alignment corridor, however the Pine Creek lahar (Mullineaux and Crandell, 1981) was noted in the valley bottom near Camp Baker. Ten major eruptive periods for Mount St. Helens are known over the last 40,000 years, including the 1980 eruption (Mullineaux and Crandell, 1981). The presence of ash beneath the glacial till suggests that there may be even more eruptive periods, although the source of the pre-glacial ash is unknown.

Because of the topographic relief and the weathering environment in this area, the surficial deposits have been subject to mass movement activities. Colluvial deposits are present throughout much of the area and are particularly extensive beneath steep slopes such as those found near Elk Rock and Hoffstadt Mountain (Figure 1). Various landslide scars and landslide deposits were noted in the general project area. A large Quaternary landslide deposit is present below the steep southeast face of Hoffstadt Mountain.

Bedrock Geology

Extrusive and intrusive igneous rocks of various ages are exposed within the Mount St. Helens area. Extrusive rocks consist of andesites and pyroclastic rocks in various states of weathering and alteration. Most of these rocks exhibit a mineralogy indicative of very low grade metamorphism. The extrusives dip gently to the southeast and probably form part of the northern limb of the southeast-trending Napavine syncline (Phillips, 1987a,b). Interflow zones which appear to contain thermally-altered soils have been observed between flows and permineralized wood has been found adjacent to flow contacts. Diamicts, which probably represent Tertiary lahars, were found exposed near the western boundary of the project area and may exist in other areas.

The volcanic pile was intruded by dikes and sills which are andesitic and basaltic in composition, as well as by gabbro. Gabbro was found exposed along Bear Creek (Figure 1) below an elevation of approximately 2,500 feet and was encountered in one borehole along the alignment beneath glacial till.

Hydrothermal alteration of the volcanic rocks is probably related to both shallow and more deep-seated intrusive activity proximal to the altered rocks. In the Bear Creek area, and probably in other areas of the site, plutonic rocks intrude older volcanic rocks, and the adjacent volcanics exhibit evidence of hydrothermal alteration. Silicification, disseminated pyrite, calcite veinlets, and zeolite mineralization are considered as evidence of hydrothermal alteration. This alteration may be pervasive throughout the entire rock or may be localized along specific discontinuities. Rocks adjacent to hypabyssal intrusions are similarly altered.

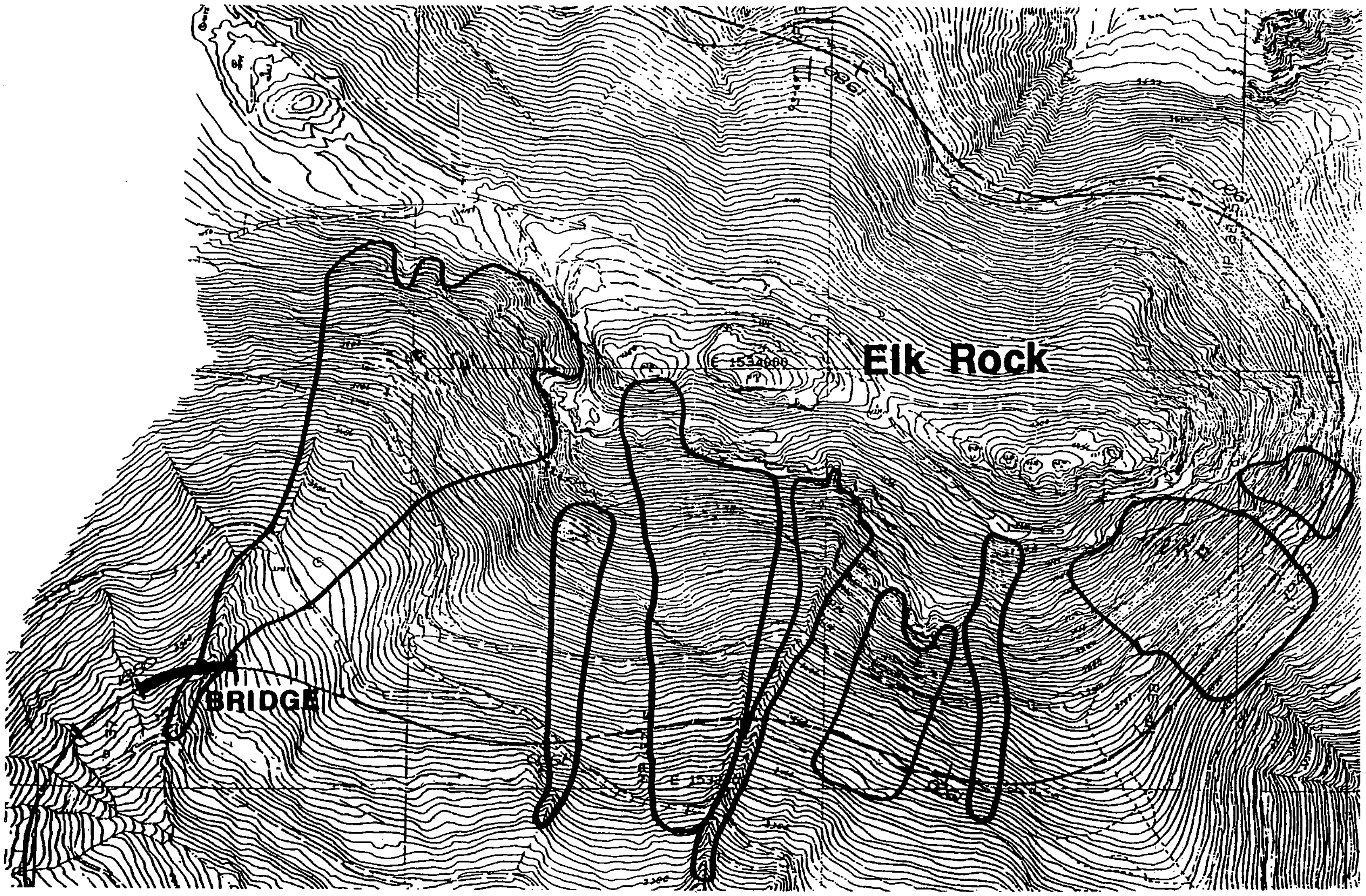
All of the bedrock units in the project area contain discontinuities. In some areas, the orientation, frequency, and persistence of the discontinuities appeared to be related to rock type. In some intrusive rocks, cooling joints were formed perpendicular to the cooling surface; these joints terminate at the boundary with the host rock. In the Hoffstadt Mountain area, the discontinuity frequency was observed to be generally higher in the andesite dikes and sills than in the tuff and agglomerate units. However, discontinuities in the tuff and agglomerate tended to be more persistent (up to 50 feet in length) than the discontinuities in the andesite. Although local areas may have fairly well-defined discontinuity sets, such as at Cow Creek and at Elk Rock, the orientation of discontinuities is not consistent over the project area. In general, the majority of the discontinuities are steeply dipping.

Mount St. Helens is in a tectonic environment which can be characterized by horizontal compression associated with plate motion. Seismicity following the May 18, 1980 eruption had a different spatial distribution than that occurring before the eruption. Prior to the eruption earthquakes were confined to a small volume in the shallow crust beneath the north side of the volcano. After the eruption seismic events occurred at depths down to 10 miles and were distributed both north and south from the mountain. These more recent seismic events are consistent with a strike-slip seismic zone trending northwest from the mountain (Weaver et al, 1981). Cataclastic rocks were found along the highway alignment, sometimes coinciding with lineaments. Evaluation of fault activity and seismic hazards in the project area was beyond the scope of this investigation.

GEOLOGIC ISSUES RELATED TO THE 1980 ERUPTION

Deforestation

Of the 17 miles of alignment investigated by Golder Associates Inc. approximately 15 miles had been deforested by the lateral blast associated with the 1980 eruption or by clearcut logging. Deforestation gave rise to potential snow avalanche zones where none had existed before the eruption. Steeper areas along the alignment which formerly had sufficient forest cover for the snowpack to be firmly anchored to the slopes were now almost entirely devoid of vegetation. The deforested condition provides a smooth base upon which large avalanches may release and flow given the appropriate snowpack conditions. As shown in Figure 3 potential avalanche paths in the area of Elk Rock cross the alignment in eight places (Mears, 1988). Should snow



(after Mears, 1988)

0 800 1600
Feet

Figure 3
Avalanche paths in the Elk Rock area;
along the proposed route of SR-504

avalanches actually occur in these areas the vegetation regrowth may be inhibited, further perpetuating the susceptibility of the slopes to snow avalanches.

Deforestation can also trigger landslides. Areas on Elk Rock which, based on pre-1980 aerial photographs, were forested and had a stable soil cover are now subject to numerous small landslides. Tree-root disintegration becomes a significant factor in landslide development after a period of 3 years or longer (Bishop and Stevens, 1964) and may be a factor in the slides found along the slopes of Elk Rock. These landslide areas were found mostly below the alignment or were shallow in the alignment area, and did not affect design considerations. Had the alignment been placed on the lower and steeper slopes of Elk Rock these slides would have merited serious consideration.

Hoffstadt Creek has large amounts of logging/blown-down debris and volcanic ash entrained in its channel. Much of this material was mobilized during the December storm of 1980 and could be mobilized again during low frequency, high intensity runoff events. In the upper portion of Hoffstadt Creek, the debris formed a temporary dam which impounded water. When this dam broke, the discharge of the creek almost certainly rose very rapidly, as did the capacity of the stream to transport boulders and large logs. Such debris torrents could potentially compromise the integrity of bridge piers should they be located in the channel area used by the torrent.

Soil Erosion

Tephra was deposited over the area of the alignment in a layer which varied from less than one-half centimeter to over 50 centimeters thick (Waitt and Dzurisin, 1981). Evidence of tephra and wood debris plugging culverts and the accumulations of up to 6 feet of tephra adjacent to existing logging roads brought up the issue of erosion rates and how to handle large volumes of loose silt adjacent to the alignment.

Qualitative field observations suggested that much of the tephra had stabilized and would not present unusual erosion problems unless it was significantly disturbed by construction equipment. Quantitative measurements by Collins and Dunne (1986) indicate that the rapid rates of erosion during the first year after the eruption declined substantially in the following two years, once a stable rill network developed and more permeable and less erodible substrates were created or exposed. Projecting the rate of decline of erosion Collins and Dunne (1986) suggest that only about one-sixth of the tephra on the hillslopes around Mount St. Helens will be removed by water erosion before soil creep and other forms of mass wasting again dominate hillslope evolution.

Based on this information it was determined that no particular measures of tephra stabilization were required to decrease the rate of tephra erosion along the highway alignment after construction mitigation measures had been removed. Although revegetation of soft loose soils is consistent with sound practice, it is interesting to note that even without the effects of revegetation, erosion rates of the 1980 Mount St. Helens tephra decreased rapidly over a period of three years.

Safety of Personnel

Safety precautions, with respect to the hazards of eruptive activity during the field investigation, were considered adequate. The apparent rarity of lateral blasts of the magnitude of the May 18 eruption (Crandell and Hoblitt, 1986), the extensive monitoring systems which are operated by the U. S. Geological Survey, the emergency warning system operated by Cowlitz County, and Golder Associates Inc. evacuation plans supported this assertion. However, lakes dammed by the 1980 debris avalanche deposits are subject to breakout, and were a cause for concern during the project planning stage. Catastropic flooding of the North Fork Toutle River valley could destroy the trailer camp established by Golder Associates Inc. for project field operations. Initial reports suggested that this hazard was not large (Meyer et al, 1985).

The level of Spirit Lake on the north side of Mount St. Helens was raised approximately 200 feet by the debris avalanche deposits and Spirit Lake was considered the most hazardous of the lakes dammed by the avalanche. In 1985 the U. S. Army Corps of Engineers completed a tunnel to drain the lake to a lower level (Sager and Chambers, 1986) and hence, that hazard was mitigated by the time the field work was undertaken. Castle Lake adjacent to Mount St. Helens on the south side of the North Fork Toutle River has also been considered susceptible to failure (Meyer et al, 1985; Laenen and Orzol, 1987), however, it was the subject of only minor remedial measures.

Possible failure of the Castle Lake blockage was publicized in the popular media during the field investigation phase of this project in 1987, which prompted a re-investigation of this hazard by Golder Associates Inc. personnel. For failure to occur, Laenen and Orzol (1987) postulated that a magnitude 6.8 earthquake would need to occur at a time of unusually high groundwater levels. After independent analysis of the available data on the lake blockage, site inspection, and discussions with various U. S. Geological Survey personnel, it was concluded that the earthquake would represent the best early warning of a flood. Evacuation plans tied to site personnel sensing a major earthquake were considered adequate warning and protection against this hazard.

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FROM GEOPHYSICS TO DESIGN IN AN ENVIRONMENTALLY SENSITIVE AREA

by

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ABSTRACT

Relocated U. S. Route 36 passes through an environmentally sensitive area adjacent to the Illinois River in west central Illinois. The alignment, court contested, has constraints in response to environmental concerns restricting right of way and site access and requires seasonal work shutdowns. The difficult terrain and environmental limitations hampered conventional geotechnical investigations, necessitating an innovative geophysical program to adequately describe subsurface conditions within the time schedule. Seismic refraction was employed to determine depths to bedrock in the cut sections, and along retaining wall alignments. At selected bridge foundation locations compressional and shear wave velocity values were measured resulting in moduli and unconfined compressive strength calculations for individual layers. Recommendations for bridge foundation design were based upon the comparison of conventional geotechnical exploratory data and seismic refraction and cross-hole velocity measurements.

INTRODUCTION

During the last two decades, the State of Illinois has had a relocation of U.S. Route 36 in various stages of planning, design and construction. The new alignment is known as I-72 from Champaign to Springfield and as the Central Illinois Expressway from Springfield to Quincy. This paper deals with a portion of the Central Illinois Expressway improvement on the west side of the Illinois River Valley through an environmentally sensitive area.

The original corridor study evaluated several alignments for crossing the Illinois River prior to selecting the alignment known as the Napoleon Hollow alternate passing through the Pike County Conservation Area (refer to Figure 1). Following the receipt of bids for constructing two parallel bridges across the Illinois River in 1980, an injunction was filed delaying bridge construction until the United States Federal Court could be assured of the propriety of the selected alignment. A new environmental impact statement was prepared evaluating several alignment alternates considering both comparative economics and effect upon the environment. Following a review of these additional studies, it was determined that the original alignment through Napoleon Hollow was still preferable.

The court allowed construction of the approach embankments to begin in 1985 and construction of the bridges themselves began in 1986. Bids were received in late 1987 for construction of six structures, eleven retaining walls, grading and drainage for the roadway on the west side of the Illinois River. The anticipated final cost of the two miles of improvement including the Illinois River structures and Napoleon Hollow is approximately $60 million.

PROJECT RESTRICTIONS

During the five years of litigation, all field activities associated with design and construction were completely curtailed in the Napoleon Hollow area until the injunction was lifted. One of the conditions of the court settlement restricts all activities in the hollow between November 15th and March 1st of each year due to possible presence of wintering Bald Eagles.

Additionally, trees cannot be cut in the hollow between April 2nd and September 14th of each year to prevent disturbance of nesting and breeding of the endangered Indiana Bat. Further, the right-of-way for the roadways skirting Napoleon Hollow has been laterally restricted to minimize land taken in the Pike County Conservation Area. This precludes the embankment and cut slope configurations normally used. This restriction necessitated construction of 104,000 square feet (surface area) of retaining walls in the mile long length of Napoleon Hollow.

ALIGNMENT DESCRIPTION

The terrain along the west side of the Illinois River through the entire reach under consideration is that of a limestone bluff, very steep, and heavily timbered, with a total relief of about 200 feet. The alignment approaches the Illinois River valley from the west through Napoleon Hollow, a steep and rugged tributary valley. To take advantage of the natural topography and the scenic beauty of the area while lessening the impact on wildlife, the river bridges, as well as the roadways through the hollow, are constructed on a divergent alignment (refer to Figure 2). The eastbound and westbound roadways actually are located at the southern and northern edges, respectively, of Napoleon Hollow rather than through the center of the valley. The two roadways diverge at the west end of the hollow and follow along the bluffs on either side. At the west abutments of the river bridges positioned high on the west bluffs of the Illinois River, the two roadways will be about 700 feet apart. This arrangement will provide the traveler a scenic view, reduces the size of staging and construction zones and skirts the Napoleon Hollow area rather than bisecting it.

GEOLOGY

The general geologic background of the Napoleon Hollow is that of an erosional valley tributary to the preglacial Mississippi River. The Illinoian and Wisconsin glaciers displaced the Mississippi River westward to its present course. The abandoned river valley is now occupied by the Illinois River. Thus, the geologic history of Napoleon Hollow dates back to the Pre-Pleistocene era.

During the Illinoian glaciation, the hollow was partially filled with glacial debris. Subsequently, it was again downcut by stream action, however, some of the side tributaries to Napoleon Hollow do not now directly overlay the former ones. Thus, the bedrock profile is somewhat less than a mirror image of the ground surface.

The bedrock encountered throughout this project is of the Burlington-Keokuk Formations of Mississippian age. The combined thickness of these two formations is about 200 feet. Limestone, chert layers and nodules, along with some beds of dolomite are the major rock types. It is believed that advanced in-situ weathering of the dolomitic beds results in the formation of fine-grained silt size particles of varying degrees of induration. At least the upper portion of this member has been subjected to degradation and solution by ground water. The rock is commonly badly weathered at its surface, and, at times, to substantial depths. The patterns and vertical extent of joint weathering are not completely definable by the usual investigative techniques and cannot be predicted with accuracy.

The profile in Napoleon Hollow from the surface downward consists of 30 or more feet of loessial material deposited during the Wisconsin glaciation. The loess rests upon Illinoian glacial till, or, where the till had been removed by erosion, directly upon the weathered bedrock surface or residuum.

SUBSURFACE STUDIES (STANDARD BORING INVESTIGATION)

IDOT had a considerable fund of experience with the Burlington-Keokuk Formations prior to involvement in Napoleon Hollow. The wide range in variability in weathering, both vertically and horizontally as well as its unpredictability, had been experienced in other rock cuts in the area as well as its effect on aggregate production in local quarries. Therefore, it was acknowledged at the outset that detailed sampling of rock materials would be essential to meaningful interpretation.

The geotechnical investigation pursued by IDOT consisted of split spoon sampling through the unconsolidated materials and continuous rock cores of the bedrock. The borings for the retaining walls were offset from the roadway centerline about 70 feet to agree with the wall locations. The equipment used in the geotechnical investigation consisted of a Mobile B-47 mounted on an all-terrain carrier; where topography permitted, the drilling operation was supplemented with a truck mounted CME-75. Both units employed the hollow auger system through the unconsolidated materials.

A total of 600 lineal feet of rock core were taken. Approximately 90 percent of the rock coring was accomplished with BX tools and the remainder with NX. Both rock core systems employed the double tube split inner barrel system. Each run was logged for its characteristics, percent recovery, and percent RQD. While the majority of the core runs were five feet in length, the detailed core description provided an opportunity to adjust "theoretical runs" to better reveal low RQD values for limited zones.

Geotechnical work was completely curtailed during the years of the injunction and was seriously limited by the work cessation restriction favoring wildlife. With the time lost due to the injunction and the restrictions placed on field work, there developed a sense of urgency to place this section under contract to keep pace with the timetable of overall C.I.E. completion. This sense of urgency combined with the restrictions and coupled with the problems of difficult access precluded a completely adequate geotechnical investigation using the usual methods. The early recognition of this situation resulted in the boring locations being distributed throughout the various structures. To provide the additional data necessary to describe the subsurface geology, a contract was awarded to Weston Geophysical to supplement the boring information using geophysical methods.

GEOPHYSICAL EXPLORATION PROGRAM

A comprehensive site visit by IDOT and Weston Geophysical personnel defined the objectives and scope of the initial seismic refraction exploration program. The overall project concept and specific design and environmental considerations were reviewed along with available geologic data and test boring results. Bedrock and overburden outcrops, landforms and topographic features as observed in the field were factored into the refraction profiling program.

