PROCEEDINGS OF THE 44TH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

MAY 19-21, 1993
Tampa, Florida

CO-SPONSORED BY

Department of Civil Engineering, University of South Florida

and

Florida Department of Transportation

CHAIRMAN OF LOCAL ARRANGEMENTS

Pamela Stinnette
Department of Civil Engineering
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4202 E. Fowler Ave., ENG 118
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Greetings and Welcome

We're pleased that you have chosen to join us for the 44th Annual Highway Geology Symposium. It is our hope and endeavor that the next few days in the "Sunshine State" and the exchange of information at the Symposium will prove to be both professionally rewarding and personally enjoyable.

The Symposium has been co-sponsored by:

The University of South Florida,
Department of Civil Engineering and Mechanics:

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A special thanks to Mr. Holton Harders, H. G. Harders and Son, Panama City, Florida, for his help and providing the negative from which the cover photograph was produced.

We give our wholehearted thanks to the following sponsors. Their sponsorship has helped make the 44th Annual Highway Geology Symposium a real success.

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Brugg Cable Products Inc.	Robert Thommen	Field Trip Lunch - Thursday
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44TH HIGHWAY GEOLOGY SYMPOSIUM
MAY 19-21, 1993

TAMPA, FLORIDA

Program: TUESDAY, 18 MAY - 5:00 to 7:00 PM Registration (Holiday Inn)
WEDNESDAY, 19 MAY - 8:00 to Noon Registration (Holiday Inn)

10:00 to 10:30 AM Coffee Break, Sponsored by Jim Stidham & Associates

[1] Unable to attend.

Highway Geology Symposium

HISTORY ORGANIZATION AND FUNCTION *

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on February 16, 1950, in Richmond, Virginia. Since then 39 consecutive annual meetings have been held in 26 different states. Between 1950 and 1962, the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated, for the most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Annual Symposium moved to different locations as follows:

Year	**HGS Location**	**Year**	**HGS Location**
1965	Lexington, KY	1966	Ames, IA
1967	Lafayette, IN	1968	Morgantown, WV
1969	Urbana, IL	1970	Lawrence, KS
1971	Norman, OK	1972	Old Point Comfort, VA
1973	Sheridan, WY	1974	Raleigh, NC
1975	Coeur d'Alene, ID	1976	Orlando, FL
1977	Rapid City, SD	1978	Annapolis, MD
1979	Portland, OR	1980	Austin, TX
1981	Gatlinburg, TN	1982	Vail, CO
1983	Stone Mountain, GA	1984	San Jose, CA
1985	Clarksville, IN	1986	Helena, MT
1987	Pittsburgh, PA	1988	Park City, UT
1989	Montgomery, AL	1990	Albuquerque, NM
1991	Albany, NY	1992	Fayetteville, AR
1993	Tampa, FL		

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

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

Meeting sites are chosen two or four years in advance and are selected by the Steering Committee following presentations made by representatives of potential host states. These presentations are usually made at the steering committee meeting which is held during the Annual Symposium. Upon selection, the state representative becomes the state chairman and a member protem of the Steering Committee.

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

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite a few examples, in Wyoming, the group viewed landslides in the Big Horn Mountains; Florida's trip included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt with principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland, the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Francisco Bay area which illustrated the planning, construction and maintenance of transportation systems. In 1985, the one day trip illustrated new highway construction procedures int he greater Louisville area. The 1986 field trip was through the Rockies of recent interstate construction in the Boulder Batholith. The trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

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

* Revised from the 41st Highway Geology Symposium Proceedings.

Highway Geology Symposium

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NOTE: Officers' terms expire at conclusion of 1993 Symposium.

Highway Geology Symposium

H.G.S.
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HISTORICAL SEQUENCE OF ORGANIZATION NAME:

Office of State Engineer and Geologist, 1852-55
State Geologist, 1886-87
Florida Geological Survey, 1907-71
Florida Bureau of Geology, 1971-83
Florida Geological Survey, 1983-present

NAMES AND TITLES OF ORGANIZATIONAL DIRECTORS AND DATES SERVED:

State Engineer and Geologist

Francis L. Dancy, 1852-55

State Geologists

John Kost, 1886-87
E. H. Sellards, 1907-19
Herman Gunter, 1919-58
Robert O. Vernon, 1958-71
Charles W. Hendry, Jr., 1971-88
Walter Schmidt, 1988-present

HISTORY OF THE FLORIDA GEOLOGICAL SURVEY

The origin of the Florida Geological Survey can be traced to the year 1852, when the office of State Engineer and Geologist was authorized by the legislature. "General" Francis L. Dancy, a former militia officer and mayor of St. Augustine, was chosen to head this office. Although Dancy did not have geological training, his extensive experience in engineering was useful to his office's responsibility to drain lowlands for agricultural development. In November 1855, Dancy requested $500 to do soil tests in various parts of the state, whereupon the legislature abolished his post.

The discovery in the 1880's of commercially valuable phosphate deposits in Florida prompted Governor E. A. Perry to appoint Dr. John Kost, a medical doctor and amateur geologist, as State Geologist in 1886. Dr. Kost completed studies of phosphate and other minerals in 1887. Dr. Kost's request to the legislature to extend his tenure and duties met the same fate as his predecessor's.

In 1907 enabling legislation was passed creating an autonomous, permanent Florida Geological Survey, and an office of State Geologist, with four support staff positions. The new Survey was given latitude to formulate its own choice of studies and research.

A reorganization of state government in 1933 placed the Florida Geological Survey under the newly-formed State Board of Conservation. The Survey remained essentially autonomous. State government was reorganized again in 1971. The Florida Geological Survey was placed in the Department of Natural Resources and its name changed to the Bureau of Geology. In 1983 the legislature reestablished the name of Florida Geological Survey, leaving unchanged its position in the department's hierarchy. Significantly during fiscal year 1986-87 the Florida Geological Survey celebrated its 78th year of service to the state. It is the oldest state agency functioning under both its original establishing legislative statute and its original title.

STATE GEOLOGISTS

E. H. Sellards: 1907-19

Since passage of the 1907 law, there have been five state geologists. The first was Dr. Elias Sellards, who for 2 years was Assistant Paleontologist with the Kansas Geological Survey. He received his B.A. and M.A. degrees from the University of Kansas and his Ph.D. from Yale University. He taught geology and mineralogy at Rutgers University and, in 1904, became an instructor at the University of Florida.