Ultimately, the geophysical investigations included two phases of seismic refraction profiling for cut sections and retaining walls, and a special program of in-situ velocity measurements to obtain information on the nature of the subsurface materials at the selected bridge foundation locations.

REFRACTION PROFILING FOR CUT SECTIONS AND RETAINING WALLS

The objectives of the first phase of the surface seismic refraction investigations required determination of depths to rock in the cut sections and at the positions of the retaining walls, and to determine the characteristics of land forms. The refraction results were also used to corroborate geologic mapping information such as the quality of bedrock observed in outcrops and the presence or absence of bedrock in stream valleys. During the initial phase in 1985, 16,600 linear feet of refraction profiling were acquired by one seismic exploration crew supplemented by a second crew to complete this effort by the end of the court decreed exploration period.

The second phase of the refraction investigations included the acquisition of detailed refraction data in the vicinity of proposed bridge structures and at locations where the anchors for the retaining walls would be placed. During Phase II, completed in 1987, 7,500 linear feet of profiling data were acquired.

In Phase I, most of the refraction data were acquired using 24 channels spaced along a 400 foot long seismic spread with geophone intervals of 10 and 20 feet as shown on Figure 3. In Phase 2, we further improved the resolution power relative to individual layers and to the seismic velocities of those layers by shortening the spreads to 250 feet in length and utilizing only 10 foot geophone spacings (see Figure 3).

The seismic techniques, both refraction and in-situ, make use of the earth's ability to transmit sound waves; the velocity of the sound in a material is a direct function of the physical properties of that material. From an overall design standpoint, material identifications are based on compressional wave velocity values. For detail design considerations such as bridge foundations where loads are a factor or in the case of dynamic design, the shear wave velocity values of the materials become significant. The use of both compressional and shear wave velocity values allows calculation of Poisson's ratio and the various dynamic moduli (shear, Young's and bulk), recognizing the fact that these measurements are made at relatively low strain levels of 10^{-3} and 10^{-4} in/in.

At some sites, including this project in Illinois, we were able to expand a generalized velocity/material correlation as shown on Figure 4 into a site-specific means of identifying different types of materials through correlations with test borings and field observations. The site specific material correlations developed for the Napoleon Hollow area are therefore presented on Figure 5.

The condition of the bedrock is critically important to slope and foundation design as well as for determining quantities for rock excavation payment purposes. There is an overall consistency throughout the project area in terms of a relationship between seismic velocity values and bedrock conditions. From the surface refraction measurements, the velocity of the bedrock has ranged from as low as 4,000 ft/sec for materials classified as weathered and broken limestone, to as high as 14,000 ft/sec for competent limestone beds. Velocities of 13,000 ft/sec and higher are indicative of competent limestone, probably relatively thickly bedded with few, if any, thin zones of weathered or decomposed materials, such as silt layers (decomposed dolomite); boring logs for bedrock in this velocity range show high recovery and generally high RQD values. Velocities in the range of 11,000 to 12,000 ft/sec are indicative of a more thinly bedded but still competent limestone, probably containing a few fractures and weathered zones and perhaps some softer silt layers between the individual limestone beds; boring logs for bedrock in this velocity range show medium to high recovery and low to medium RQD values. As the seismic velocity values decrease to 10,000 ft/sec or less, the amount of weathering in the bedrock and the number of silt layers apt to be encountered increases, is evidenced on the boring logs by low to medium recovery and very low RQD values.

BEDROCK CONDITIONS AND SEISMIC VELOCITY CORRELATIONS

To illustrate the correlation between bedrock condition and seismic velocity values, velocities were measured on exposed bedrock along the roadways at abutment locations for the bridges crossing the Illinois River (east end of our survey area). At the bridge abutment on the eastbound roadway, velocities were measured at the top of the fractured and broken bedrock exposure shown in the photo on Figure 6. Velocity of the near-surface bedrock was determined to be 5,000-6,000 ft/sec. This seismic velocity increases to 11,000 foot per second in the unweathered and unfractured (but relatively thinly bedded) limestone units at the bottom of this exposure. A more massive bedrock condition was encountered at the Illinois River bridge abutment for the westbound roadway. Velocities of 13,000 ft/sec were measured at the base of the cut shown in the photo on Figure 6 on a relatively massive limestone unit similar to those exposed near the base of the cut. The somewhat more thinly bedded and slightly weathered bedrock near the top of the rock exposure in Figure 6 has seismic velocities, determined from surface refraction measurements, in the range of 10,000 to 11,000 foot per second. In effect, the more thickly bedded the unit, the higher the seismic velocity for a comparable degree of weathering and fracturing.

One of the key questions for the Napoleon Hollow area concerned the presence of silt layers within the bedrock sequence and, therefore, the question at what depth does competent intact bedrock exist. Theoretically, the seismic technique is able to resolve the presence of layers with a thickness equivalent to one quarter of the seismic wave length or greater. Typical frequency values for seismic wave arrivals from the surface refraction technique were in the order of 100 hertz. For bedrock with a velocity of 10,000 foot per second, this translates to a seismic wave length of approximately 100 feet. The measured seismic velocity represents an average of bedrock conditions within that 1/4 wave length thickness; therefore, the seismic technique was able to detect a rock mass whose thickness is 25 feet or greater. In other words, if a layer of bedrock is detached or separated from an underlying layer of rock by a continuous silt layer, the layer of bedrock would not be detected seismically unless it is approximately 25 feet thick or greater. For overburden layers at depth, the lower seismic velocities result in shorter wave lengths with the resulting capability of resolution for individual overburden units in the range of 10 to 15 feet.

The Phase I and Phase II seismic refraction coverages are illustrated on Figure 7. In the Wall #2 area, seismic data were acquired along the baseline for quantity estimates, along the 70 foot right offset for retaining wall depth and foundation conditions, and along the 130 foot right offset line for the retaining wall anchors. Crosslines were oriented along topographic features to maximize the quality and accuracy of the confirmatory information. The seismic profiles for the Wall #2 area, presented on Figure 8, depict a complex subsurface situation with bedrock shallow and high in elevation to the west (elevation 540) and decreasing rapidly in elevation to the east.

The velocity of approximately 10,000 ft/sec is indicative of bedrock containing competent zones and a significant amount of weathering and fracturing. Borings at 65+00 and 65+80 along the retaining wall (70 feet right of the baseline) describe a bedrock condition consistent with the seismic velocity value identification. The boring at 65+00 actually shows that the bedrock is relatively more competent near surface but decreases in quality with depth and also laterally towards another boring at 65+80. This is a good example of how the seismic velocity value is descriptive of the overall or average bedrock condition. If the 20 foot thick section of competent bedrock encountered in the boring at 65+00, 70 feet right, extended the full length of a 400 foot seismic spread, the measured seismic velocity value would likely be in the order of 13,000 ft/sec.

The layer with a seismic velocity of 5,000-6,000 ft/sec correlates with the broken and weathered limestone identified on many of the test borings drilled between Stations 68+50 and 71+50. The westward extent of this low velocity rock zone was further investigated by crosslines at Stations 68+53 and 67+53. The crossline at 67+53 indicates that this weathered rock zone with a velocity of 6,000 ft/sec is thickest along the baseline and eventually disappears to the west in the vicinity of Stations 66+00 to 67+00.

The 1,100 and 1,600 ft/sec velocity values correlate with and are typical of the loess deposits that exist throughout the Napoleon Hollow area.

BRIDGE FOUNDATIONS

The overall objective of the geophysical investigations for the bridge structures was to determine the physical properties of a deep section of bedrock so that a foundation elevation and type of foundation can be selected by the design engineers. Existing boring information indicated that some silt and/or weathered rock conditions could be present between layers of competent rock at several bridge foundation locations. Detailed seismic refraction profiling using 10 foot geophone spacings along 250 foot long seismic spreads was conducted at many of the bridge structure locations. These data were correlated with previous seismic investigations and test boring information to obtain as complete knowledge as possible concerning subsurface conditions at these bridge foundation structure locations prior to any in-situ geophysical measurements. At many of the bridge locations, the correlated seismic refraction and test boring data clearly defined the depth and type of bridge foundation; spread footing, piles, or drilled caissons. At other locations, the type and depth of foundation required further evaluation due to suspected and indicated variations in the condition of the bedrock with increasing depth.

At five selected bridge structure locations representative of these varying conditions, the bedrock was further evaluated through a program of geophysical borehole logging and seismic in-situ velocity measurements using a pattern of four test borings. The selected boring locations insured that the cross-hole velocity measurements defined the subsurface velocity layering throughout the pier foundation area. The borings were drilled using the percussion technique.

Since only field logs of penetration conditions were obtained from the percussion drilling operation, these boreholes were geophysically logged to determine the nature of the subsurface materials. The logging suite included single point dry hole resistivity/caliper, natural gamma ray, and high resolution gamma-gamma density. The purpose of the borehole logging program was to define the stratigraphic sequence more accurately, and correlate zones between boreholes for locating the elevations of the cross-hole measurements.

The most diagnostic logging technique was the high resolution gamma-gamma density tool. This density log provided a continuous record of the bulk density of the strata, thereby identifying both the hard and soft zones of interest. Hard limestone zones showed a bulk density of 2.5 to 2.7 grams/cc [156 to 168 lbs/cu ft], soft zones showed bulk densities as low as 1.6 grams/cc [100 lbs/cu ft]. The caliper log identified enlarged sections of the borehole, also indicative of soft rock.

Based on the layering determined by the density log, in-situ seismic measurements of both the compressional and shear wave velocity of the bedrock were conducted at selected elevations using the cross-hole technique. Based on these measured velocity values, the dynamic elastic properties, Young's modulus, bulk modulus, shear modulus, and Poisson's ratio are determined. Poisson's ratio is directly computed and the various moduli (Young's, shear and bulk) are calculated using in-place density values of the materials. The unconfined compressive strength of the bedrock has also been determined based on previously established empirical relationships with the compressional wave velocity (see Appendix A).

Through the use of a digital data acquisition and processing system with timing capabilities to a few tens of microseconds, seismic wave frequencies in the range of 300 to 500 hertz were observed for the in-situ crosshole velocity measurements. Based on the calculation of wave length previously described above, we conclude that the seismic crosshole technique was able to investigate the lateral variations/continuity of low velocity bedrock zones in the range of three to five feet thick. To resolve the velocity values of the thinnest layers, the boreholes were closely spaced to provide control and resolution on refracted velocity data as depicted on Figure 9. In a situation where rock conditions are apt to very laterally and vertically over a short distance, the seismic velocity measurements have a clear advantage over the analysis and testing of drill core since the material recovered from drilling is more likely to be the higher quality material, thereby leading to a potentially biased view of the actual subsurface conditions.

The investigation at Bridge 75-6B-1 on the eastbound roadway (see Figure 7) are illustrative of the previously described methodology. Detailed seismic refraction data were obtained along crosslines near the west abutment, Pier numbers 1 and 2, and at the east abutment. All of these crosslines disclosed zones of low velocity material identified by test borings as broken and weathered limestone for the most part (see Figure 10). The crosslines in the vicinity of the west abutment indicate that competent bedrock is at a minimum depth of 50 feet below ground surface (elevation 450 to 460 feet).

In the vicinity of Pier number 3 at Station 72+88, the seismic crossline and baseline information described a condition where the velocity of the bedrock gradually increases from a low value of 3,000 ft/sec near-surface through a zone of 6,000-7,000 ft/sec, probably broken limestone, to relatively hard and competent bedrock with a seismic velocity of approximately 14,000 ft/sec at a depth of about 18 to 23 feet (elevation 465 to 470 feet).

The areas in the vicinity of Pier 1 and Pier 2 were selected for borehole geophysical investigations. Generally, the four boreholes drilled at each structure were positioned such that the in-situ seismic velocity measurements would be representative of the rock mass beneath the entire structure. In the vicinity of Pier 1, the velocity measurement locations (boreholes) were slightly offset from the pier (see Figure 11) because the drill rig could not be positioned on the steep topography at the pier location. The results of the geophysical logging and the measured in-situ velocity values for the Pier 1 area are presented in Figure 11. The measured shear wave velocity values are quite high at this location indicating competent bedrock; however, the lower density zones noted on the geophysical logs at approximate depths of 32 and 39 feet (elevations 440 feet and 433 feet respectively) clearly have lower seismic velocity values translating to lower strength values. The in-situ seismic velocity values and the corresponding elastic moduli and unconfined compressive strength values are presented in tabular form on Figure 12.

Based on the overall conditions encountered throughout the area, it was recommended that bridge foundations be located on materials with a shear wave velocity of 5,000 ft/sec or higher. As noted earlier, each measurement elevation represents an average velocity value for a few foot thickness of the bedrock. Accordingly, zones with shear wave velocities of 4,000 to 4,500 ft/sec or less probably include a significant amount of soft material.

OVERVIEW OF GEOTECHNICAL INVESTIGATION

As previously stated, the geotechnical investigation was unique in that numerous limiting conditions posed a significant deterent for timely completion of this study. The cumulative effect of court ordered environmental restrictions, adverse site terrain, complex bedrock conditions, and a stringent time deadline, were all obstacles which hampered progress.

As with any geotechnical investigation, its value is measured by the extent that the program satisfies the objectives. The forementioned constraints required an investigative method capable of accumulating a considerable fund of data in a comparitively short time. By utilizing geophysical methods to interpolate between and extrapolate from relatively widely spaced borings, the Napoleon Hollow geotechnical study was completed on schedule.

As of this date, 50% of construction having been completed, no redesign has been necessary, and the combined geophysical and geotechnical program has proven very satisfactory for both design and construction purposes.

The following paragraphs consider first the rock cuts and retaining walls, second the bridge structures, and last the writers conclusions.

ROCK CUT DISCUSSION

The rock quality for the several cuts appeared to be satisfactory for IDOT conventional 1/4:1 rock slope design. The rock removal in the cut sections is accomplished with the pre-splitting technique after the tieback walls have been constructed. The weight per delay of the blasts will be adjusted to limit ground motion (peak particle velocity, inches per second) to acceptable levels based upon the distance to nearby bridges and/or walls, and the age of concrete in those structures.

In spite of the variable character of the rock materials, it has only been necessary to design one localized rock cut stability bench. It has been recognized that locally joint weathering can be quite advanced. While no specific location is presently known to exist, if encountered during construction a weather wall can be built to assure continuity of the apparent cut face across the weathered zone.

WALL TYPE ALTERNATES

The unconsolidated material overlying the bedrock consists of residuum and loess, and with the near vertical design slopes must be protected by some type of earth retaining structure. Among the wall types considered were vertical cantilever walls, Reinforced Earth, Evergreen Wall, and anchored tieback walls.

Vertical cantilever walls require excavations behind the wall face equal to about 40% of the wall height to accomodate footing construction; Reinforced Earth requires excavation behind the wall face equal to about 75% of the wall height to provide for select backfill and placement of reinforcement strands; Evergreen Wall is a gravity type constructed of elements forming cribs which are backfilled and extend back into the slope, the horizontal extent falling between the two previous listed types. In each of the foregoing wall types, excavation behind the wall face would require removal of trees to a point back of the construction excavation line, 50% or more of the wall height behind the wall face. Additionally, to maintain the temporary earth slope in a vertical attitude with heights to 40 feet during construction would require temporary protection such as steel sheeting and/or some form of tieback retention.

SELECTED DESIGN WALL TYPE

Environmentally, the anchored tieback wall system was clearly the most desirable since it is least damaging to the environment. Additionally, since any of the alternate wall types considered would require a temporary retaining system to permit construction, the anchored tieback wall also represented significant cost advantages. Esthetically, retaining wall heights have been limited to about 20 feet with a 12 foot bench at the top of the rock cuts, and 10 feet between upper and lower walls (Refer to Figure 13). The walls have earth toned textured concrete faces, and the benches will be planted with suitable native flora.

Soil anchors were briefly considered for the applicable wall locations, but were judged trouble prone. The loess soil frequently has an in-situ moisture content of 5% or less, and texturally consists of about 95% silt size particles with a weakly cemented structure of low density. Under these conditions, a shear failure develops at low strain and results in a significant loss in volume due to a collapse of the soil structure. The in-situ conditions of the loess made the taking of soil samples of acceptable quality for laboratory testing very difficult.

With this background, rock type anchors were selected for all walls. Some of the anchor strands exceed 100 feet in length and a substantial proportion are designed for installation at 45^{o} to the vertical because of the depth to rock.

BRIDGE STRUCTURE DISCUSSION

Within the Napoleon Hollow split alignment there are six bridge structures, consisting of highway grade separations and stream structures; four are 3 span units, the other two consist of 4 spans each. The average span length is about 100 foot. Other than for environmental considerations, a 5x5 box culvert would have satisfied hydraulic opening requirements for all bridges except the grade separation.

The foundation types employed in support of these bridges include spread footings, point bearing steel H Piles and 36" diameter drilled shafts designed for friction and/or end bearing. The loess soils were not considered adequate for the design footing loads, all substructure units being designed to transfer the stresses to bedrock or glacial ice contact deposits.

The lightest loaded foundation element is an abutment spread in rock with a design loading of 1.3 TSF; the heaviest spread footing is also an abutment with a maximum footing pressure of 2.8 TSF.

In reviewing the subsurface data the depth of weathering often times appeared to deepen toward the the bottom of the tributary valleys. Thus, at the pier locations near the existing channel bottoms, the quality of the rock frequently left something to be desired. Further, in at least one location, the valley had in earlier times been entrenched as much as 40 feet deeper than at present, and during the Pleistocene been filled with a mass of limestone cobbles and boulders. Drilling and coring did not distinguish this location as an unusual area but the seismic longitudinal and cross line profiles clearly outlined the valley and the valley fill. The valley fill has a seismic wave velocity of 3000 to 4500 feet per second, and is underlain by rock with a velocity of 13000 feet per second.

Certainly, this would be a less than desirable location to drive steel bearing piles. Penetration depths would be very erratic, and refusal would no doubt develop on one of the boulders rather than bedrock. Drilled shafts appeared a logical answer, but placement through 40 feet plus of boulders and cobbles promised to be expensive. The designers instead chose to employ these shafts as friction elements, and determined that the required lengths would be about half of that required for end bearing on rock.

Though the foregoing paragraph describes what was believed to be an atypical situation, with the combination of deep weathering, silt filled joints, low RQD, and low seismic velocities, it seemed prudent to employ the friction concept for virtually all of the drilled piers.

CONSTRUCTION DRILLING METHOD

The Burlington-Keokuk Formations typically includes numerous chert layers occurring both as nodules and as single beds varying in thickness. In zones of limited weathering, the chert seams are highly resistant to drilling with abrasion type drill tools. Rate of bit penetration has often proven very slow with the contractor at times getting penetrations of one foot per hour or less. To increase rate of penetration for the soldier pile rock sockets, the drilling contractor subsequently utilized a 30 inch diameter "down-the-hole hammer" improving production about 5 fold.

CONCLUSIONS

Our experience has demonstrated that the core borings can report for similar percents of recovery and RQD several geological conditions. The rock recovered by coring of in-situ limestone where advance weathering has occurred looks quite similar to the product of coring in a mixture of residuum and ice contact deposits. The geophysical testing clearly defines the two conditions. Recognizing these conditions better serves the designer in his selection of the appropriate structural support method, and protects against a design of a foundation erroneously placed on "a rock" rather than "the rock".

The experience with this contract has demonstrated to the authors that the use of neither geophysical surveys or conventional drilling and sampling by themselves would have been sufficient to provide an adequate insight to the subsurface engineering geology of this complex and variable site. It is the authors judgement that conventional sample boring and coring is an ideal basis for field exploration to provide bearing capacity input for the designers use. We would also recommend that this boring program be supplemented and complimented by a geophysical survey. This combined technique is conducive to a more detained understanding of subsurface geological conditions.