While at the University of Florida, Sellards devoted a considerable amount of time to the study of Florida's underground water resources, a subject of special concern to the state's agricultural interests. Water resources studies subsequently became a primary focus of the early work done by the survey staff. These early investigations, which included the underground water supply of central Florida and a survey of road materials, were directed toward serving Florida's economic needs. In later years, the emphasis became more academic and expanded to include paleontology and general Florida geology.

Under Sellards' guidance, the Geological Survey continued as a permanent part of state government. After Sellards' resignation in April of 1919, he joined the Bureau of Economic Geology of the State of Texas. His former student and staff assistant, Herman Gunter, assumed the position of State Geologist.

Herman Gunter: 1919-1958

Herman Gunter's association with the Florida Geological Survey spanned almost 52 years--a length of service unmatched by any other Florida State Geologist. Gunter graduated from the University of Florida with a B.S. degree in 1907 and in that same year joined the Survey staff. His advancement to director in 1919 ensured that the position was staffed by someone well versed in Florida geology.

As the Geological Survey's second director, he changed the survey's emphasis somewhat by making its reports more diverse and less academic in outlook and by more closely relating the Geological Survey's work to the needs of state government. In his role as administrator, Gunter encouraged cooperation with the state's public schools and enlarged the Geological Survey's museum and library. Gunter acted on the belief that a primary purpose of the Florida Geological Survey was to serve as a highly accessible source of information on Florida geology.

Under Gunter's direction, the Florida Geological Survey initiated a conservation campaign aimed at exposing the gross damage being done to the state's underground and surface water supplies by careless drilling practices and misuse of water. His interest in the preservation of the water resources of Florida also propelled him to the forefront as an opponent of the Cross Florida Barge Canal (originally conceived as a sea level ship canal across Florida).

Gunter also began work on the investigation of Florida's mineral resources. He sought and obtained funding for a cooperative venture with the U.S. Geological Survey to complete topographic mapping of the state.

It was also largely through his efforts that the legislature authorized and funded the construction of a geologic center comprised of the Florida State University's Department of Geology and the Florida Geological Survey. The proximity of these entities, which are housed next to each other on the campus of Florida State University, has provided for a cooperative use of scientific equipment and library facilities and has encouraged an open and stimulating exchange of ideas between the university and the survey over the years. This relationship has enhanced opportunities for student employment at the Florida Geological Survey and has benefited the survey staff by supplying skilled, knowledgeable graduate students to assist in areas involving practical geological research.

His contributions to geologic research were formally recognized by the University of Florida in 1944, when he was awarded an honorary Doctorate of Science. His accomplishments laid a firm foundation for the future. In recognition of his service, the Florida Geological Survey building on the campus of Florida State University was named the Gunter Building.

Robert O. Vernon: 1958-71

Herman Gunter's successor was Robert O. Vernon, who joined the survey as an Assistant State Geologist in 1941. Vernon received his B.S. from Birmingham Southern College, his M.S. from the University of Iowa, and his Ph.D. from Louisiana State University.

Emphasizing geologic research, Vernon conducted or participated in a large number of investigations concerning Florida geology. Part of his research emphasis resulted in the expansion of the cooperative program in water resource investigations between the Florida Geological Survey and the U.S. Geological Survey.

Recognizing the need for conservation of Florida's limited water resources, much of his time was spent informing the public about Florida geology and hydrology through numerous publications, public forums, and presentations to schools and civic organizations. The Florida statutes relating to conservation of water resources are principally the direct result of Vernon's efforts.

In November of 1971, Vernon resigned as State Geologist and accepted the position of Director of the Division of Interior Resources in the Department of Natural Resources. Robert Vernon is remembered as a dedicated professional who devoted many years of effort to Florida geology.

Charles W. Hendry, Jr.: 1971-88

Upon Vernon's resignation in 1971, Charles W. "Bud" Hendry, Jr., assumed the post of State Geologist. An employee of the Florida Geological Survey since 1949, Hendry held a number of positions including draftsman, stratigrapher, director of water resources investigations, and Assistant State Geologist. In addition to earning his B.S. from Florida State University, Hendry had the distinction of receiving the first M.S. degree in geology awarded by the Florida State University.

Under Hendry's direction the Oil and Gas Section significantly upgraded Florida's oil and gas regulations, providing better protection for the environment and conserving oil reserves. Seeing a state-wide need for geologic data to assist planners, Hendry had an environmental geology map series completed.

Walter Schmidt: 1988-Present

In 1985 Walter Schmidt was appointed Chief of the Florida Geological Survey and assumed the post of State Geologist in March 1988, upon Hendry's retirement. Schmidt did his undergraduate work at Florida Institute of Technology and the University of South Florida. His masters and doctoral work were carried out at Florida State University, with his doctoral dissertation specifically dealing with Florida Neogene stratigraphy.

One of his first acts as Chief of the Florida Geological Survey was to create a Mineral Resources and Environmental Geology Section. This program is designed to provide interpreted geologic data to the planning community as Florida's population continues to grow. As a supplement to maintaining the basic geologic repositories, one goal is to computerize all data and to upgrade the Survey's computer capabilities, providing better data management and graphic displays.

The State's oil and gas rules have been extensively rewritten, responding to Florida law changes, the constant technological changes in industry, and the environmental awareness of Florida's citizens.

FLORIDA GEOLOGICAL SURVEY PROGRAMS

Ranking first in the nation in the production of phosphate rock and titanium concentrates, second in fullers earth, crushed stone, and peat, as well as nineteenth in oil production in 1985, Florida's natural resources contribute about $2 billion annually to the state's economy.

The Florida Geological Survey has had, since its inception, two basic objectives. The first is to collect, interpret, report on, store and maintain geologic data. These data are used by governmental agencies, industry, and the public, and contributes to the responsible use and understanding of Florida's natural resources including groundwater. The second objective is to conserve Florida's oil and gas resources and minimize environmental impacts during exploration and production. The Survey has responded to these needs by establishing three sections: the Geological Investigations Section, the Mineral Resources and Environmental Geology Section, and the Oil and Gas Section.

The Survey maintains a geologic core and well cuttings depository of over 16,000 wells. It also operates a trailer-mounted auger rig and a full-time core drilling rig to acquire geological samples for projects. A depository of geophysical wire line logs contains over 4,800 logs from throughout the state.

The Survey's geologic data base is further strengthened by its paleontological invertebrate fossil depository, containing over 20,000 specimens of macro-fossils and over 10,000 micro-fossil specimens. The collections of Florida typical and/or guide fossils consist of mollusks, echinoids, ostracodes, foraminifers, bryozoans, corals, nannofossils, and diatoms.

An integral part of the Survey's research capabilities is its library, with its special collections of geological, industrial mineral, and petroleum industry publications. Its holdings include over 15,000 maps and aerial photographs, 11,000 government documents, 2,300 technical books, and lengthy runs of 35 scientific periodicals. It has access to GEOREF, a national computer-based information retrieval system, which significantly increases its value for research.

Geological Investigations Section

The Geological Investigations Section carries out primary applied geological research throughout the state. Current projects include revising the geologic map of Florida, county hydrogeologic reports, a summary of the state's economic minerals, and a study of the phosphate-bearing, Miocene age Hawthorn Group sediments.

This section also acts as a technical consultant for other governmental agencies regarding aquifer contamination and recharge, geologic hazards, minerals mapping, and community development and planning. Wells and cores on file are worked and analyzed with the information being added to the Survey's computer-coded data base, currently containing nearly 5,000 well logs. These data and associated computer programs are used by other governmental agencies and private firms.

Mineral Resources and Environmental Geology Section

This group maintains contact with the mineral industry in Florida and publishes biannual status reports. The state legislature recently required all counties to prepare comprehensive growth plans. This section is assisting the counties with the identification of their mineral resources by preparing mineral resource reports. These reports contain geologic data which are needed to make sound planning decisions. In addition, environmental geology reports are being prepared for several metropolitan areas. These studies will provide a document for land use planners which will focus on the physical environment, and mineral and groundwater resources.

Oil and Gas Section

This section regulates exploration and production of oil and gas in Florida through a system of permits and inspections. The section's main office is located in Tallahassee, with two field offices located near the producing fields at opposite ends of the state. One field office is in Jay, in the western panhandle, and the other is in Ft. Myers, in southwest peninsular Florida. Oil and Gas permits regulated by the section include: applications to drill oil, gas, or water injection wells; workover notifications; plugging and abandonment of wells; authorization to transport oil or gas from lease; and applications to conduct geophysical seismic operations.

USE OF AGGREGATE IN THE SUNSHINE SKYWAY BRIDGE PROJECT

BY

Robert E. Goddard, P.G. and William A. Wisner, Jr., P.G.
State Materials Office
Florida Department of Transportation
Gainesville, Florida

INTRODUCTION

The original Sunshine Skyway Bridge was built to connect the Bradenton-Sarasota area with the St. Petersburg-Tampa area without having to drive around the entire outline of the Tampa bay. This roadway reduces the travel mileage by approximately 44 miles between some of the most heavily visited areas of Florida's west coast. The original project consisted of one lane of traffic in each direction with no shoulder or breakdown lane and had a steel bridge structure and grated steel driving surface at the pinnacle. An identical project was added which paralleled the existing road. This added two additional lanes of traffic in which each span was designated with traffic going one way. On a stormy day, a ship coming into the channel hit the northern main pier of the younger western span and collapsed part of the roadway near the peak. Many cars and a bus hurled over the edge falling approximately 150 feet. Of the cars falling over there were no surviors. In the aftermath of this tragedy, the state of Florida decided to build a new roadway corridor rather than trying to rebuild and fortify the old structures. A tunnel was proposed, but was not cost effective. A new design was chosen which had worked well in Europe. In such a structure the aggregate would have to be durable and dependable. A study was done with recommendations given.

The primary concern was for the protection of the bridge from ships coming into the bay through the channel directly under the structure. Not only were the main piers reinforced and designed to withstand ship collisions, but obstacles called dolphins were placed strategically around all the primary and secondary piers so that ships out of control would be hindered or greatly slowed before getting to the main pier. Futhermore, around the main piers, islands of rock were built to further preclude any collisions. Ships would run aground on the rocks before reaching the main pier.

In all of these areas, specific aggregates were needed. Aggregates and the use of aggregates were as important as the design of the bridge itself. The sources come from as far away as Venezuela and as close as Brooksville, Florida, approximately 40 miles to the north. The Florida Department of Transportation approves sources of all aggregate intended for use of its projects through the suppliers (producers, brokers, and shippers) submitting a Quality Control Program and corresponding test data demonstrating that a stated product(s) meet(s) all applicable specifications for its intended use as stated by contract.

CONCRETE - MAIN STRUCTURE

The main structure of the Sunshine Skyway has basically two parts: the piers and the segments which comprise the roadway surface and the structural support. The main roadway consists of segments which start at each main pier. Each segment is added to the previous segment and tensioned. The large cables tie the segments to the top of the main piers and add stability. As with all concrete used by the Department, the concrete for the main structure was designed to meet specific criteria. The aggregate used was a big part of the concrete and its characteristics. In that Florida limestone aggregates in general tend to be more porous and lighter than concrete aggregates used in most of the United States, a panel of experts was conviened to determine the optimum aggregate source to be used for such an immense and important project. After aggregate suitability was determined, the project went out for bids. After the contractor was chosen, the mix designs for the main structures was submitted to the State Materials Office in Gainesville for testing and approval.

Aggregate Recommendations

A blue ribbon panel was assembled for the purpose of evaluating aggregate sources for suitability in the concrete for the main structure. The conclusions of the panel were that the Department of Transportation should not use Florida aggregates in a concrete pour of this kind because the aggregate has never been used in this massive a concrete pour before. This recommendation was based on the performance of other aggregates in massive concrete and was made to eliminate the uncertainty of Florida aggregate performance. The Department decided not to exclude aggregates from Brooksville, Florida, based on a good performance record within the Department and other governmental agencies. The contractors chose this as their aggregate source for cost, familiarity, and the past performance in concrete.

Main Piers - Base Structure

The base structure of the main piers has three components to it: drilled shafts, footings, and top seals.

Drilled Shafts

Each main pier is supported by 44 drilled shafts five to five and one-half feet in diameter located down to an elevation of -100 ft using class IV 3400 psi concrete.

Footings

The footings for the two main piers were tied to the shafts and has a diameter of 102 feet. Requirements for the concrete footing are 4200 psi class IV steel reinforced concrete.