Specific seismic velocity data can be obtained at a specific footing location employing crosshole techniques, however, where discreet abrupt lateral transitions occur in bedrock lithology, this condition is averaged by sophisticated seismic techniques. It would be difficult for both designer and builder to employ seismic information without rock cores for comparison. The total fund of data that may ultimately be accumulated by conventional geotechnical survey and seismic surveys should ideally be sythèsized in a manner to be readily available for the users thus reducing the likelihood of an engineer arriving at a decision without cognizance of all the available data.

Significant benefits have been realized on this project with the addition of geophysical testing to the usual geotechnical investigation. Time constraints and difficult terrain precluded a normal boring program, and the cost incurred by inclusion of geophysical testing was substantially less than the number of borings it replaced. Further, the use of cross hole testing on this project provided a means of seismically investigating weaker strata below high velocity beds as well as a more detailed testing below substructrure units. Complex geology resulted in a given bridge structure employing spread footings, point bearing steel H piles and drilled shafts. The data provided to the designers proved sufficiently detailed for plan preparation. Additionally, the geophysical data have proven valuable in the resolution of disputes between IDOT and its contractors in the area of changes in conditions.

APPENDIX A

Theoretically, the seismic velocities for rocks are related to the modulus of deformation and modulus of rigidity of the rock mass (Brown and Robertshaw, 1953; Envision, 1956; Coates, 1981; Schreiber, Anderson, and Soga, 1973).

Poisson's Ratio (σ), the ratio of the transverse normal strain to the longitudinal normal strain of a body under unaxial stress, is calculated from seismic velocity values as follows:

$$\sigma = \frac{\frac{1}{2} - \frac{V_s}{V_p}^2}{1 - \frac{V_s}{V_p}^2}$$

The modulus of deformation (E), better known as Young's modulus, is related to compressional wave velocity,
V_p, according to the following:

$$V_p = \sqrt{\frac{E\ g\ \ (1-\sigma)}{\rho\ \ (1-\sigma-2\sigma)}}$$

Where g = is the acceleration due to gravity and ρ is the unit weight of the rock.

The modulus of deformation, E, is defined as the ratio of normal stress to normal strain or the tangent to the stress-strain curve. This modulus is called the modulus of elasticity for elastic bodies.

The modulus of rigidity or shear modulus (G) is related to the shear wave velocity, V_s, according to the following:

$$V_s = \sqrt{\frac{G\,g}{\rho}}$$

This modulus is the ratio of shear stress to shear strain and is determined from the slope of the tangent to the stress strain curve.

The Bulk Modulus or Modulus of Compression (K) is defined as the ratio of the change in average stress to the change of volume and is computed from seismic velocities as follows:

$$K = \rho\left(V_p^2 - \frac{4}{3} V_s^2\right)$$

The unconfined or unaxial compression strength describes a materials response to a vertical compressive load without any horizontal confinement. Since this is a measurement of strength only and not of elastic properties, a theoretical relationship between unconfined compressive strength and seismic velocity values can not be established. However, empirical data has established a relationship between unconfined compressive strength and the compressional wave velocity (the modulus of elasticity) (Coates, 1981).

REFERENCES

Schreiber, E., Anderson, O.L, and Soga, M., 1973, Elastic Constants and Their Measurement, McGraw-Hill Book Company, New York, 196 p.

Coates, D.F., 1981, Rock Mechanics Principles, Canada Center for Mineral and Energy Technology, Energy Mines and Resources Canada.

Envision, F.F., 1956, The Seismic Determination of Young's Modulus and Poisson's Ratio for Rocks In-Situ, Geotechnique, 118-123p.

Brown, P.D. and Robertshaw J, 1953, The In-Situ Measurement of Young's Modulus for Rock by a Dynamic Method, Geotechnique, 283-286 p.

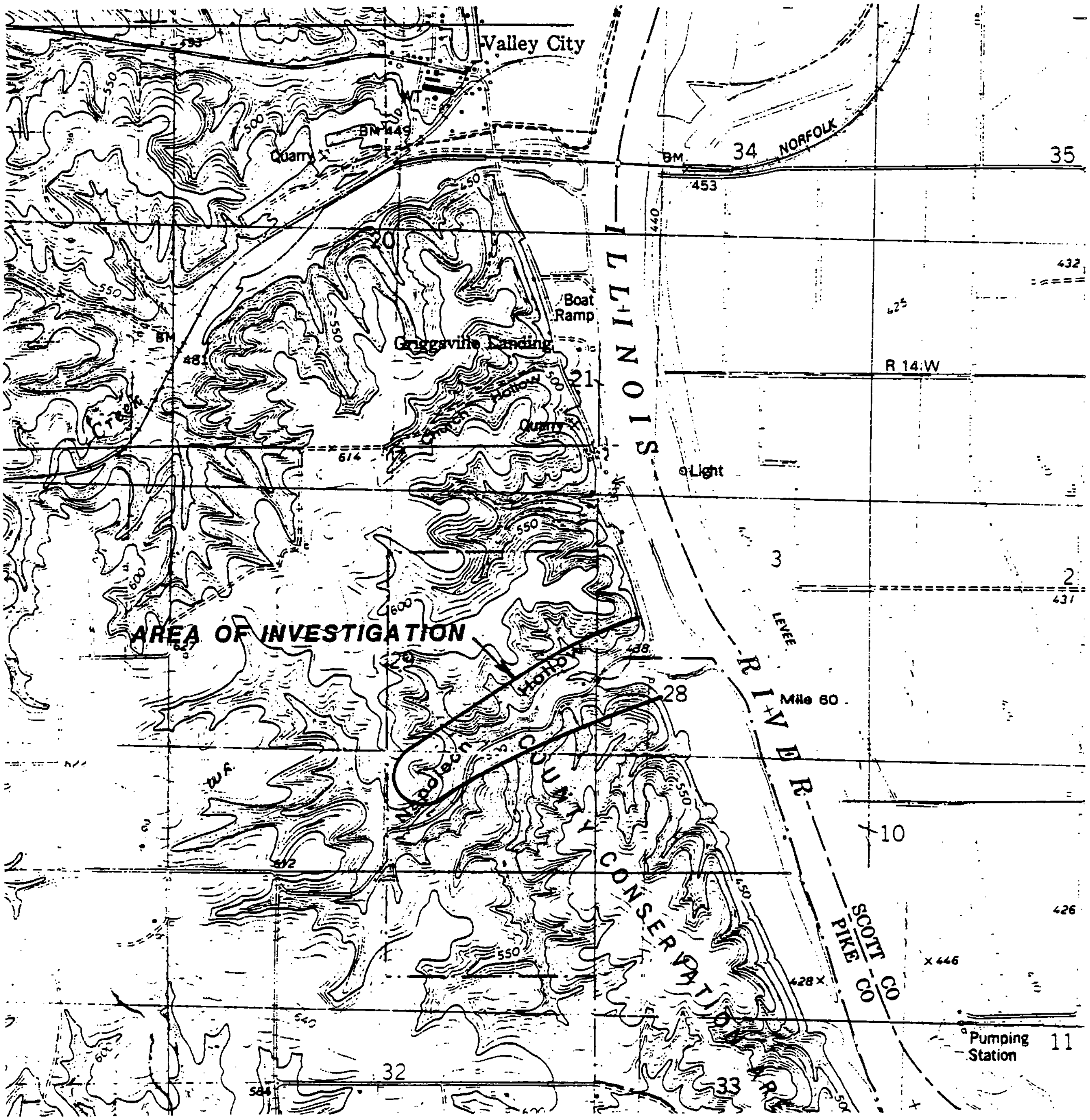


Figure 1. Area of investigation of Napoleon Alignment, FAP Route 408, Pike County, Illinois.

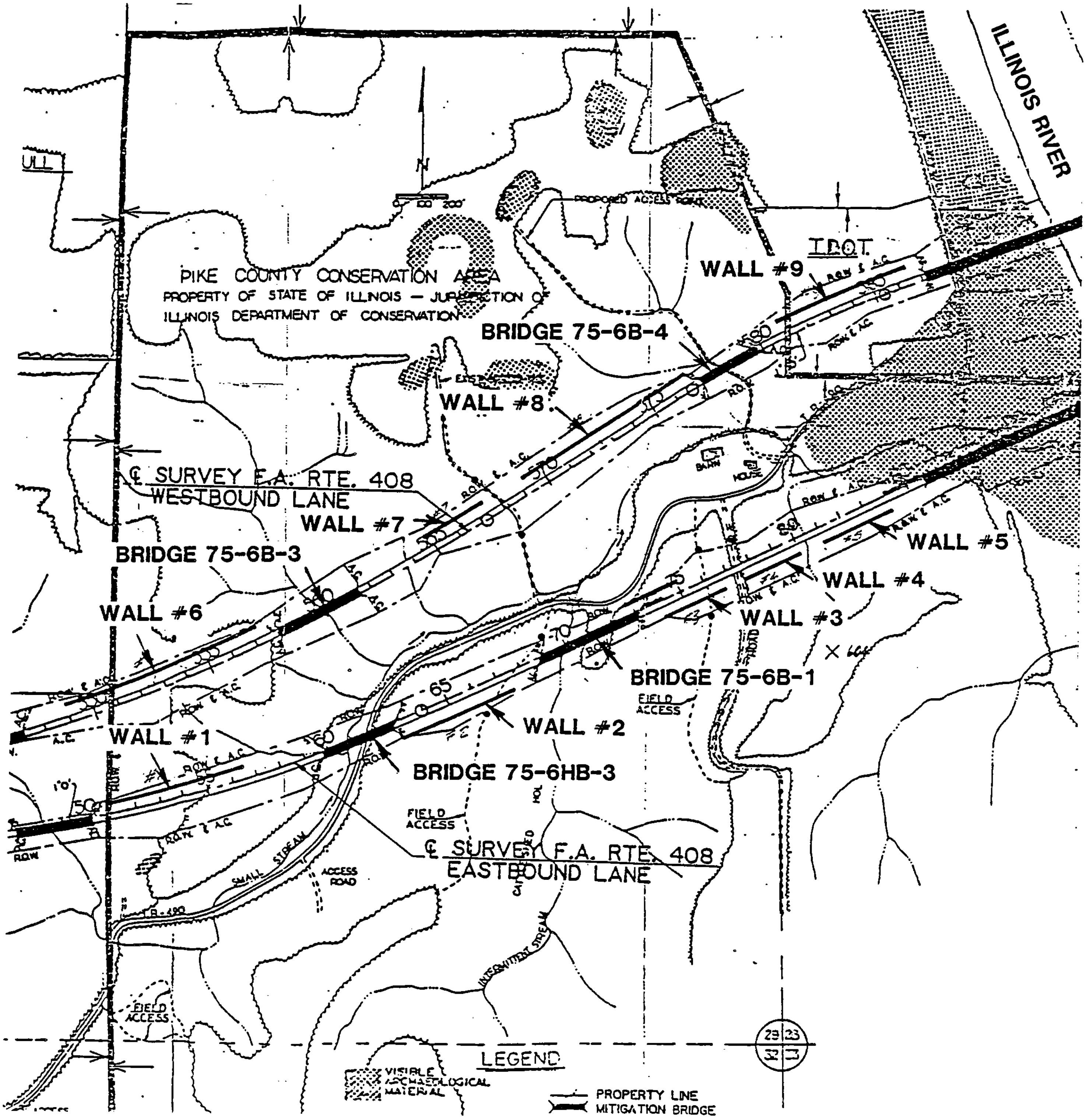


FIGURE 2. Plan map of proposed retaining wall and bridge locations.

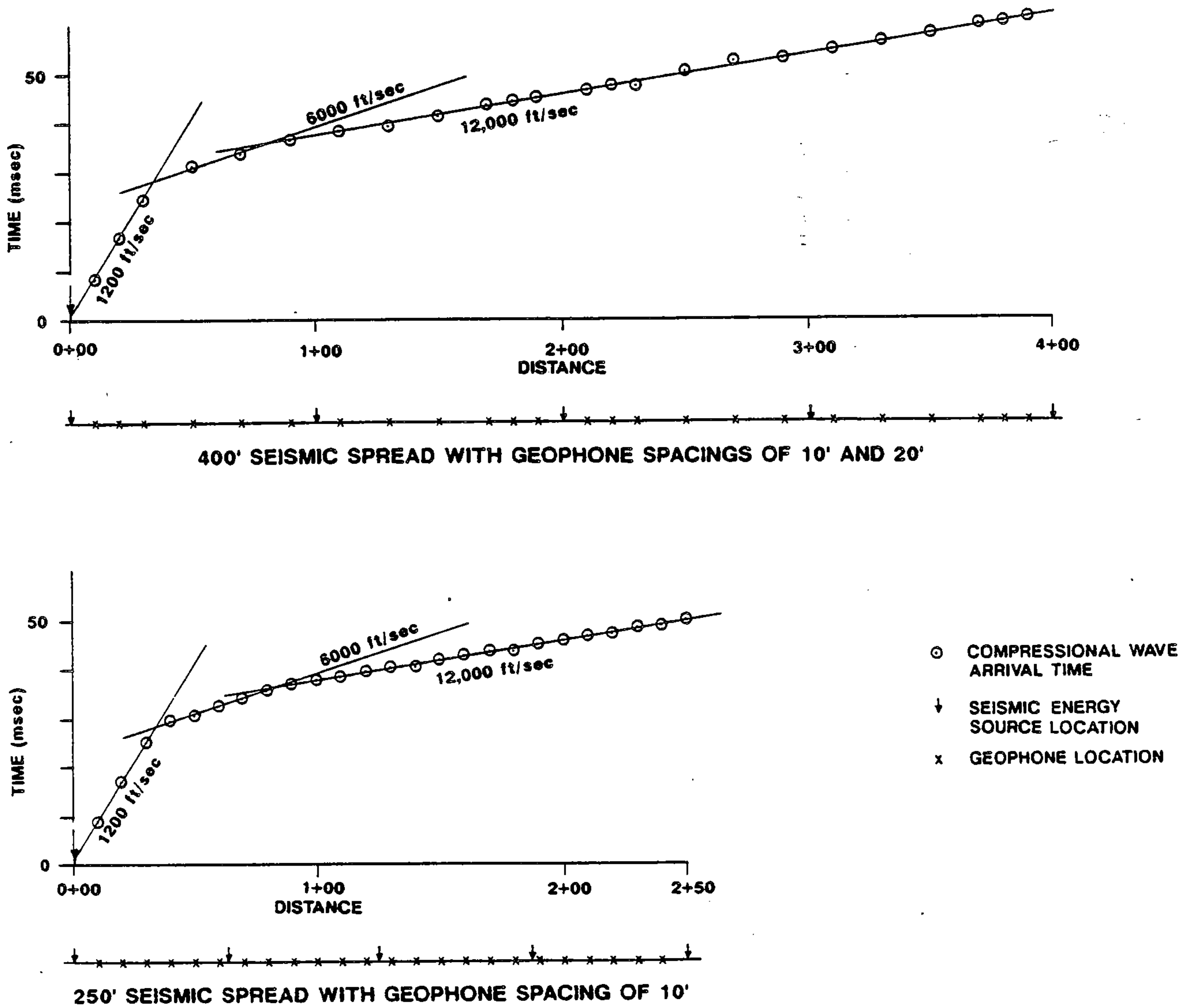


Figure 3. Velocity resolution versus geophone spacing variations.

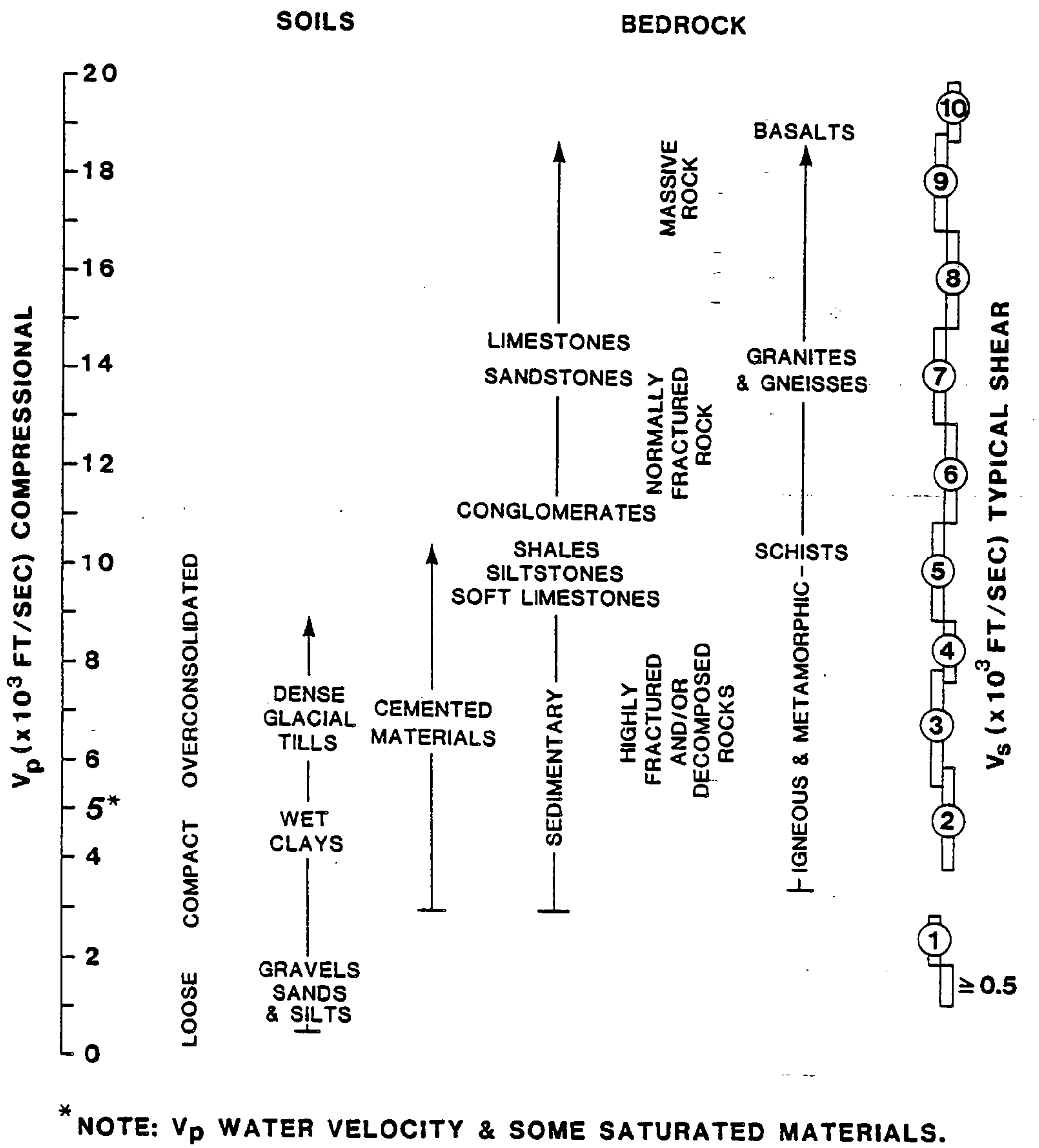


Figure 4. Generalized velocity-material correlations.