Top Seals

The top seals of the base structure of the main piers were tied to the footing. Requirements for the top seals are 3000 psi class IV concrete.

Coarse Aggregate

Many design mixes were submitted for the main structure. Most of the design for coarse aggregate involved an ASTM Grade 57 in the precast segments. The source was FDOT Mine 08-005, Florida Mining and Materials, North of Brooksville, Florida. This material is a limestone. A summary of the gradation and selected physical tests is given at the mine for the time period of shipment to the project.

	Percent Passing						
	2"	1½"	1"	3/4"	3/8"	-200	LA abrasion
ASTM Grade 4	100	94.5	44.8	12.7	3.4	1.33	36
	1½"	1"	½"	No. 4	No. 8		
ASTM Grade 57	100	99.0	39.0	5.7	3.6	0.99	36.7
	1"	3/4"	3/8"	No. 4	No. 8		
ASTM Grade 67	100	94.4	33.6	6.9	3.9	1.02	36.2

The batch weights for the various design mix components are:

	Pounds Per Cubic Yard of Concrete			
Aggregate Grade	Drilled Shafts	Footings	Top Seals	Precast Segments
ASTM 4	620	1139	1046	
ASTM 57				1700
ASTM 67	944	772	713	

Although the Los Angeles abrasion of this rock is relatively high compared to most other rock in the United States used in concrete, the performance is comparable. Possible reasons are that while most aggregate acts as a filler in concrete, more porous aggregate better bonds to the cement paste and actually adds strength to concrete.

Fine Aggregate

The fine aggregate is a silica sand from FDOT Mine 16-081, Florida Mining and Materials (DeVane Silica Mining), located near Polk City, Florida, approximately 45 miles east-northeast of Tampa, Florida. This source produces a number of different grades of silica sand including concrete pipe sand and FC-4 sand which is used in asphalt friction course to increase skid resistance. A summary of the physical characteristics for specification requirements are given below.

Gradation	Sieve	Weight percent retained
	No. 4	0 to 5
	No. 8	0 to 15
	No. 16	3 to 35
	No. 30	30 to 75
	No. 50	65 to 95
	No. 100	93 to 100
	No. 200	minimum 96

Color .... not greater than 3

The mine gradation for the given time period of shipment to the project is:

Weight Percent Retained							
No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	Color
0.1	2.0	13.1	44.6	74.6	97.6	99.43	1.00

The batch weights for the various design mix components are:

	Pounds Per Cubic Yard of Concrete			
Aggregate Grade	Drilled Shafts	Footings	Top Seals	Precast Segments
Silica Sand	1212	1035	1064	899

Precast Segments

The bridge segments were precast at FDOT Concrete plant 13-247, Port Manatee, Florida, and required 5500 psi class IV concrete. The main span segments were post tensioned together with selected designed segments held by cables to the main pier towers. Each segment contains all four lanes of roadway, emergency lanes, middle section, and structural support (Figure 1). The top surface dimension of the main span sections are 42 feet wide.

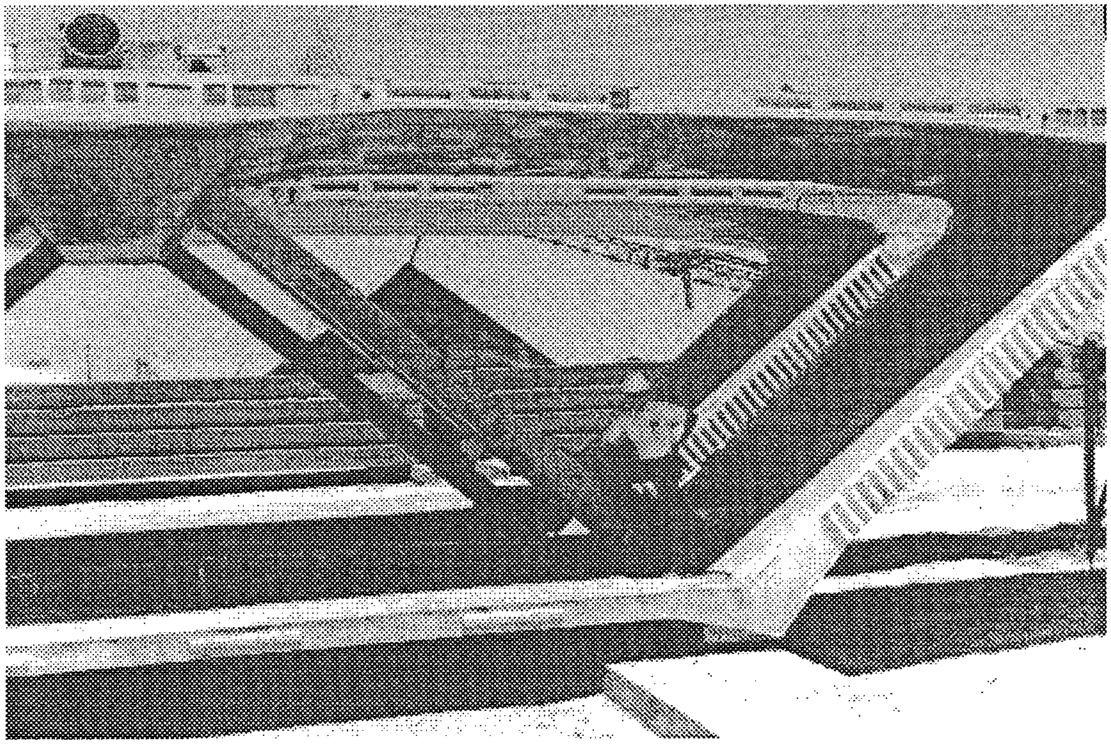

Figure 1. Precast Segment.

PIER PROTECTION

With the danger of hurricanes and rough seas, even though the main piers were designed to withstand collision by ocean going ships, the Department decided to expand the pier protection with barriers. The barriers took the form of islands around the piers and cylindrical bumpers called dolphins. In all 36 dolphins protect 12 piers of the main span.

Dolphins

These protection devices are sheet pile walled cylinders 54.33' to 47' feet in diameter and capped with reinforced concrete. They are filled with crushed aggregate. They are driven like a hollow tube into the bottom of the bay. The dolphin has scour protection. In a collision, the rock in the dolphins would absorb most of the impact and with the steel sides giving way if the collision were severe. The sides of the dolphin are protected by timber fendering consisting of treated structural timber lining the circumference. A cross section view of a dolphin is given in Figure 2.

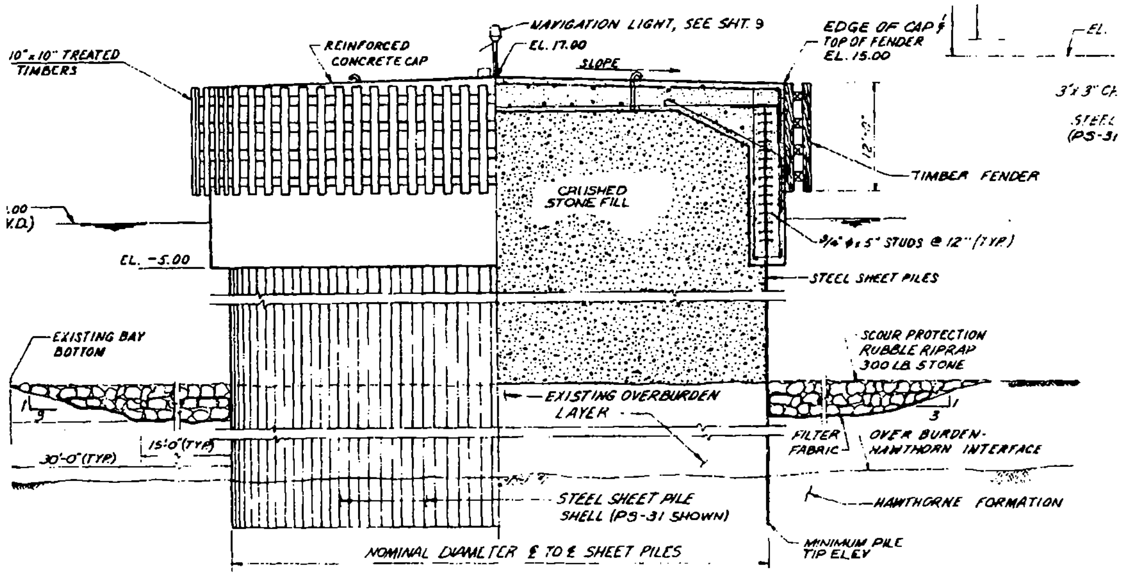

Figure 2. Dolphin Cross-Section.

Concrete Cap

The reinforced concrete cap for the Dolphins was constructed with a slightly conical top surface and a lip of 20 feet attached to and over the sheet piles. The concrete was designed for 5000 psi type IV concrete. The sources of the aggregate are:

Mineralogy	Source	Aggregate Grades (Batch Weights (PPCY))	Location	Producer
limestone	08-005	ASTM #57 (1605)	Brooksville, FL	Florida Mining and Materials
silica	16-081	Concrete Sand (1145)	Polk City, FL	Florida Mining and Materials (DeVane Silica Mining)

Gravel Fill

The interior of the dolphins was to be filled with crushed stone of ASTM Grades 357 or 4. The rock used was ASTM Grade 4. The source of the gravel fill is:

Grade	Mineralogy	Source
ASTM #4	granite	FDOT Source NS-315, Lone Star, Nova Scotia, Canada, through terminal TM-322, Dravo Basic Materials, Tampa, Florida

Scour Protection

Around the base to a thickness of five feet at the base into the bottom of the bay, rubble riprap was placed to prevent scour. The existing overburden material was excavated and filter fabric put down prior to placement of the rubble riprap. The rubble riprap wedges out and extends 30 to 35 feet around the dolphins. The specification for the rubble riprap was 300 pound stone uniform. The aggregate source is the same as for the main pier islands.

Islands

The final aspect of pier protection was to build an island of which the pier would be the center.

Design

Soil fill material would not be suitable as it would be swept away by the channel current. Likewise, typical construction of an aggregate island, coarser material at the base to prevent scour getting gradually finer upward, would also be impractical due to channel current and wave action degrading and carrying away the finer top aggregate. An island of all one size aggregate that would withstand the natural affects would be suitable but would be impractical as all of the aggregate would have to be placed individually. The final conclusion was to have the base of the islands the smallest size aggregate, under 400 pounds and then successively larger aggregate stacked on top. The succesive layers would hold the previous more susceptible aggregate in place. The smaller grades would be easier and faster to place by quantity leaving only the very top layers to be placed individually. Next to the main piers surrounded by filter fabric sand fill was used as an inverted wedge ring around the pier and expanded with the size of the island parallel to the channel. The sand core is surrounded by Quarry Run (Type V) stone and Type IV stone. The smaller grades would also act as a bedding stone for the larger aggregate and be better at yielding if a ship came that close. The subaqueous part of the island has a slope of 1:2 with the islands in the shape of a cone. The toe of the island has a concrete armor mat of a width of 58 feet and extending 25 feet past the toe of the outer slope. The result is a layered oblong cone-shaped island with smaller aggregate supporting the larger aggregate, and the larger aggregate protecting the smaller aggregate with the pier located at the center. This design reduces the damage to ships that collide with it also. A cross section view parallel to the roadway is given in Figure 3.

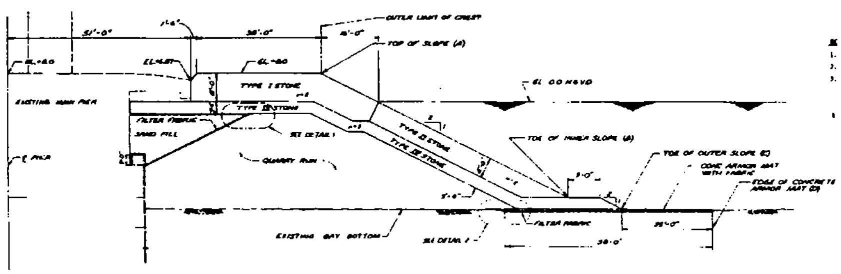

Figure 3. Main Pier Island Cross-Section

Aggregate Grades

The following is a list of grades used and the minimum density required of each grade.