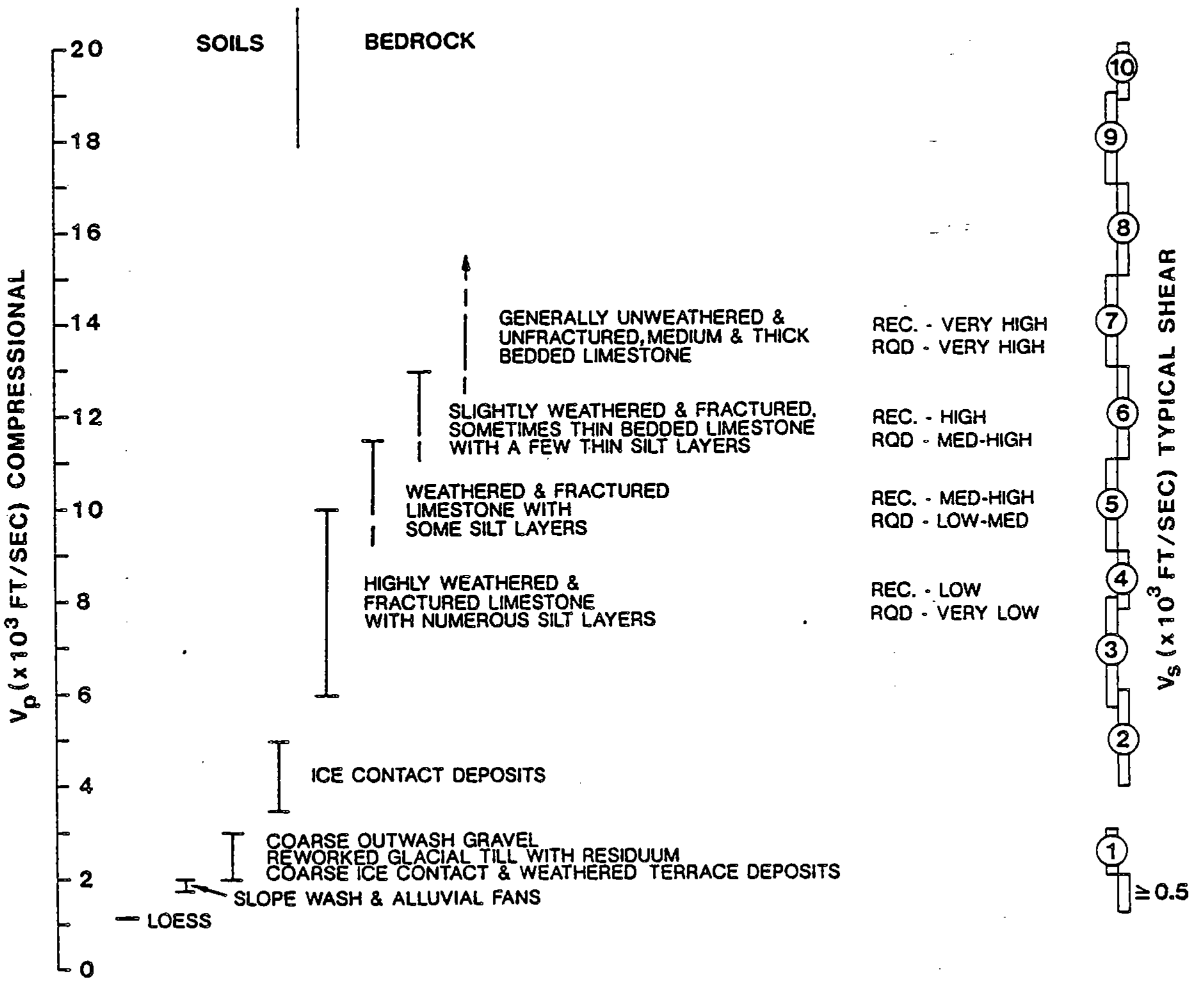


Figure 5. Napoleon Hollow velocity-material correlations.

Photo of thinly bedded limestone at station ±82+00 EBL.

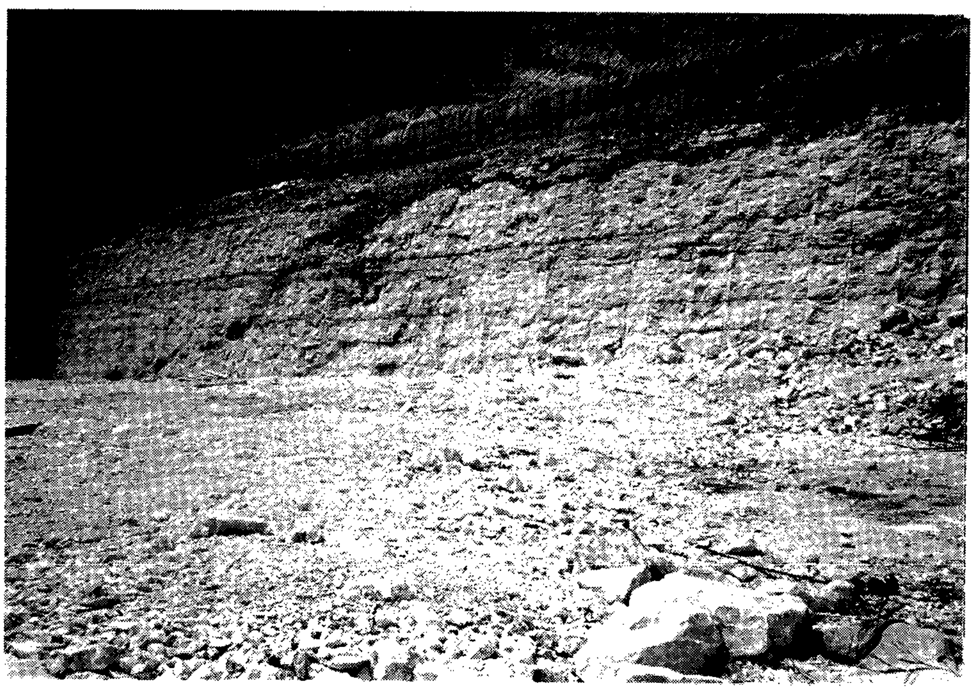


Photo of predominately medium to thickly bedded crystalline limestone behind west abutment WBL (±sta. 596+00).

Figure 6. Photos of velocity measurement locations, eastbound and westbound abutments of the Illinois River Bridge.

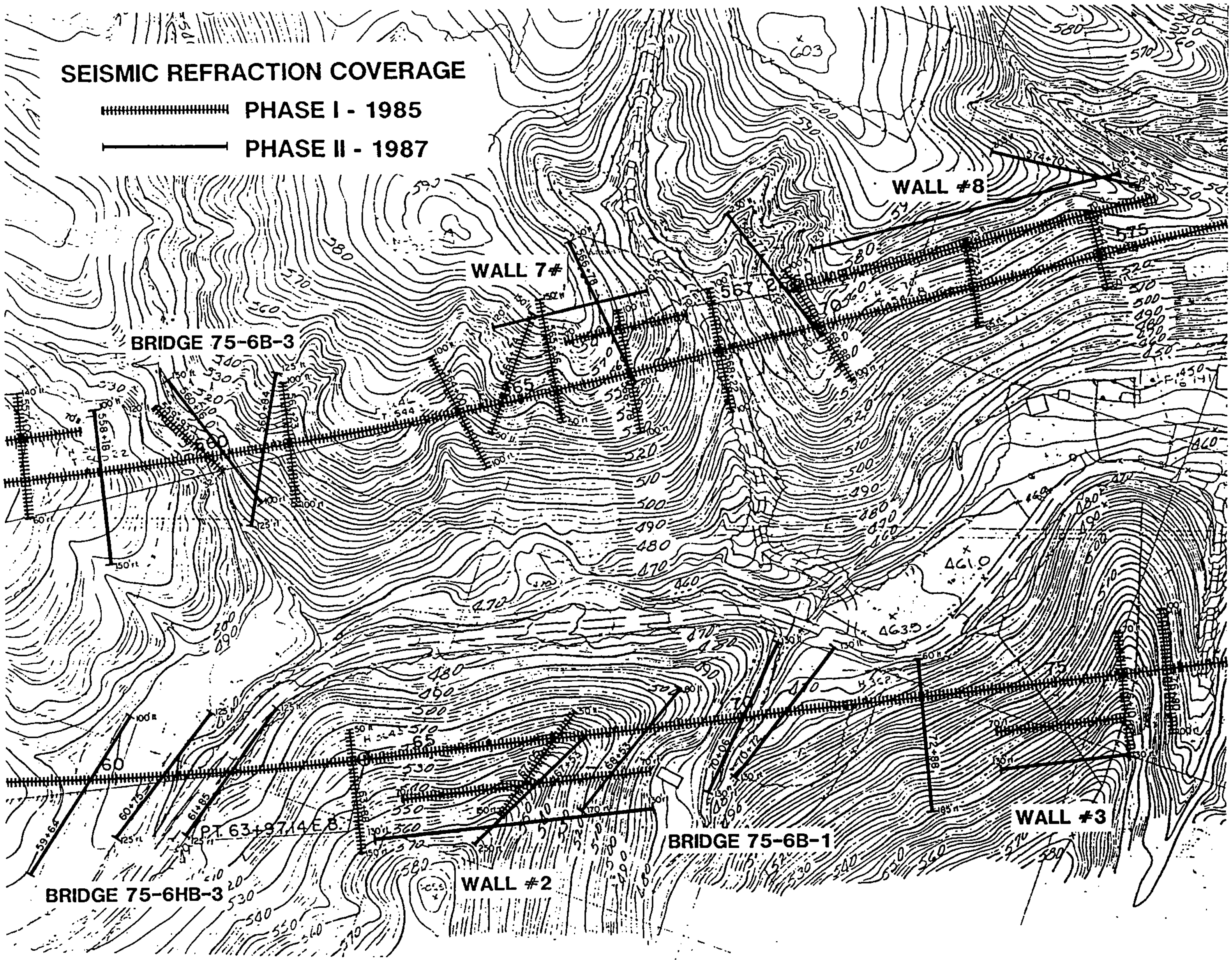


Figure 7. Plan map of Phase I and Phase II seismic coverage at the Wall #2 and Bridge 75-6B-1 areas.

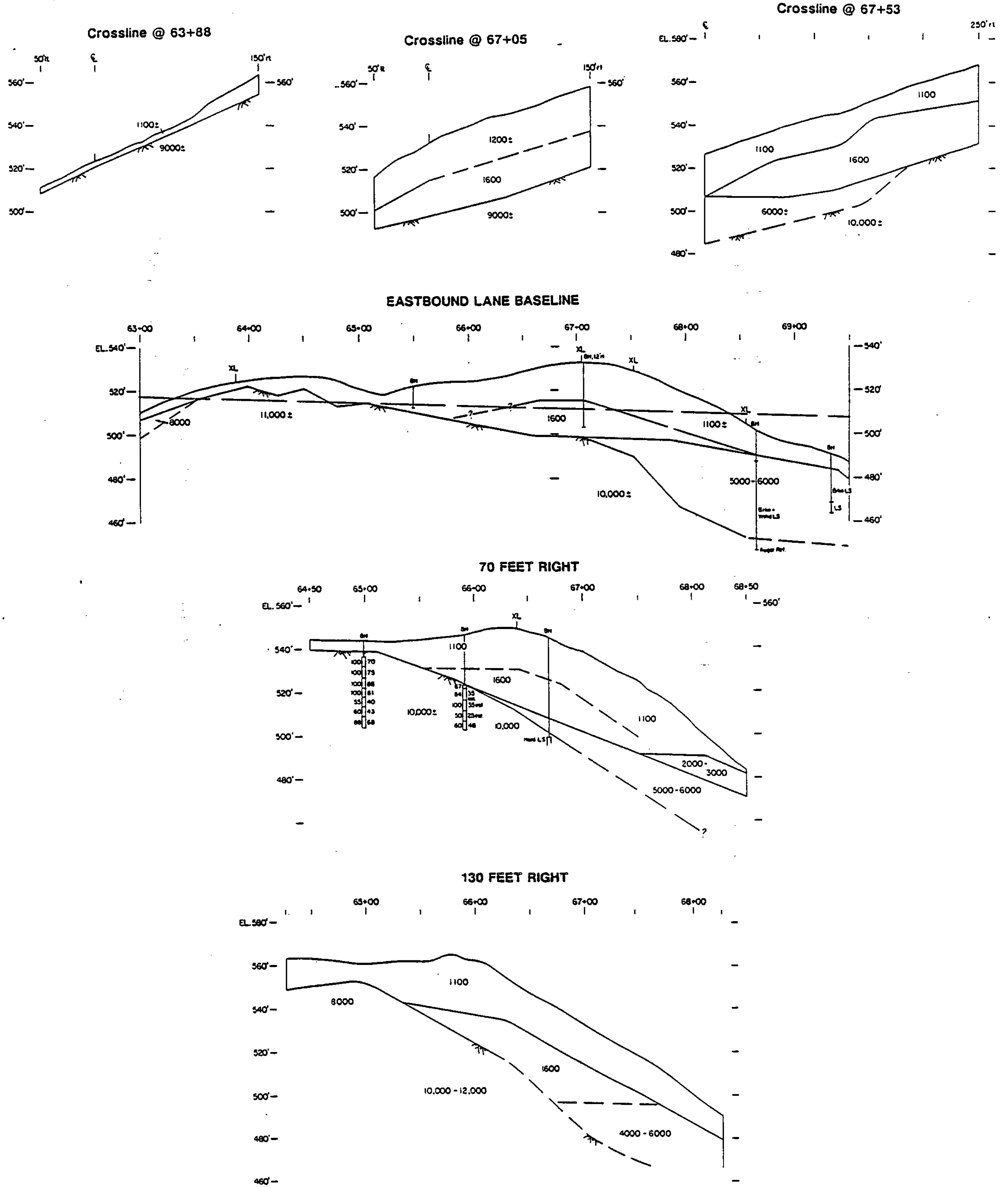


Figure 8. Seismic profiles at the Wall #2 area.

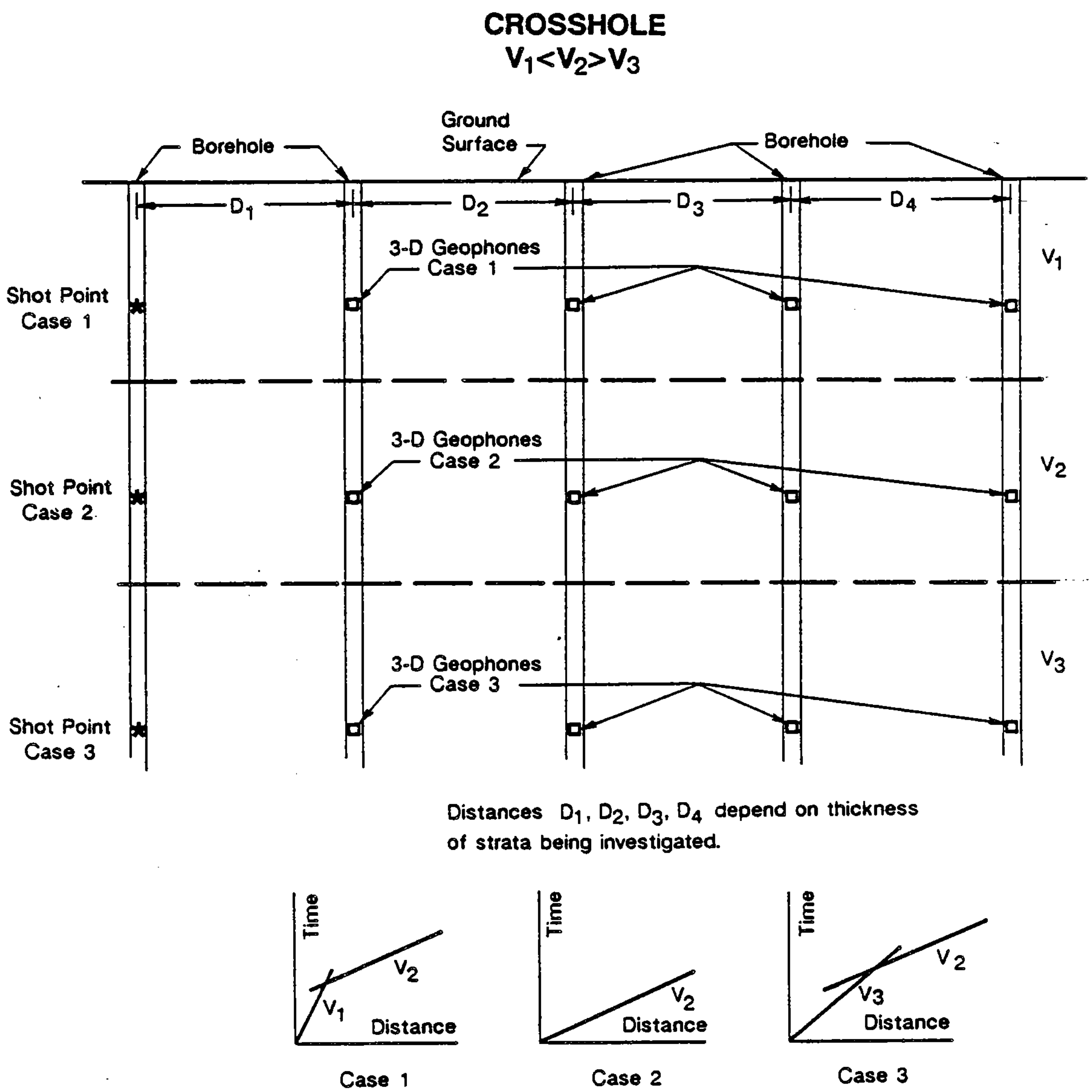


Figure 9. Definition of high and low velocity layers by crosshole velocity measurements.

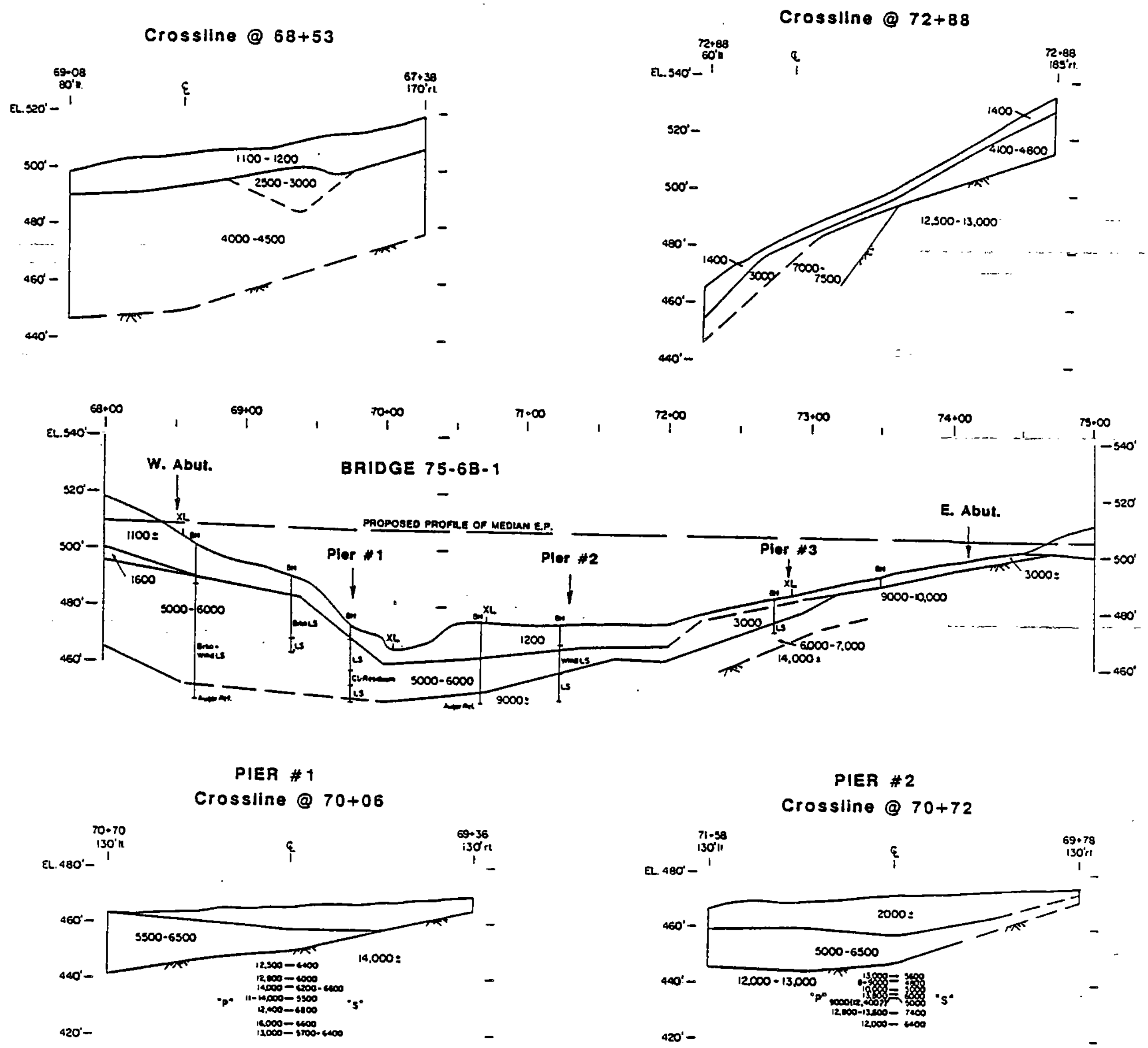


Figure 10. Seismic profiles at the Bridge 75-6B-1 area.