Type Stone	Description	Minimum weight/ cubic yard	Nominal Range in stone type weight in pounds
I	Rip Rap Armor	164	8000 - 14000
II	Rip Rap Armor	164	3000 - 8000
III	Rip Rap Armor	144	500 - 2000
IV	Rip Rap Armor	144	50 - 1000
V	Quarry Run	144	Minus 400 lbs,
--	Rip-Rap Rubble	144	200 - 600

Aggregate Sources

Density requirements along with sizing were the primary concerns. Florida does have material to meet these requirements in the form of chert boulders located at limestone mines. The Florida chert boulders, however, are widespread and not in sufficient quantity to meet the supply. The majority of the aggregate was to come from out of state. Producing and sizing aggregate for a specific large grade in time consuming and expensive and produces a relatively low yield. In that the project was at the entrance to a port. Transportation of the majority of the larger aggregate was restricted fiscally to ship or barge transport.

In the case of Florida, the contractor has the option of specifying the aggregate source. The stipulation is that the source must be approved by the Department prior to shipment of any grade of aggregate. The sources of stone are:

Grade	Rock Mineralogy	Source
Riprap	limestone	Sun Minerals, FDOT Source JA-341, Duncan's, Jamaica, through terminal TM-342, Sun Minerals, Tampa, Florida
Riprap	limestone	VENMARCO, FDOT Source VE-346, Pertigalete, Venezuela, through terminal TM-342, Sun Minerals, Tampa, Florida
Riprap	limestone	VENMARCO, FDOT Source VE-346, Puerto Cabello, Venezuela, through terminal TM-342, Sun Minerals, Tampa, Florida
Riprap	chert	Sun Minerals, FDOT Source 08-344, Brooksville, Florida

RESILIENT MODULUS vs STRENGTH in CEMENT STABILIZED BASE COURSES

Khalid Jamil and Sam I. Thornton

ABSTRACT

Unconfined compressive strength is usually used to design cement treated base courses. A minimum of 400 psi in 7 days is used by the Arkansas Highway and Transportation Department. The resilient modulus, however, is now a part of the design process. As a result, it would be beneficial to find a relationship between the unconfined compressive strength and resilient modulus.

For the study, samples of crushed limestone and cement were cured 28 days, capped and tested for resilient modulus and unconfined compression. Deviator stresses of 40, 80, and 160 psi were used in the resilient modulus testing.

The resilient modulus increased slightly with increased unconfined compressive strength. Increases in deviator stresses resulted in increases in the resilient modulus.

INTRODUCTION

Cement treated base courses are often designed by specifying a minimum unconfined compressive strength. An example is the Arkansas Highway and Transportation Departments 1988 Specifications, "Specimens of soil aggregate, cement and water must develop a compressive strength of at least 400 psi in 7 days".

The new AASHTO Guide for the Design of Pavement Structures requires the use of the resilient modulus, M_R, (AASHTO T294I) to design flexible pavements (Elliott and Thornton, 1988). As a result, it would be beneficial to find a relationship between the unconfined compressive strength and resilient modulus of cement treated base materials.

BACKGROUND

Only a few reports of attempts to correlate M_R with unconfined compressive strength in cemented material have been made. K. P. George (1990) concluded that M_R increases with increasing unconfined compressive strength but that the relation depends on the AASHTO classification. For the same unconfined compressive strength, A-1 soils had the highest M_R and A-4 & A-6 soils were lowest. A-2 & A-3 soils were in between.

Lofti and Witczak (1985) found a semi-logarithmic relation between M_R and unconfined compressive strength:

$$\log M_R = -0.141 + 0.00035\, q_u$$

The relation, based on samples of dense graded aggregate and cement, had a correlation coefficient, r, of 0.84.

THE STUDY

For the study, samples 6 inches diameter by 12 inches high were compacted in five layers according to the method of aggregate preparation for dynamic testing (Qedan, et al, 1988). The aggregate was a well graded crushed limestone and cement contents varied from 4 to 10%. Water contents were 5 and 7%. Samples were cured 28 days, capped and then tested for M_R and unconfined compression strength. Because of the high strengths, M_R testing was unconfined. Deviator stresses were 40, 80, and 160 psi.

Results

The resilient modulus increases slightly with increased unconfined compressive strength. Increases were found at 40, 80, and 160 psi deviator stresses (Figures 1, 2 & 3). The correlation, however, was not high. R was 0.56 for the 40 psi deviator stress, 0.64 for the 80 psi deviator stress, and 0.61 for the 160 psi deviator stress.

Increases in deviator stress resulted in increased M_R values (Figure 4). Both the intercept and slope of the line increased in Figure 4 as deviator stress was increased.

No significant difference in the M_R vs unconfined compressive strength relationship was found for changes in cement content or water content.