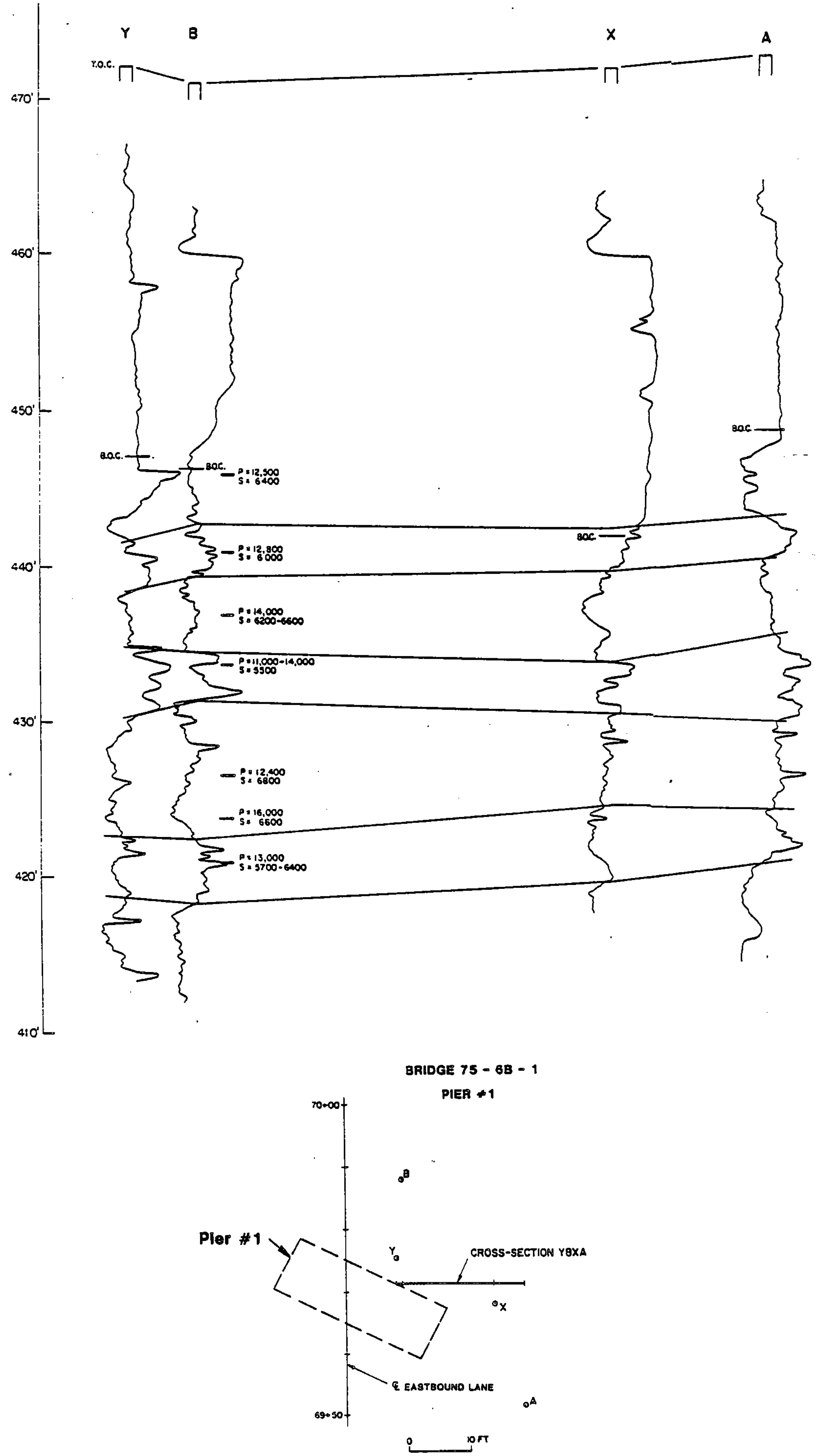
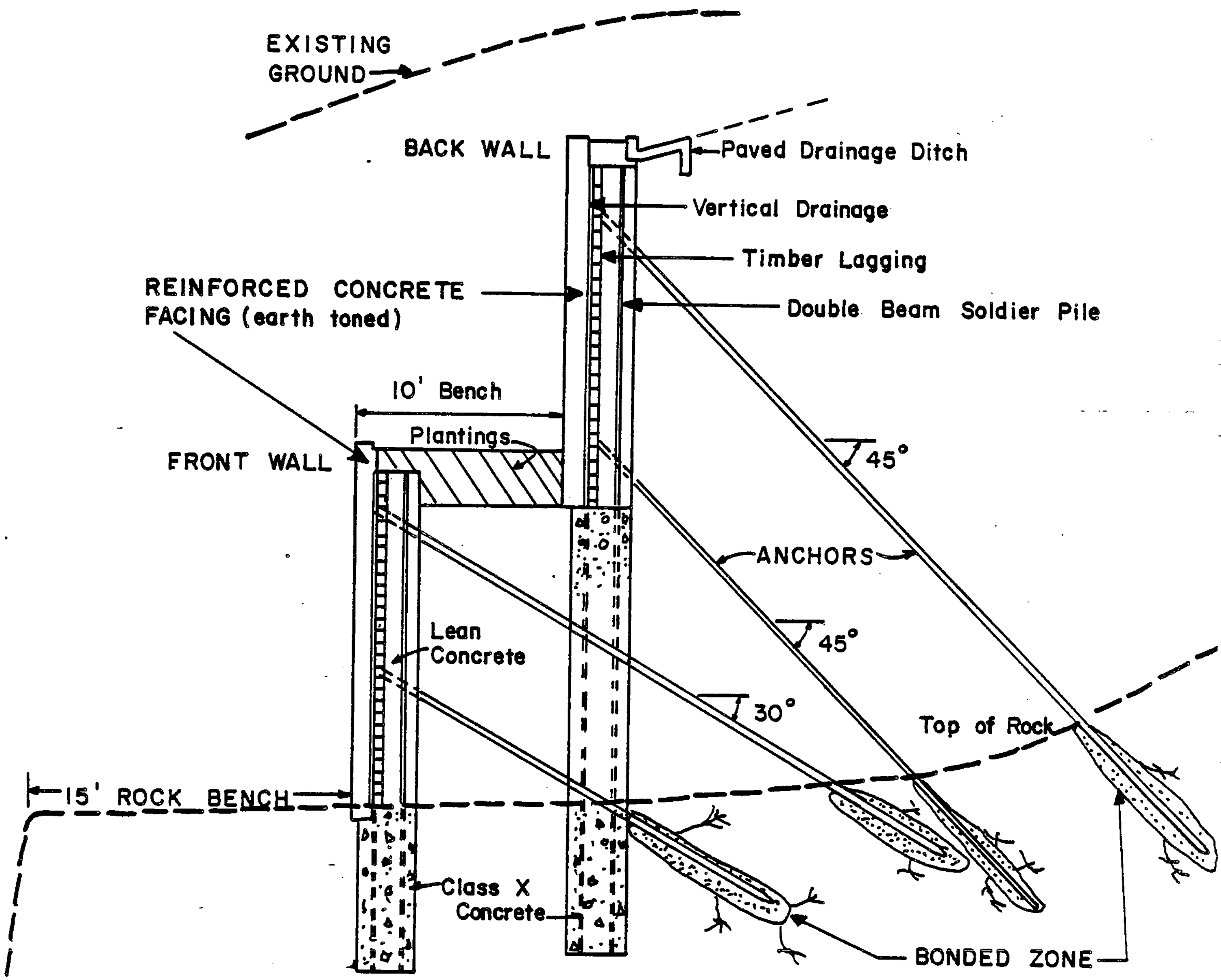


Figure 11. Geophysical logs, borehole location plan and in-situ seismic velocity values for Bridge 75-6B-1, Pier #1.

BRIDGE 75-6B-1
PIER #1 - STATION 69+76, HOLE B

Elevation [ft]	"P" Wave Velocity [ft/sec]	"S" Wave Velocity [ft/sec]	Unit Weight [wet] [lbs/ft^3]	Poisson's Ratio	Young's Modulus [lbs/in^2] [x10^6]	Shear Modulus [lbs/in^2] [x10^6]	Bulk Modulus [lbs/in^2] [x10^6]	Unconfined Compressive Strength
446	12500	6400	150	.32	3.51	1.33	3.29	3350
441	12800	6000	150	.36	3.17	1.17	3.75	2730
437	14000	6200-6600	160	.37	3.87	1.41	4.88	3370
434	11000	5500	140	.33	2.44	.914	2.44	1990
	14000	5500	160	.41	2.94	1.04	5.38	1940
426.5	12400	6800	150	.28	3.85	1.50	2.98	4030
424	16000	6600	160	.40	4.20	1.50	6.84	3960
421	13000	5700	150	.38	2.91	1.05	4.07	2300
	13000	6400	150	.34	3.55	1.33	3.70	3440

Figure 12. Table of seismic velocity and elastic moduli values for Bridge 75-6B-1, Pier #1.



TYPICAL TIEBACK WALL
SYSTEM

FIGURE 13

INTERPRETATION OF SEISMIC REFRACTION DATA ON A MICROCOMPUTER

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ABSTRACT

SIPB, a Seismic Interpretation Program designed to run in Batch mode on mainframe computers, has been used extensively by the Federal Highway Administration for inverse modeling of seismic refraction data. The program was developed by the U.S. Bureau of Mines in 1972 and adapted for use on time-share computers (as SIPT) by the U.S. Geological Survey (USGS) in 1977. About a year ago the USGS modified SIPT so that it could be run on microcomputers (IBM-PC/XT/AT and compatibles), and renamed it SIPT1. A write-up describing SIPT1 and two floppy disks containing the source code (FORTRAN) and executable files are now available as USGS open-file reports.

The modeling approach used by of all of the SIPx programs is essentially the same. A first-approximation 2-D depth model is computed from time-distance data using the delay-time technique, and then the model is refined with three iterations of ray-tracing and model adjustment to achieve a good match between field-measured refraction times and corresponding computer-generated times.

Before running SIPT1 on a microcomputer it is necessary to create a disk-resident input data file for each seismic problem. Two special-purpose programs have been developed by Rimrock Geophysics for this purpose: SIPIN for entering the initial data file, and SIPEDT for modifying the data for follow-on computer runs.

Over the years, SIPB and SIPT have been used successfully for analyzing seismic refraction data used for planning and designing highways in remote areas. Examples are presented that show the results of some of these surveys as reinterpreted on a microcomputer by SIPT2, the upgraded Rimrock Geophysics version of SIPT1.

*Now retired.

INTRODUCTION

Seismic refraction techniques have been used over the past 50 years to determine the depth to rock layers of interest in mineral exploration, ground water, and engineering problems. Musgrave (1967) has assembled a good collection of papers that trace the development of the technique and illustrate its widespread applications. Stephens (1973) has described equipment and techniques applicable to highway engineering problems.

Before computers became available, hand techniques were used to obtain approximate interpretations which generally provided average depths to refraction horizons, and relied on the assumption that the horizons were flat and had constant dip with no curvature or undulations. Later, techniques were developed for modeling undulating refracting horizons. But, without computers, these techniques were cumbersome and impractical to use, and therefore various short-cuts were developed which often compromised the accuracy and reliability of the interpretations.

Computers made it possible and practical to develop sophisticated and rigorous techniques for accurately modeling subsurface features without much concern for the complexity of computations needed, and this approach soon replaced hand calculations for analyzing seismic data. FSIP1 was one of the first seismic refraction inverse modeling programs developed for this purpose (Scott, Tibbetts and Burdick, 1972; Scott, 1973). Although early computers made sophisticated seismic modeling techniques possible, the techniques were somewhat awkward to use because the old machines required that input data be entered on punched cards or punched paper tape, and computer runs had to be made in batch mode with turn-around times of several hours or even overnight. Time-share computers using remote terminals represented a significant advance in simplifying the process of entering and running data, and speeded up the turn-around time. The original Bureau of Mines program, FSIP1, was upgraded and adapted for use on time-share computers by the USGS, and renamed SIPT (Scott, 1977).

Personal computers, now available with large memories, have all of the advantages of time-share mainframes, and in addition they are so inexpensive that they can be used by the engineering geologist in a dedicated mode, making turn-around time almost immediate. Portable PC's operating on battery power sources can even be taken to the field, and interpretations can be made before leaving the field site to be sure that field measurements are sufficiently complete and accurate. In addition, the interactive screen displays available on the PC makes running programs relatively simple and straightforward, even for users who are not knowledgeable or experienced in using seismic interpretation techniques.

SIPT1 is one of several seismic refraction interpretation programs now available for use on PC's. The USGS write-up describing SIPT1 (Haeni, Grantham and Ellefsen, 1987) and the two floppy disks containing the source and executable codes are available as Open-File Reports 87-103-A (text) and 87-103-B (disks) from:

Books & Open-File Reports Section — Phone (303) 236-7476
U.S. Geological Survey
Box 25425, Federal Center
Denver, CO 80225

The USGS version, SIPT1, requires a PC with 640K-bytes of random access memory (RAM), and an 8087 or 80287 math co-processor chip.

The upgraded Rimrock Geophysics version of this same program, SIPT2, also requires 640K of RAM, but will run on a machine with or without a co-processor chip. This version is available from:

Rimrock Geophysics Inc. — Phone (303) 985-2522 or (303) 238-7908
12372 W. Louisiana Ave.
Lakewood, CO 80228

Programs SIPIN and SIPEDT were developed by Rimrock Geophysics to simplify and speed up the task of creating and editing input data files for SIPT1 and SIPT2. These programs, and a demo disk that shows how they work, are also available from the above address.

SEISMIC REFRACTION MODELING WITH SIPT1 AND SIPT2

The first step in using these PC programs is to create an input data file and store it on a disk drive (floppy or hard disk). Each data set consists of control information that specifies how the problem will be run, and positional information for all of the shotpoints (up to 7 per spread) and all of the geophones (up to 48 per spread) for each spread (up to 5 spreads may be linked together in a problem). In addition, the data set must include refraction arrival times that are picked from the field records and tagged with the number of the refracting layer that they represent (the programs can handle up to 5 layers). All of this information must be included in the data file prepared in either free-field or fixed-field format. An example example of fixed-field format is shown in Table 1 on the next page.

Table 1. Example of fixed-field input data prepared for entry to programs SIPT1 or SIPT2.

```
TEST DATA FOR PROGRAM SIPT1 OR SIPT2
1 6 3 0  1.0  2. 2. 0.0  0.0  0.0  0.0  0.0  0.0  0.5 10.0 0 0 0
1  5 12          0.
1  1    910.  -110.     0.   .3   0.   0. 0
1  2    910.    -5.     0.   .3   0.   0. 1
1  3    910.    55.     0.   .3   0.   0. 0
1  4    910.   115.     0.   .3   0.   0. 2
1  5    910.   220.     0.   .3   0.   0. 0
1  1    910.     0.     0. 66.3 3  7.6 1 47.7 2 63.7 3 81.4 3
1  2    910.    10.     0. 68.5 3 16.8 2 40.5 2 61.7 3 79.4 3
1  3    910.    20.     0. 70.9 3 24.1 2 32.1 2 60.1 3 77.4 3
1  4    910.    30.     0. 72.5 3 31.1 2 25.6 2 58.1 3 75.4 3
1  5    910.    40.     0. 74.1 3 37.7 2 16.8 2 55.7 2 73.3 3
1  6    910.    50.     0. 75.7 3 44.5 2  6.0 1 53.3 2 71.7 3
1  7    910.    60.     0. 78.2 3 50.1 2  7.6 1 47.7 2 70.1 3
1  8    910.    70.     0. 80.6 3 56.9 3 16.8 2 40.9 2 69.3 3
1  9    910.    80.     0. 81.7 3 61.7 3 24.1 2 32.0 2 66.9 3
1 10    910.    90.     0. 82.9 3 63.7 3 31.0 2 25.6 2 65.3 3
1 11    910.   100.     0. 84.9 3 65.3 3 37.7 2 16.8 2 64.7 3
1 12    910.   110.     0. 86.5 3 67.1 3 44.3 2  6.0 1 62.1 3
```

After you have created the input data file, the hard work is over and the fun begins. You initiate the execution of either SIPT1 or SIPT2 on your PC system by simply typing in the appropriate program name. The program then prompts you to type in the name of the input data file and to specify its format (free or fixed field). Additional SIPT1/SIPT2 program prompts guide you through the rest of the execution by presenting you with options for manipulating the program flow, controlling the amount of printout, and specifying whether output is sent to your screen, printer, or both. Printing a large volume of output on a dot-matrix printer takes a long time, and so it is usually expedient to suppress all but the most vital printout showing the results of the interpretation.

The program begins by computing the velocity of each of the refraction layers by using the arrival times and layer designations entered in the input data set. Then, after shifting all of the geophones and shotpoints to a sloping datum plane fitted through the geophone positions, the program makes a first-approximation depth estimate of the base of Layer 1 by use of the delay-time technique (Pakiser and Black, 1957). Layer 1 is the near-surface low-velocity layer usually composed of soil or weathered rock.

Next the program checks and refines the position of the base of Layer 1 by tracing rays from each of the shotpoints to all of the geophones receiving arrivals critically refracted along the base of Layer 1. The program then adjusts the model to improve the agreement between the refraction times measured in the field and the corresponding times computed by tracing rays through the initial model. Two more iterations of ray-tracing and model adjustment (total of 3 iterations) are available to further refine the position of the base of Layer 1.

Layers below Layer 1 are modeled in the following manner. First, the overlying layer or layers are mathematically stripped away, and the delay-time method is again used to make a first-approximation estimate of the depth to the base of the next deeper layer. As before, three iterations of ray-tracing and model adjustment are available to improve the agreement between measured and computed arrival times. After this process has worked its way down to the deepest layer, a final iteration of ray-tracing is made from the base of Layer 1 to the bottom of the model in order to make adjustments for anomalous surface conditions. If surface layer velocities are anomalous at a certain geophone (perhaps because of a bad "plant" or locally disturbed or compacted soil), then all refraction arrivals to that geophone from all shotpoints and all layers will come in a little too late or a little too early. The final iteration of ray-tracing removes the effect of such anomalous conditions if they exist, and also trims up the overall accuracy of the model.

ADVANTAGES OF PC PROGRAMS SIPT1 AND SIPT2

We obtained a copy of SIPT1 from the USGS as soon as it became available and were very favorably impressed with it. The primary advantage of the PC programs (SIPT1 and SIPT2) over the original mainframe versions is the ability to use them in the field. In the past, you would shoot your spreads, and then take the data back to the office so that it could be entered and run on a mainframe or minicomputer. If it turned out, as it often did, that you had gotten some "bad" data, or had neglected to acquire everything that you needed, you had two alternatives: either go back to the field site and re-shoot some of the data, or try to limp along without it. The first alternative was very expensive and sometimes impossible, while the second was extremely risky as the interpretations were often made using insufficient information. Needless to say, the second alternative was the one generally chosen.

Using SIPT1 or SIPT2 to interpret your data in the field, however, allows you to shoot one or two shots into a spread of geophones, and then stop and make an analysis before even picking up the cables. If you decide that additional shots are needed, you simply shoot them, and then re-run the analysis after adding the new data to your input file with SIPEDT. As a bonus, this in-the-field computing capability gives you a chance to catch and correct errors in the survey data (distances and elevations) as well.

The time for data entry will vary with the typing ability of the user as well as whether a line editor such as EDLIN, a word processing program, or the SIPIN program is used. Time for entering data with SIPIN is usually about 5 to 10 minutes per spread, which is much less than if you use EDLIN or a word processor. After data entry, it takes only a few minutes to complete a run using SIPT1 or SIPT2, excluding the time required to print the output, which depends on the speed of your printer. If you use the "first-look" capability of screen viewing without printer output, you can review your analysis within 3-4 minutes, while if you print it out, it will take 5-10 minutes. If you use a portable PC in the field, you can have the interpreted results in about 15-20 minutes (barely enough time for your field crew to grab a cup of coffee).

EXAMPLES OF COMPUTER OUTPUT

The first page of computer output from the test data listed previously in Table 1 is shown in Table 2 below. This first page of printout simply displays the input data as read in by the program so that you can check its validity.

Table 2. Example of first page of data printed by PC program SIPT2.

** SIPT2 **

TEST DATA FOR PROGRAM SIPT1 OR SIPT2

control card data

				plot scales			datum point 1		override point 2						
				elev	horiz	time									
sprds	exit	layers	vcards	ft/col	ft/row	ms/col	elev	x pos	elev	x pos	blim	tlim	trace	off	dip
1	-6	3	0	1.0	2.0	2.0	0.0	0.0	0.0	0.0	0.50	10.0	0	0	0

shotpoint and geophone data

spread 1, 5 shotpoints, 12 geophones, xshift = 0.0, xtrue = 0

sp	elev	x loc	y loc	depth	uphole t	fudge t	end sp
1	910.0	-110.0	0.0	0.3	0.0	0.0	0
2	910.0	-5.0	0.0	0.3	0.0	0.0	1
3	910.0	55.0	0.0	0.3	0.0	0.0	0
4	910.0	115.0	0.0	0.3	0.0	0.0	2
5	910.0	220.0	0.0	0.3	0.0	0.0	0

arrival times + fudge t and layers represented

geo	elev	x loc	y loc	sp 1	sp 2	sp 3	sp 4	sp 5
1	910.0	0.0	0.0	66.3 3	7.6 1	47.7 2	63.7 3	81.4 3
2	910.0	10.0	0.0	68.5 3	16.8 2	40.5 2	61.7 3	79.4 3
3	910.0	20.0	0.0	70.9 3	24.1 2	32.1 2	60.1 3	77.4 3
4	910.0	30.0	0.0	72.5 3	31.1 2	25.6 2	58.1 3	75.4 3
5	910.0	40.0	0.0	74.1 3	37.7 2	16.8 2	55.7 2	73.3 3
6	910.0	50.0	0.0	75.7 3	44.5 2	6.0 1	53.3 2	71.7 3
7	910.0	60.0	0.0	78.2 3	50.1 2	7.6 1	47.7 2	70.1 3
8	910.0	70.0	0.0	80.6 3	56.9 3	16.8 2	40.9 2	69.3 3
9	910.0	80.0	0.0	81.7 3	61.7 3	24.1 2	32.0 2	66.9 3
10	910.0	90.0	0.0	82.9 3	63.7 3	31.0 2	25.6 2	65.3 3
11	910.0	100.0	0.0	84.9 3	65.3 3	37.7 2	16.8 2	64.7 3
12	910.0	110.0	0.0	86.5 3	67.1 3	44.3 2	6.0 1	62.1 3

Depending on your response to prompts from the program, various amounts of output information may be obtained, ranging from simply the input data and a time-distance plot, to a complete listing of all calculated data such as intermediate velocity calculations, ray end points, depth tables, and a depth profile plot. An example of a time-distance plot is shown in Figure 1 below, and examples of numerical output are given in Tables 3 and 4. An example of a depth profile plot is shown in Figure 2. Figures 1 and 2 should be rotated 90 degrees counter-clockwise to be viewed in the normal orientation for these types of plots.

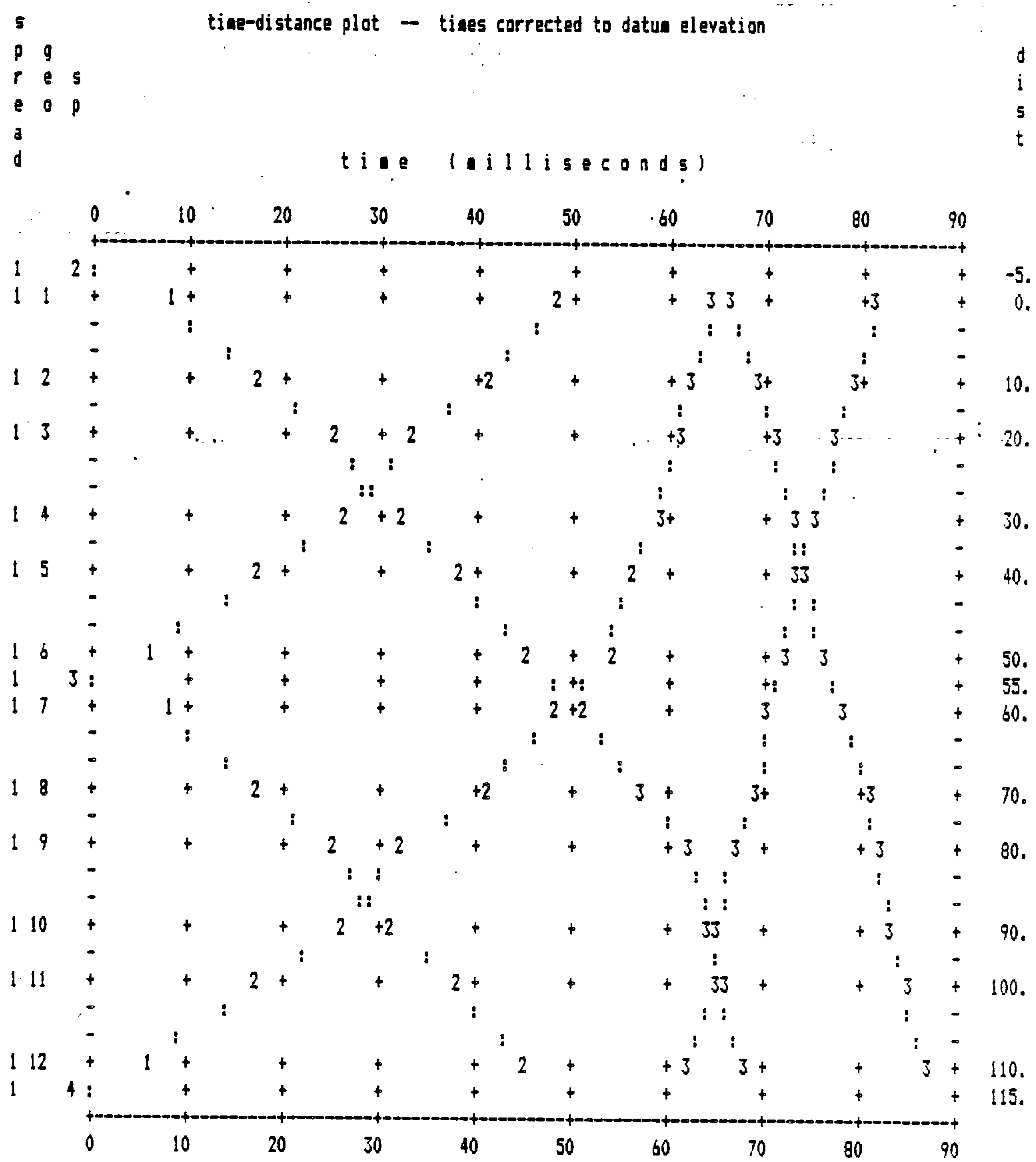

Figure 1. Example of a time-distance plot printed by program SIPT2.

Table 3. Example of preliminary numerical data printed from SIPT2.

TEST DATA FOR PROGRAM SIPT1 OR SIPT2

spread 1 ray end points beneath geophones

geo		sp 1	sp 2	sp 3	sp 4	sp 5
1	pos	-8.5 3	0.0 1	3.0 2	7.6 3	7.8 3
	elev	880.7	0.0	905.0	882.9	882.2
2	pos	3.1 3	7.2 2	12.8 2	17.0 3	17.2 3
	elev	880.3	905.1	905.3	882.9	882.2
3	pos	12.5 3	17.4 2	23.1 2	26.7 3	26.8 3
	elev	879.9	905.5	905.6	882.4	882.0
4	pos	21.3 3	27.9 2	33.0 2	36.1 3	36.2 3
	elev	880.0	905.7	906.1	882.1	881.7
5	pos	29.9 3	38.6 2	42.2 2	42.2 2	46.6 3
	elev	879.6	906.8	906.9	906.9	881.2
6	pos	40.0 3	48.0 2	0.0 1	52.1 2	57.2 3
	elev	879.6	906.6	0.0	906.5	881.0
7	pos	51.1 3	57.9 2	0.0 1	62.3 2	65.3 3
	elev	879.1	906.3	0.0	906.1	880.7
8	pos	59.7 3	60.5 3	67.6 2	72.5 2	76.9 3
	elev	878.8	881.0	905.9	905.9	879.8
9	pos	70.9 3	70.7 3	77.9 2	82.3 2	87.3 3
	elev	878.8	878.1	906.1	906.2	880.0
10	pos	80.9 3	80.5 3	87.8 2	92.4 2	98.3 3
	elev	879.1	877.8	906.2	906.3	879.7
11	pos	91.7 3	91.4 3	97.9 2	101.4 2	109.0 3
	elev	878.7	877.7	906.7	906.7	878.5
12	pos	101.6 3	101.3 3	107.9 2	0.0 1	117.5 3
	elev	879.0	877.8	906.4	0.0	879.2

ray end points beneath shotpoints

			sp 1	sp 2	sp 3	sp 4	sp 5
1=2	right	pos	0.0	-2.1	57.0	0.0	0.0
		elev	0.0	904.8	906.3	0.0	0.0
1=2	left	pos	0.0	0.0	53.3	113.1	0.0
		elev	0.0	0.0	906.4	906.4	0.0
1=3	right	pos	4.4	4.4	0.0	0.0	0.0
		elev	880.0	880.0	0.0	0.0	0.0
1=3	left	pos	0.0	0.0	0.0	107.3	107.3
		elev	0.0	0.0	0.0	880.5	880.5

Table 4. Example of additional numerical data printed from SIPT2.

TEST DATA FOR PROGRAM SIPT1 OR SIPT2

spread 1 smoothed position of layers beneath shotpoints and geophones

sp	position	surf elev	layer 2 depth	layer 2 elev	layer 3 depth	layer 3 elev
2	-5.0	910.0	5.3	904.7	28.8	881.2
3	55.0	910.0	3.6	906.4	30.0	880.0
4	115.0	910.0	3.5	906.5	31.0	879.0
geo						
1	0.0	910.0	5.1	904.9	28.9	881.1
2	10.0	910.0	4.8	905.2	28.5	881.5
3	20.0	910.0	4.5	905.5	28.6	881.4
4	30.0	910.0	4.1	905.9	29.0	881.0
5	40.0	910.0	3.3	906.7	29.0	881.0
6	50.0	910.0	3.4	906.6	29.7	880.3
7	60.0	910.0	3.8	906.2	30.2	879.8
8	70.0	910.0	4.1	905.9	30.8	879.2
9	80.0	910.0	3.9	906.1	31.1	878.9
10	90.0	910.0	3.7	906.3	31.2	878.8
11	100.0	910.0	3.3	906.7	31.2	878.8
12	110.0	910.0	3.6	906.4	30.9	879.1

velocities used, spread 1

	layer 1	layer 2	layer 3
vertical	747.	1478.	
horizontal		1478.	5372.

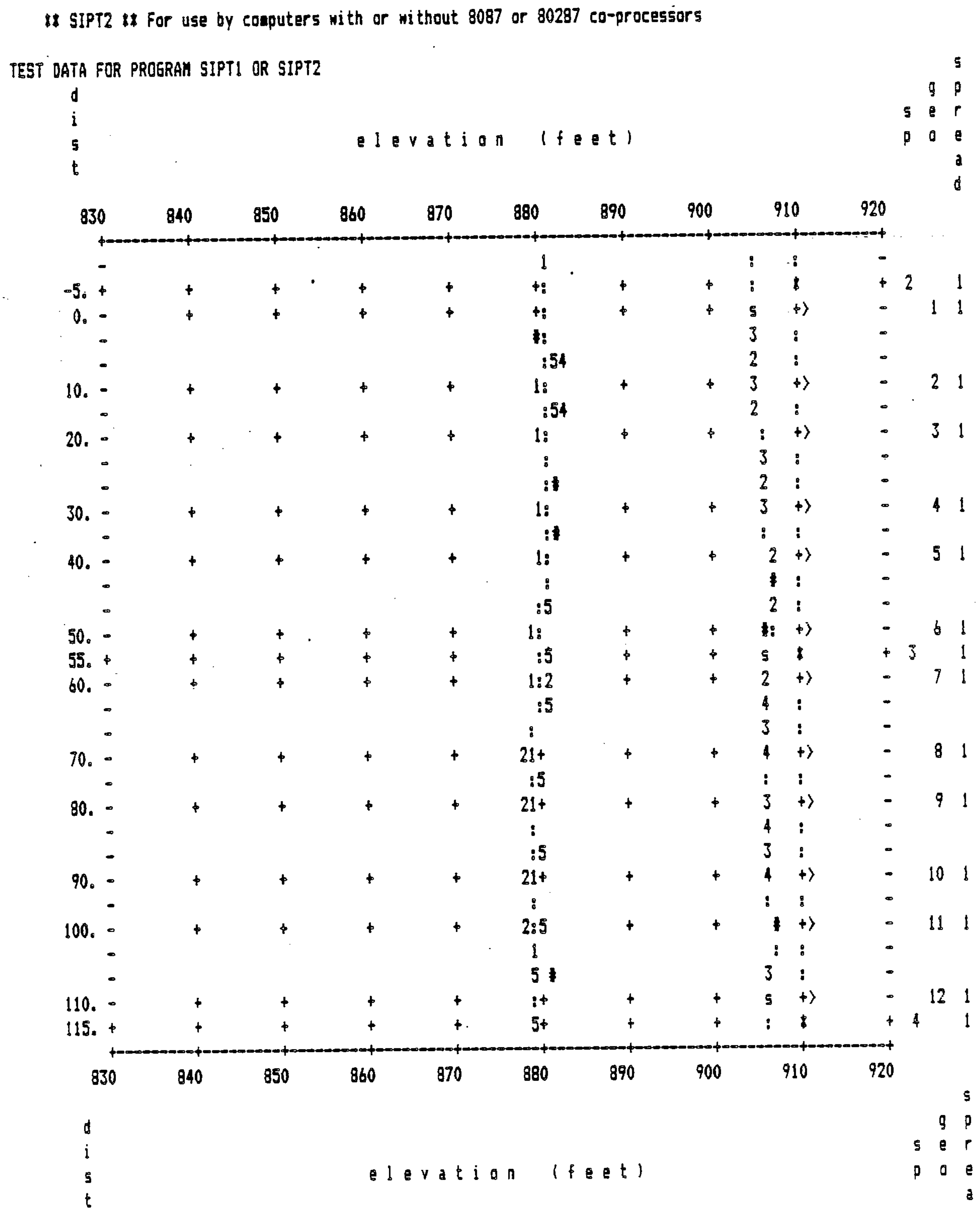

Figure 2. Example of a depth profile plot made by PC program SIPT2. Geophones are indicated by the symbol >, shotpoints by *. Numbers plotted below the surface indicate the positions of emergent ray endpoints from source shotpoints 1, 2, and 3.

EXAMPLES OF HIGHWAY APPLICATIONS

An interesting application of the refraction modeling program for highway planning is illustrated in the following two depth plots. They are taken from a study made by the Federal Highway Administration (FHWA) in the Darien Gap near the northern border of Columbia where it has been proposed that the Pan American Highway be built across a 22-km filled section of the Atrato Swamp rather than along the higher and drier, but much longer (and much more expensive to construct) Choco route (Ludowise, 1986).

Because the area is remote and access is difficult, the FHWA decided to use a portable seismic refraction system to determine the depth and suitability of bedrock as a possible borrow source to obtain fill needed for forming a roadbed across the swamp. A number of seismic lines were run over a hill known as the Middle Loma that was close to the proposed route across the swamp. The Middle Loma is one of three hills called the Lomas Las Aisladas that are thought to be weathered remnants of an igneous intrusion. Results of the seismic study indicated that the weathered bedrock might indeed provide a local and therefore relatively inexpensive source of fill if and when a decision is made to build the road across the swamp.

Figure 3 shows the computer depth plot of the interpreted cross-section for Seismic Line 6 run right across the crest of the Middle Loma in an area of severe topographic relief that would have made drilling impractical. The depth to unweathered bedrock parallels the surface on the right end of the line, but pinches out on the left end. Distances and elevations on the plot are in meters even though the printout says "feet". The program will accept distance and velocity values in any units as long as they are consistent (feet and feet/second, or meters and meters/second).

Figure 4 shows Seismic Line 13 run along the ridge at the north end of the Middle Loma near the proposed route of the highway. Here the thickness of weathered bedrock is less, reaching a minimum beneath the saddle near the right end of the line. Again, elevations and distances are in meters. Seismic Lines 6 and 13 were both shot and recorded with an Oyo PS-10 12-channel portable seismic system.

As was explained previously, the figures showing depth cross-sections should be rotated 90 degrees counter-clockwise so that the geophone symbols ">" appear along the ground surface at the top of the figure. The letter "s" is printed beneath shotpoints where down-going rays are critically refracted along a layer interface. Shotpoint symbols (A, B and C in these figures) are printed where valid up-going rays emerge from the refractor on their way to a geophone. A question mark "?" is printed at the point of emergence if an up-going ray is invalid, that is, if the ray travel time measured by the field survey disagrees with the travel time computed for the model by more than TLim, the time limit specified in the input data file. Of course the travel time or layer number indicated by the field survey could be wrong and the model could be right. Therefore it is wise to check the accuracy of arrival times (and layer numbers) picked from the field records for any rays whose endpoints are displayed as question marks. The pound sign "#" is printed if two or more symbols fall on the same spot.

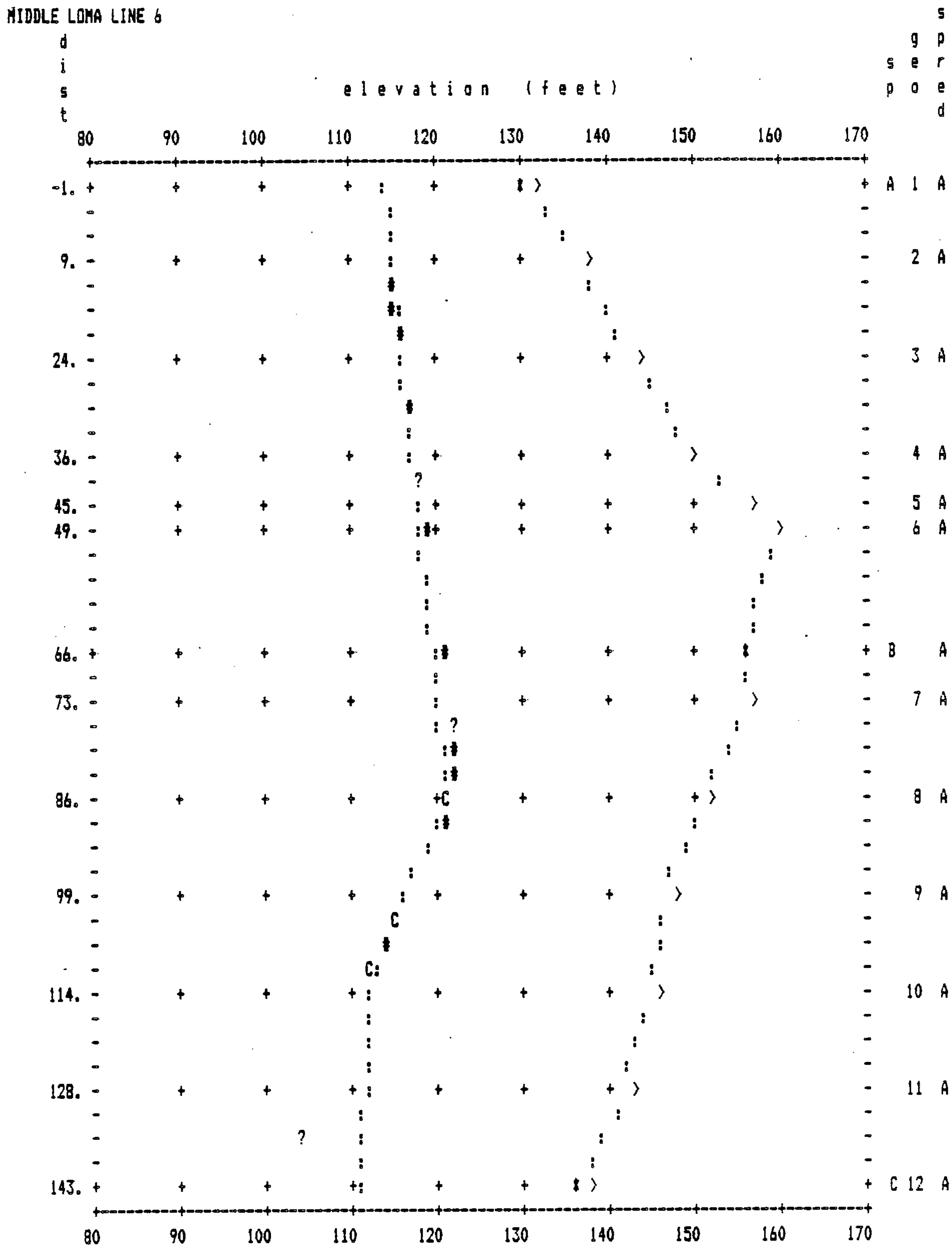

Figure 3. Depth profile for seismic line run over the top of Middle Loma near the proposed Atrato route, Pan American Highway, Columbia.

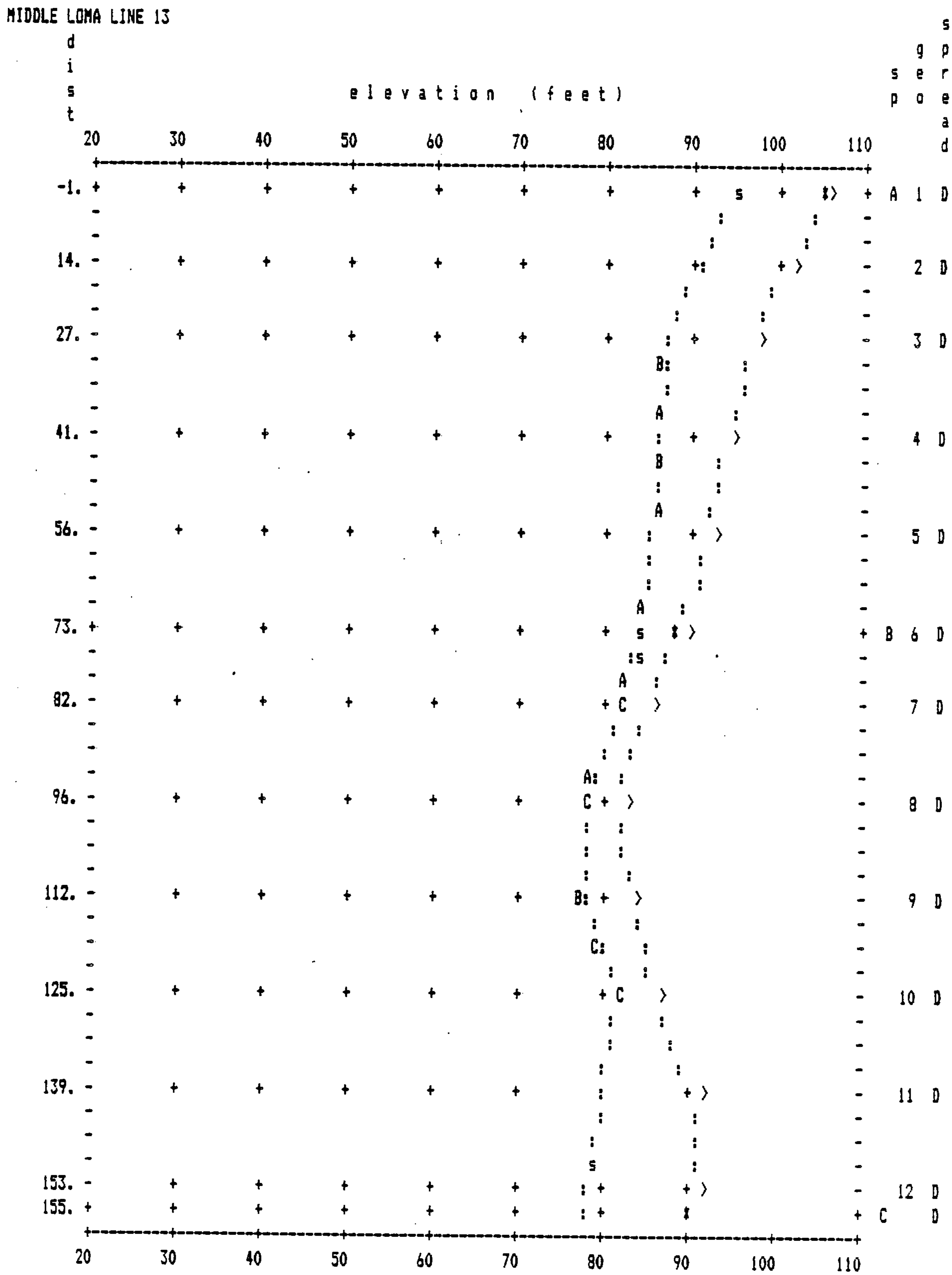

Figure 4. Depth profile for seismic line along the ridge of Middle Loma.

The next two examples are taken from a different environment, and for a different objective in highway planning. In these examples the purpose of the seismic refraction survey was to estimate the depth to bedrock for the construction of retaining walls to support U.S. Highway #101 near Crescent Lake in the Olympic National Park.

Figure 5 shows a two-spread seismic line shot across a small valley near the shore of the lake with thickening overburden beneath the middle and left side of the valley. This line, from Sta. 433+04 to 427+64, was shot with an Oyo PS-10 12-channel seismic refraction system like the one used in the Pan American Highway study made in Columbia.

Figure 6 shows a single spread shot in the same area, but with a light-weight Bison 1550 single-channel sledge-hammer unit. This example illustrates the fact that the seismic modeling program can be used with a single-channel system by simply placing the single geophone where each of the shotpoints would normally be located, and striking the ground at each of the positions along the spread where a geophone would normally be if the line were shot with a multi-channel system. The program can be tricked into handling data from a single-channel system in this manner, making it possible to interpret data from areas where equipment portability is a prime consideration.

Landslides probably constitute the most serious of all potential hazards in highway construction. The next two examples show interpretations of seismic refraction data taken along lines that cross landslides that caused severe road damage in two completely different climatic and geologic environments.

Figure 7 shows the results of a seismic line shot across the Point Lookout Landslide where it damaged the main access road into Mesa Verde National Park in southwestern Colorado. The geologic environment is represented by permeable surface material (weathered colluvium) formed by dry-climate erosion of sandstones and shales, underlain by relatively impermeable Mancos Shale which forms the slip plane of the slide when lubricated by snowmelt or rain water. The direction of the line is transverse to the direction of the slide, and shows the thickening overburden near the middle of the slide.

Figure 8 shows a seismic line shot longitudinally along the length of the Camp Creek Slide that took out part of a logging road just east of Reedsport, south of Winchester Bay along the Oregon coast, in an area of very high rainfall. The geologic setting involves alluvial and glacial fill in a glacially scoured valley. Notice the extreme slope of the ground surface near the left end of the spread. This line was shot with a Bison 8012 12-channel portable system. The results illustrate the capability of the interpretation program to handle steep surface slopes and steep sub-surface dips.

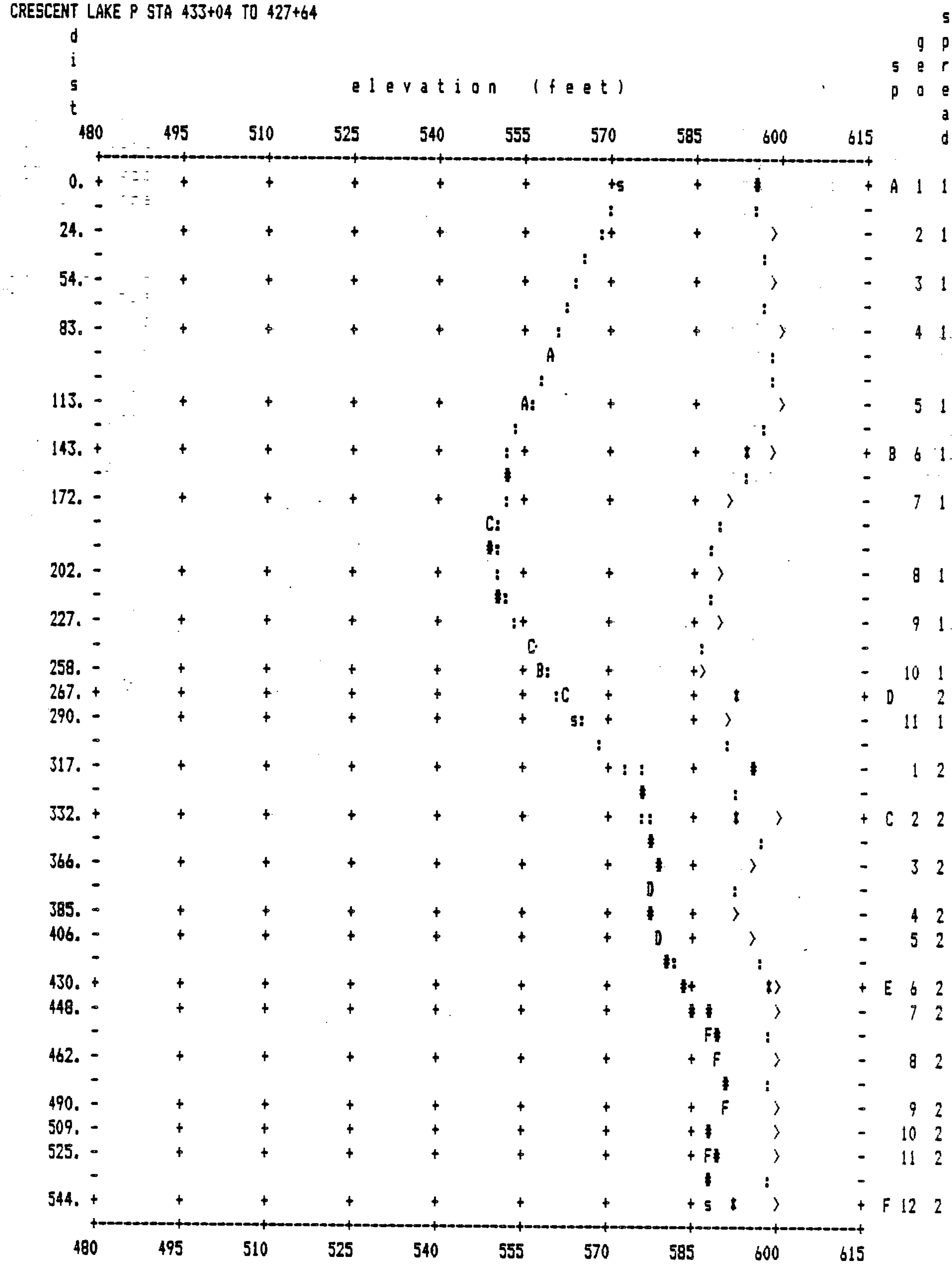

Figure 5. Seismic depth profile along the shoreline of Crescent Lake, WA.

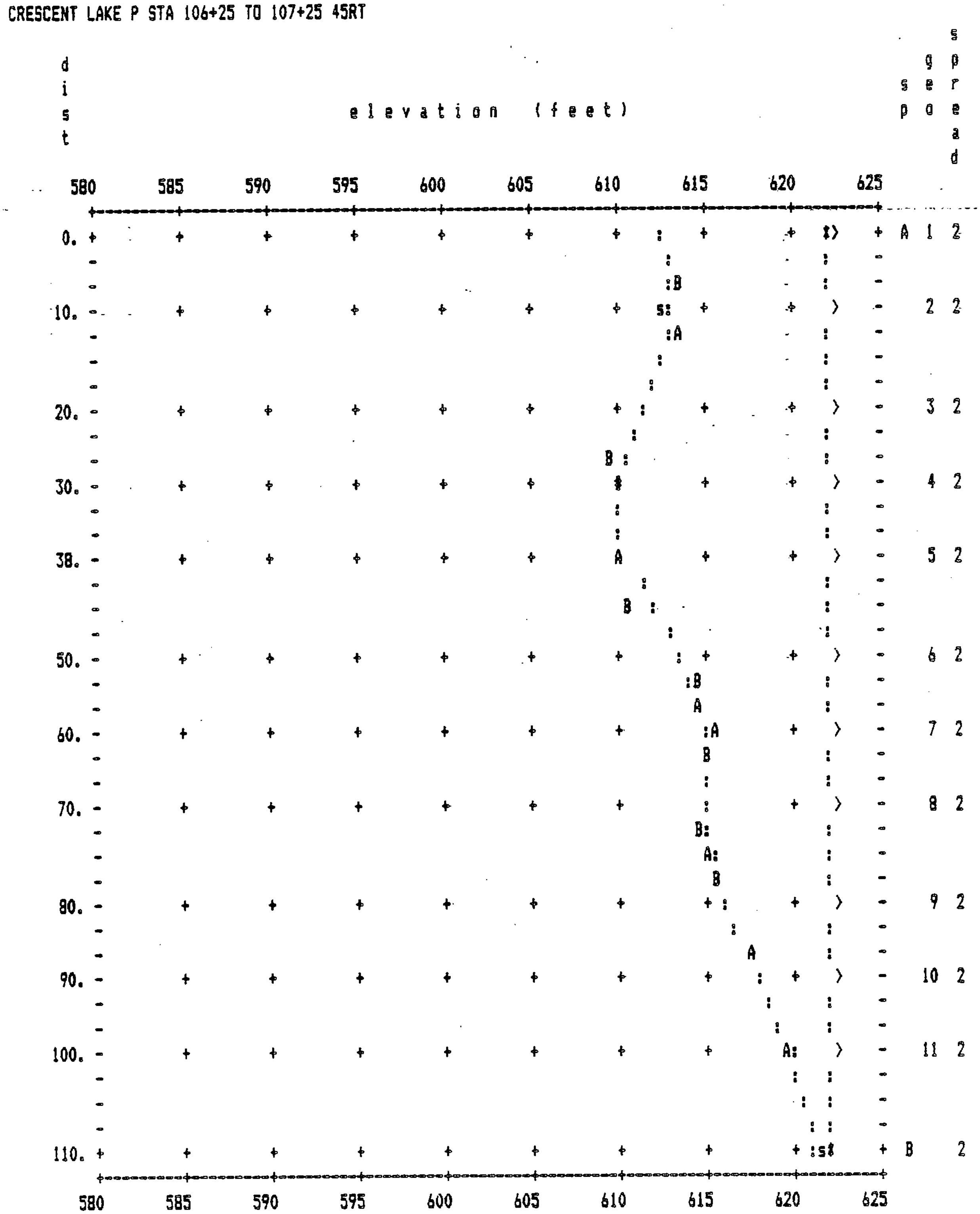

Figure 6. Depth profile for a seismic line run with a Bison 1550 single-channel system along the shoreline of Crescent Lake, WA.

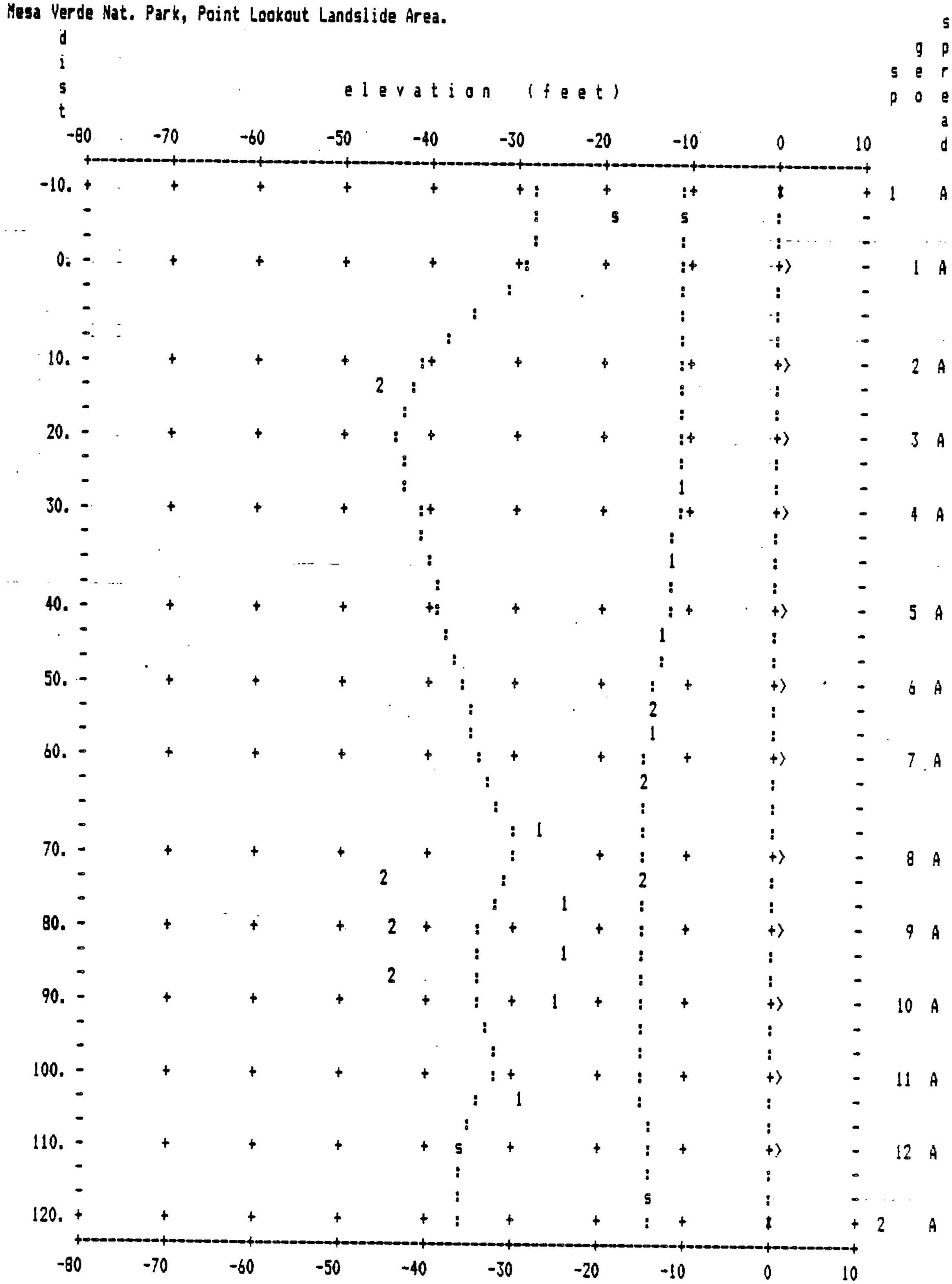

Figure 7. Profile across Point Lookout Landslide, Mesa Verde National Park.

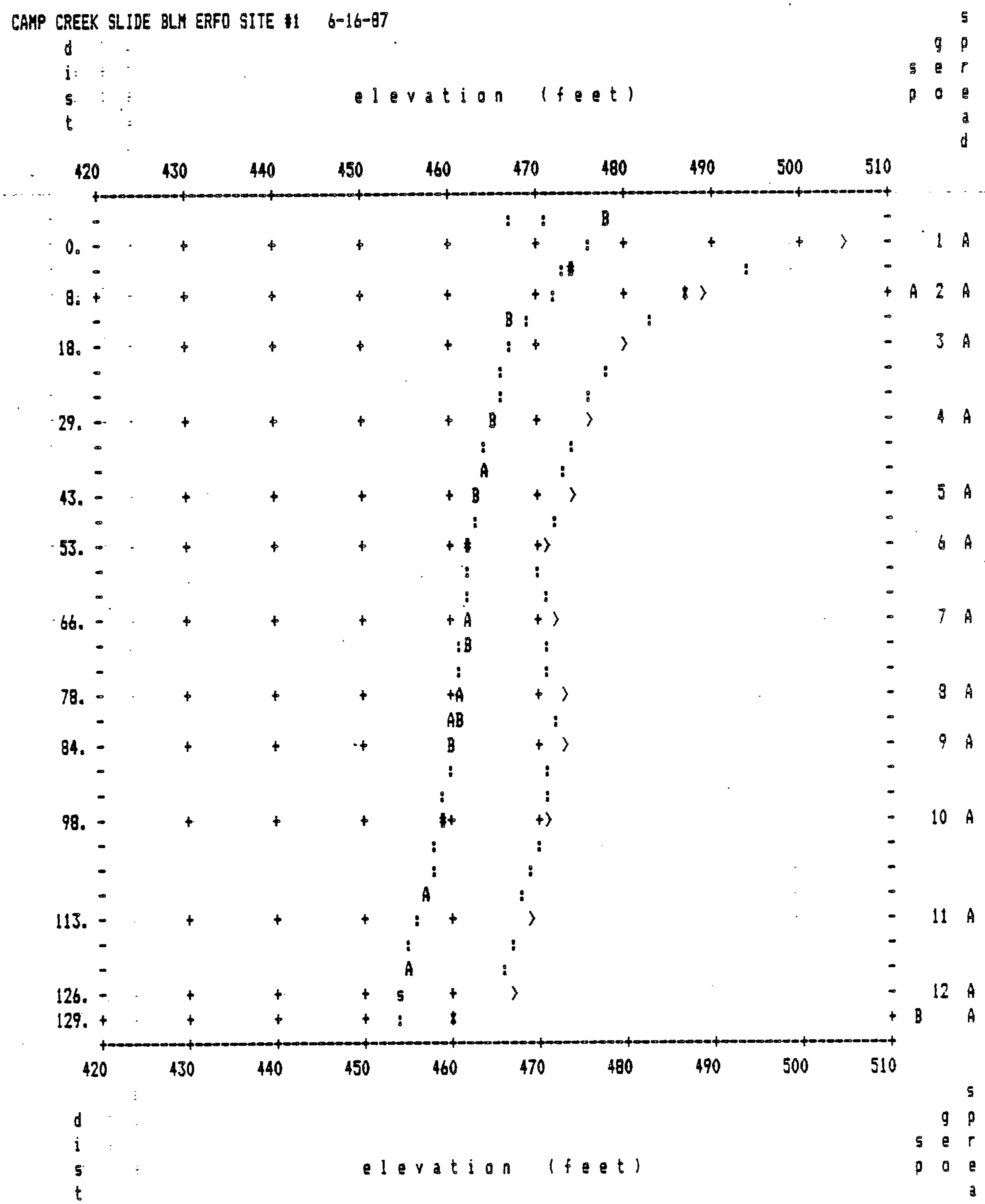

Figure 8. Depth profile along the longitudinal axis of the Camp Creek Landslide east of Reedsport near the Pacific coast of Oregon.

Our final example (Figure 9) is taken from a placer mining study made by the U.S. Bureau of Mines in the Badger Hill Area, Nevada County, California (Tibbetts and Scott, 1971). Although this example illustrates a mining application, not a highway application of interpreting seismic refraction data with the inverse modeling program, we decided to include it because we frequently get questions regarding the accuracy and reliability of interpretations made with SIPT1 and SIPT2, and at this field site, drill holes were put down to check the seismic results. The cross-sections shown in the figure are complex ones comprised of several spreads linked end-to-end across the valley containing a gold placer deposit. The gravels and sands that filled the valley had been partially stripped away by hydraulic mining during the middle 1800's, leaving the ground surface very rough and erratic. The rough terrain and the presence of complex layering (3 sand and gravel layers overlying bedrock) provided a challenging problem for the computer program. Figure 9 shows that, for the most part, the velocity layering interpreted by SIPT2 agrees quite well with the depths determined by drilling (heavy vertical lines and short crossbars).

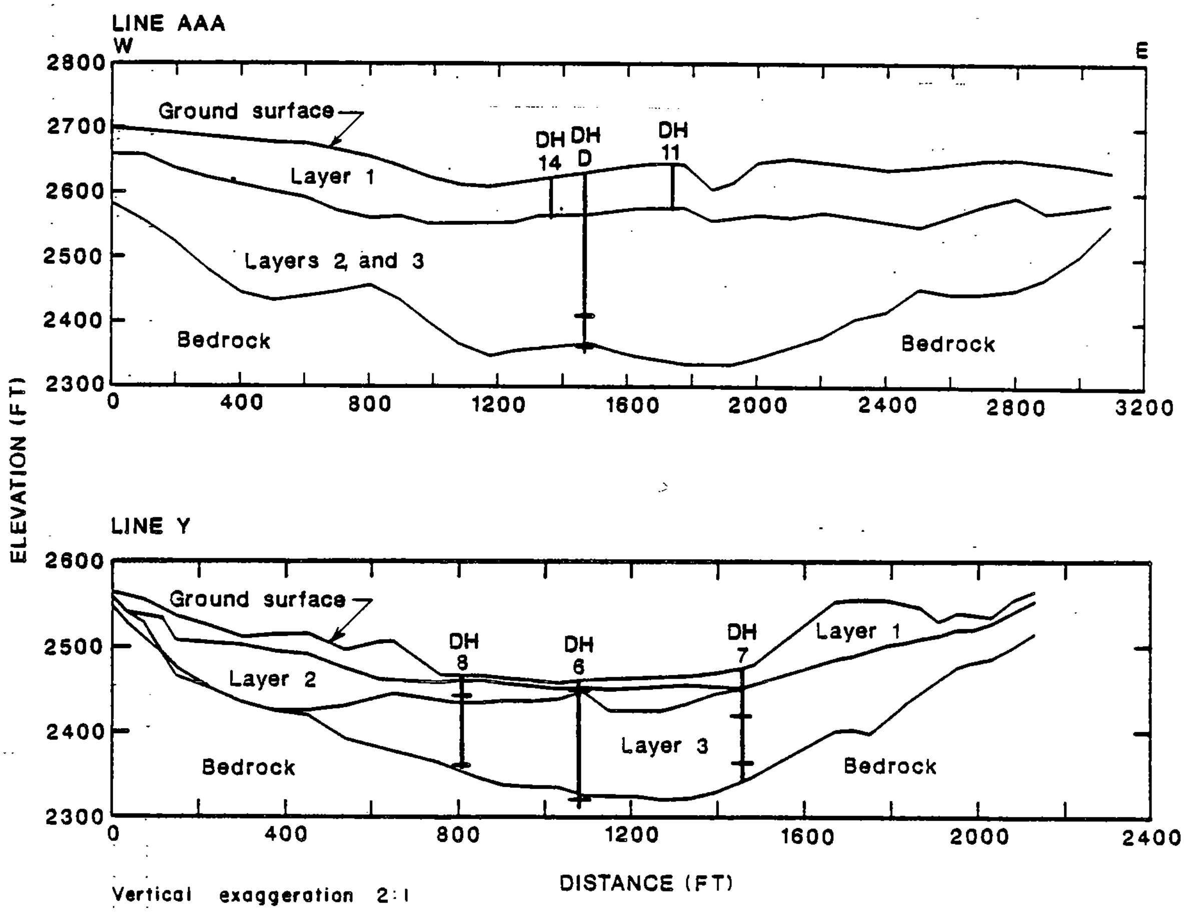

Figure 9. Seismic refraction profiles across the Badger Hill mining area, Nevada County, California (after Tibbetts and Scott, 1971).

CONCLUSIONS

Microcomputer programs SIPT1 and SIPT2, along with auxiliary programs SIPIN and SIPEDT, provide a rapid, inexpensive, and effective way of interpreting seismic refraction data obtained for highway planning and remedial studies, and for other similar shallow subsurface investigations.

ACKNOWLEDGMENTS

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RAPID SHEAR AS AN EVALUATION FOR BASE COURSE MATERIAL

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ABSTRACT

The "Rapid Shear" test is a dynamic triaxial test used to simulate failure loads in a highway base course. In the rapid shear test, a sample six (6) inches in diameter and twelve (12) inches tall is deformed two inches in one second. Chamber pressure is five (5) pounds per square inch.

Eight Arkansas aggregates were tested, two limestones, two sandstones, two gravels, a syenite and a novaculite. Water contents were near optimum for compaction (approximately 70% Saturation) and 100% saturated. Fines contents (% passing the No. 200 sieve) ranged from 6 to 12%.

Rapid Shear strengths ranged from 25 psi for a gravel with 12% fines to 116 psi for a crushed syenite with 6% fines. Increasing moisture content and percent of fines decreased the rapid shear strength. Increasing particle angularity increased the rapid shear strength.

INTRODUCTION

A base course is the layer of material that lies immediately below the wearing surface of a pavement. Since the base course is close to the surface, it must possess high resistance to deformation in order to withstand the pressures imposed upon it.

The "Rapid Shear" test was selected to evaluate base course aggregates as a measure of their dynamic load behavior. The rapid shear test is a triaxial test in which a 6 inch diameter by 12 inch high compacted specimen is deformed two inches in one second. A five psi chamber pressure was used to simulate the confining pressure which typically exists in a highway base course.

BACKGROUND

Fines content, moisture content and density all influence the stability of a base course. According to Barksdale (1984), base course materials compacted to low densities will undergo more rutting than the well compacted sample. Rutting is related to density but the mechanism which accounts for rutting appears to be primarily shear distortion rather than densification. Yoder and Witczak (1975) noted that pavement deformation is a combination of densification (volume change) and repeated shear deformation (plastic flow with no volume change). Protection against excessive deformation, resulting from densification, is insured by proper compaction.

Moisture sensitivity is basically controlled by the quantity and characteristics of the fines and plasticity. Yoder and Witczak (1975) suggested that use of a more open-graded aggregate base would decrease moisture sensitivity and would drain water at a quicker rate (increased permeability). Krebs and Walker (1971) found that the presence of water in the base course may decrease the strength by reducing the cohesive properties of the fines and by reducing the friction between aggregate particles.

Marshall Thompson used the rapid shear test in 1984 as an indicator of moisture sensitivity in base courses. Thompson found the shear strength in the crushed stone base material from the AASHO Road Test was reduced from 222 psi to 79 psi when moisture content was increased from 4.6% to 7%. Because of his successful use of the test, the rapid shear test was selected to investigate the effects of fines of base course aggregates in general use in Arkansas.

SAMPLE PREPARATION

Compaction was done by vibration with an MTS machine. The samples were prepared in five layers. Each layer was vibrated at a frequency of 10 HZ for two minutes. Details of the MTS operation and compaction device can be found in the M.S. thesis by Mr. Bashar Qedan (1987).

Prior to testing, each specimen was conditioned by being subjected to 200 load repetitions and a confining pressure of 15 psi. Normally the conditioning load had a deviator stress of 20 psi.

TEST RESULTS

Eight aggregate sources (Table 1) were tested in rapid shear: six crushed stones, one crushed gravel and a bank gravel. All of the aggregates are from active sources of base material in Arkansas.

Rapid shear strength drops as the percent of fines increases (Figure 1). For most aggregates the drop was about 20 psi when fines were increased from 6% to 12% by weight. Complete results of the testing are contained in the final report of the research (Thornton and Elliott, 1988).

Aggregate type had a much bigger effect on rapid shear strength than the fines (Figure 1). Crushed stone aggregates were strongest followed by crushed gravel and then bank gravel. At 6%, fines strength ranged from 115 psi for the crushed syenite gravel to 42 psi for the bank gravel. The relatonship of strength to aggregate type was probably due to particle angularity.

Differences in rapid shear strength for the different type aggregates were also partly due to a vacuum created in the sample by testing. In the bank gravel, the vacuum averaged less than 1 inch of mercury while it averaged just over 12 inches of mercury in the crushed syenite. The internal

Table 1

Specific gravity, maximum density and optimum % water*

Material, Source	spec. gra.	max. density	opt. % water
Sandstone, Freshour	2.60	134.8	9.6
Limestone, Midwest	2.68	143.8	6.5
Novaculite, State	na	133.7	7.9
Crushed Gravel, #3	2.53	137.8	5.8
Limestone, Anchor	2.67	137.4	7.8
Sandstone, Duffield	2.57	138.4	7.0
Bank Gravel, Boorhem	2.53	135.8	5.7
Syenite, Granite Mount.	2.62	137.0	6.7

* Data furnished by the Arkansas Highway & Transportation Department

friction, computed using the peak load during testing, the confining pressure, and the vacuum, averaged 43 degrees for the bank gravel and 55 degrees for the crushed syenite.

In order to find the effect of water content, rapid shear strengths were plotted against the corresponding specimens' percent of fines. Contours of constant water contents were drawn to illustrate the effects of the variables.

Added moisture reduced the rapid shear strength of the aggregates (Figure 2). Delta 3, Duffield sandstone, Mid State novaculite, and Boorhem Field gravel all had reduced strengths throughout the range of fines content tested. The other aggregates were not as sensitive to moisture content when fines contents were below 8%. All of the aggregates had lower rapid shear strengths at high water content when the fines content was 12%.

CONCLUSIONS

The following conclusions are based on the findings of the rapid shear strength study of Arkansas aggregates.

1. The rapid shear strength of granular materials decreased with increasing fines content.

2. The rapid shear strength of granular material was affected by aggregate type. At a given confining pressure and density, the shear strength increased with increasing particle angularity and/or surface roughness.

3. Decreasing water content from very wet condition to the optimum water content increased the rapid shear strength.

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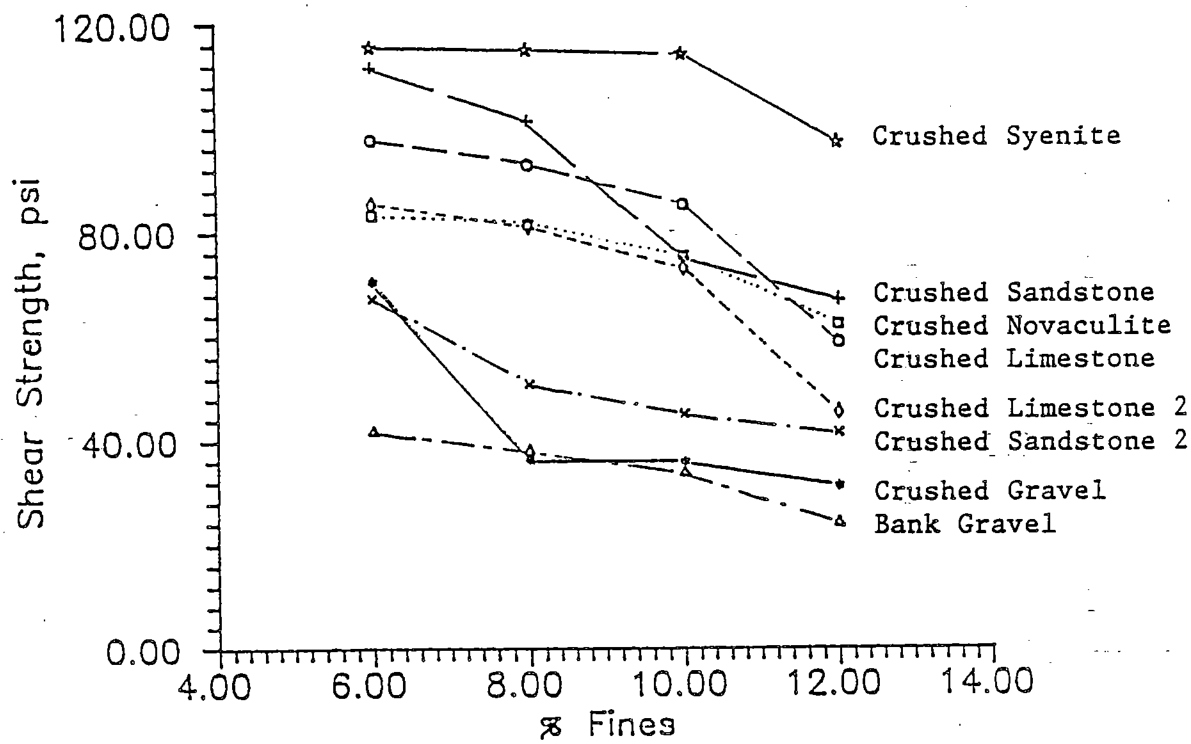

Figure 1. Rapid Shear Strength vs. % Fines

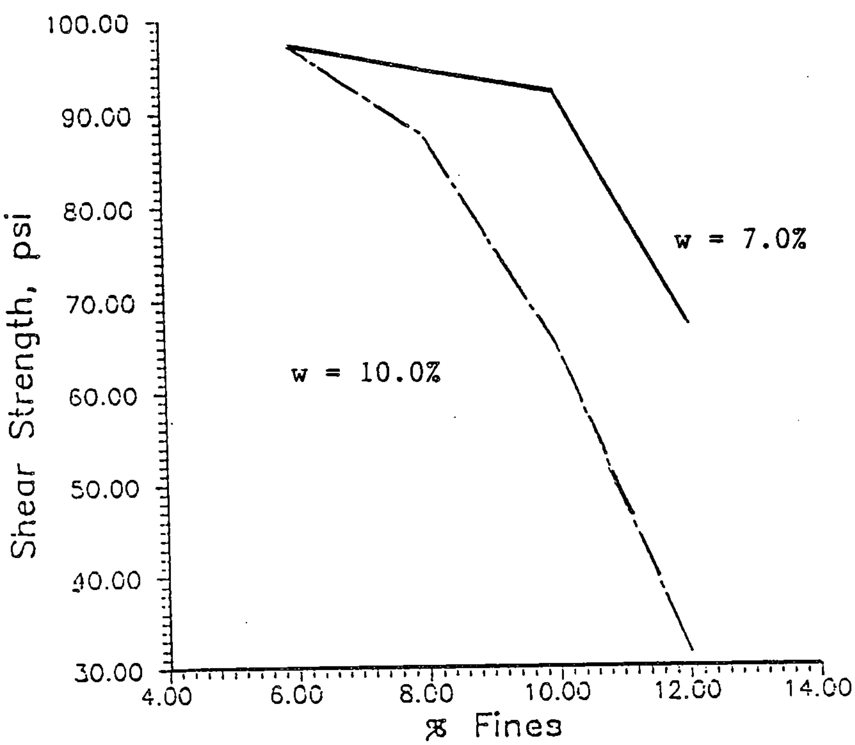

Figure 2. Moisture and Fines Effect on Crushed Limestone

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