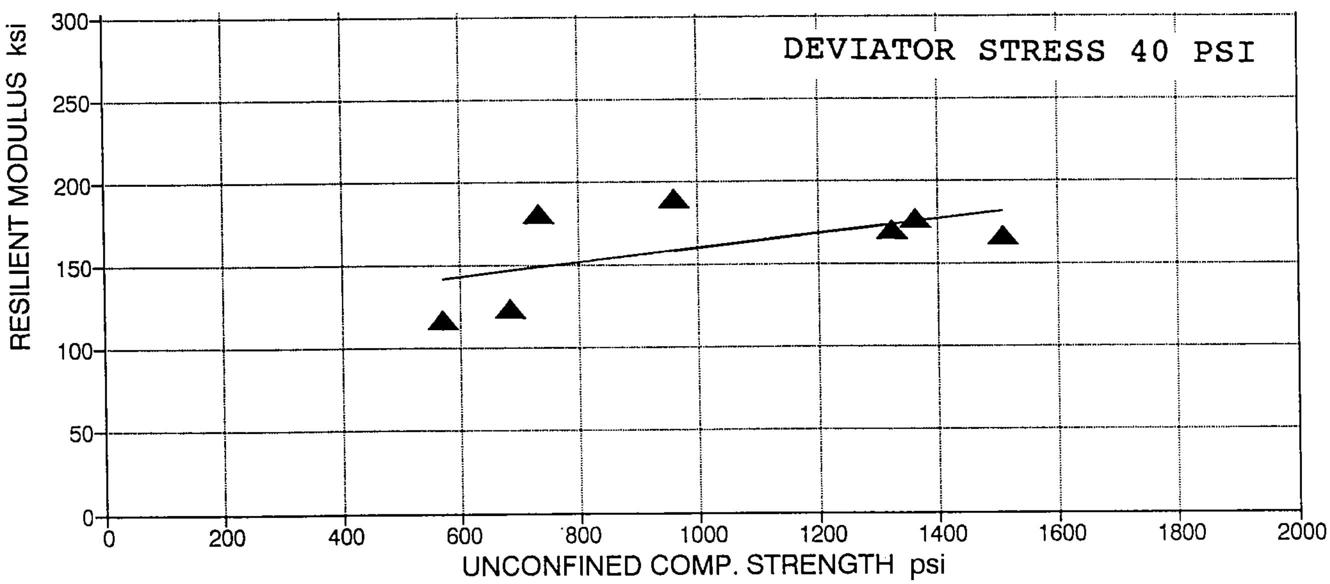

FIGURE 1

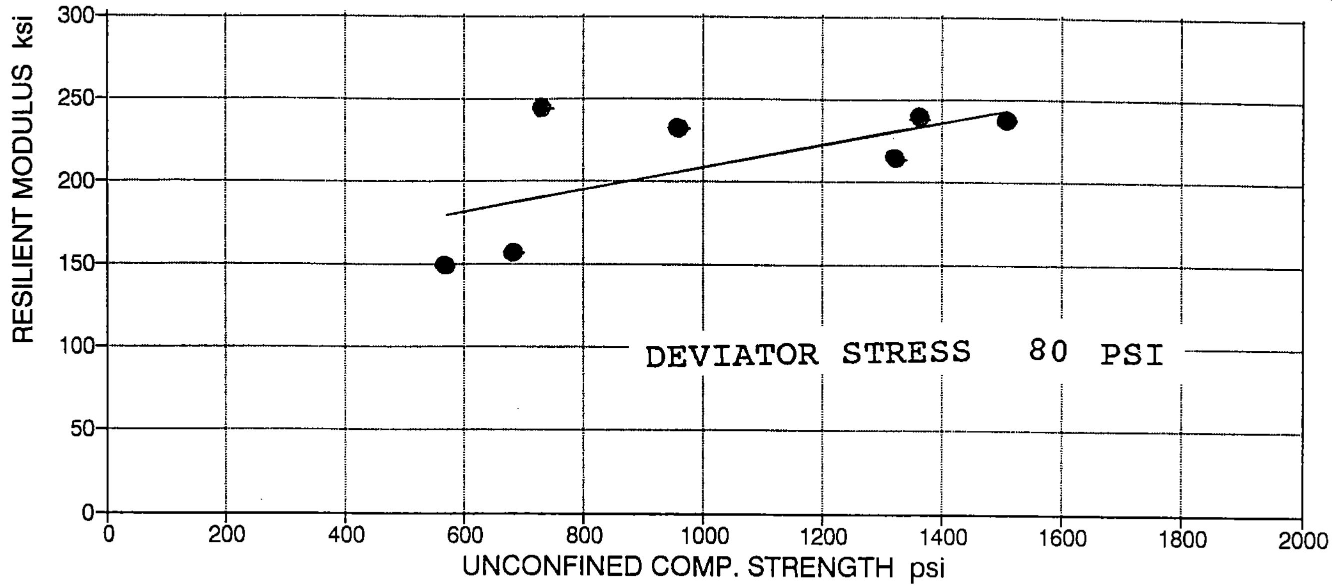

FIGURE 2

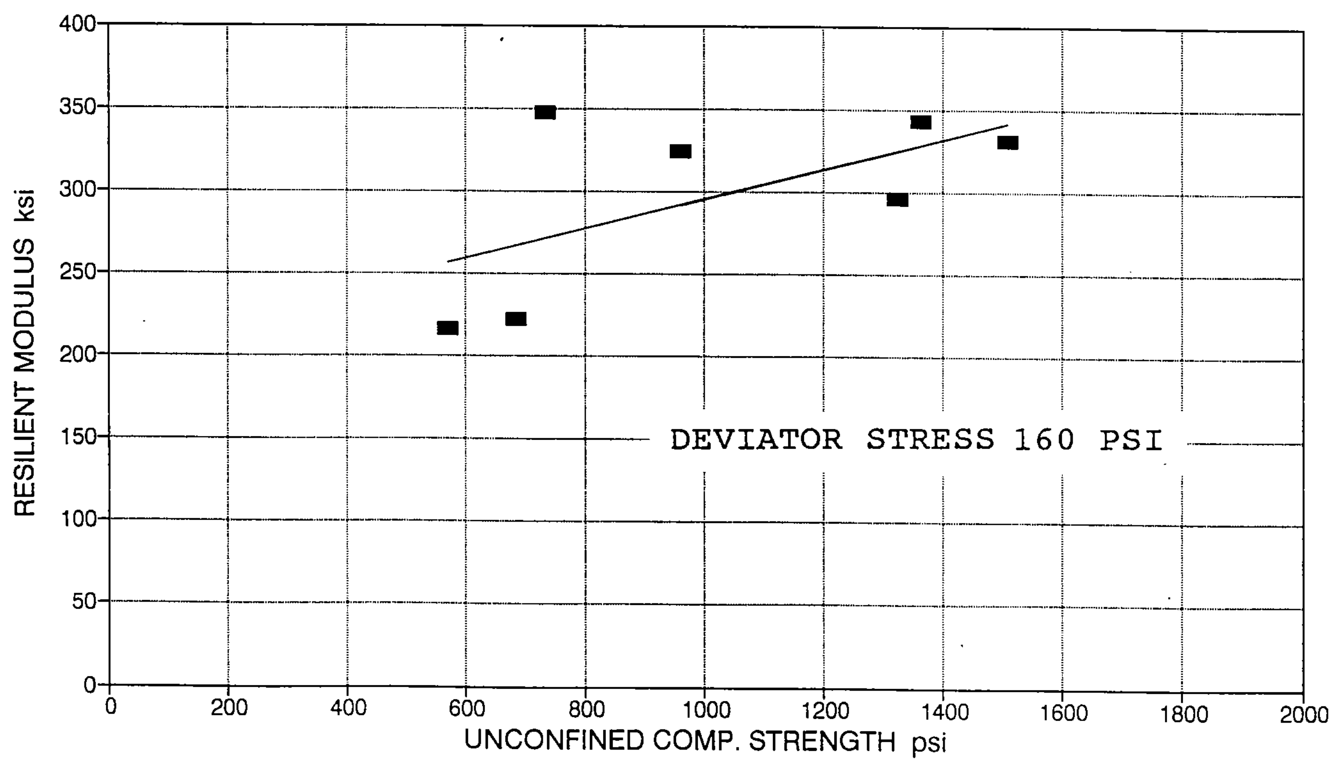

FIGURE 3

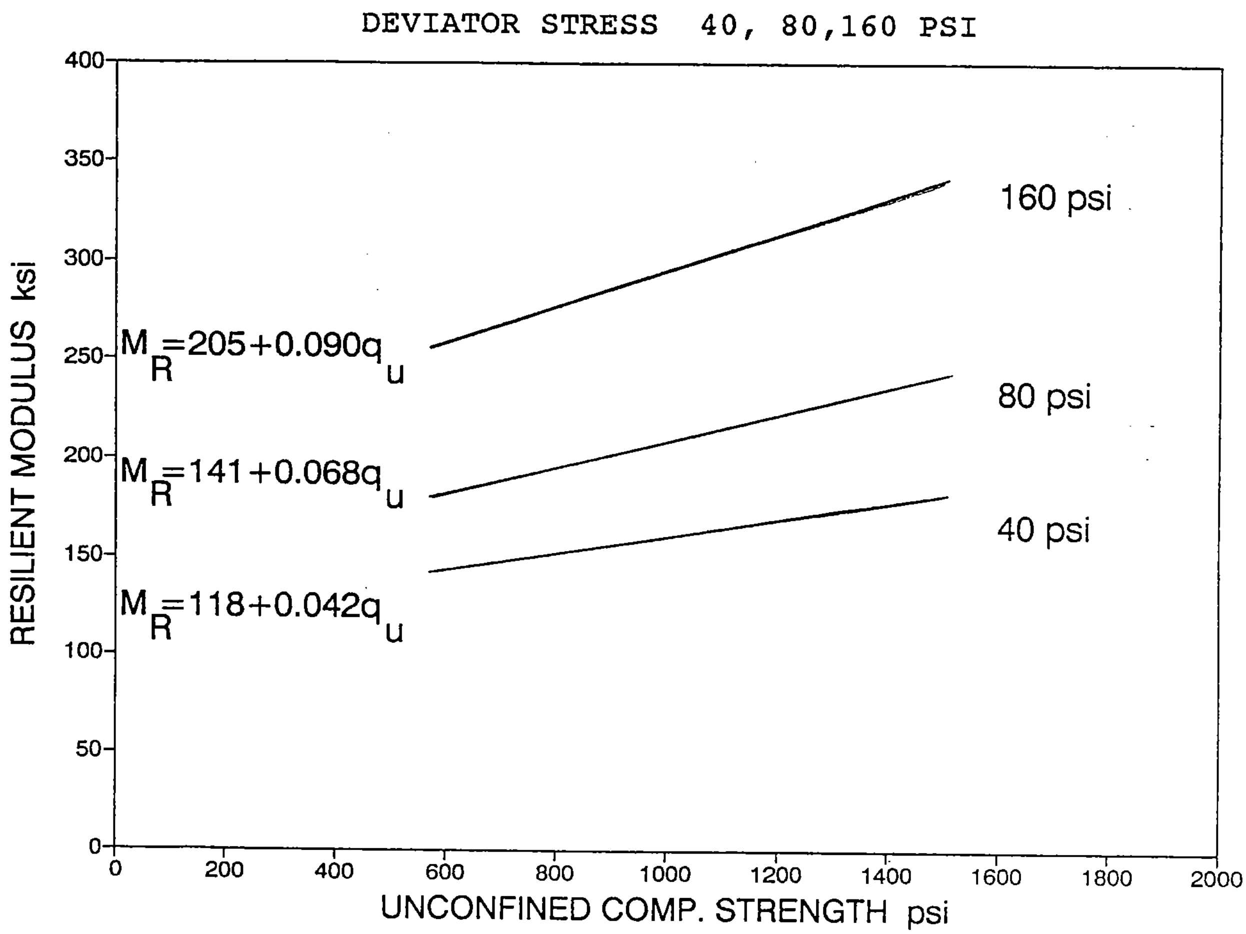

FIGURE 4

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Use of Waste Foundry Sand in Highway Construction

S. Javed[1] and C.W. Lovell[2]

Abstract

The development of innovative and constructive uses of foundry wastes will provide an opportunity for substantial savings for the foundry industry. This paper presents a review of available information on waste foundry sands, their generation processes, potential variables, and environmental concerns associated with beneficial uses.

The applications of waste foundry sand which are discussed are: fine aggregate supplement for portland cement concrete products and asphalt concrete pavements, silica additive for cement, snow and ice abrasive agent, and as a fill material.

Leaching characteristics of ferrous foundry wastes under laboratory leaching procedures and under actual field conditions are also discussed. Chemical analysis of the leachates show that waste foundry sand is non-hazardous and its effect on the quality of ground water is low.

The physical and chemical tests needed to assess the technical feasibility and enviromental acceptability are outlined. In addition, the paper includes test results on moisture-density relationships, California bearing ratio, compressibility, permeability and shear strength of these materials. The various test values and properties are compared to those of representative granular soils or contained in appropriate specifications. These comparisons will provide necessary information for judging the suitability of waste foundry sands in highway construction.

Introduction

Waste foundry sand (WFS) is a mineral resource that can be put to beneficial use. Beneficial reuse represents an alternative to waste disposal and the associated problems of diminishing landfill capacity and increasing tipping fees. Nonhazardous foundry wastes can be used for a variety of construction purposes. However it is generally required that it be demonstrated through physical and chemical testing that the particular waste is suitable for these uses. This is critical because of concern for the protection of groundwater quality.

This paper is based on ongoing laboratory study on the physical/mechanical properties of Indiana waste foundry sands. The discussion is limited to foundry wastes which are produced in molding and core making, and their use in highway construction, and does not include melting waste, slag or waste generated by other foundry processes.

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Waste foundry sand and their generation process

WFS is generated by industries that use sand to form molds and cores for castings. The mold forms the outside of the casting; the core forms the internal shape. When the part to be made has deep recesses or hollow portions, sand cores must be provided in the mold.

Although there may be differences in particular operations, the basic foundry process varies only slightly from one foundry to another. All foundry operations produce castings by pouring molten metal into a mold, often consisting of molding sand and core sand. Once the casting has hardened, it is separated from the molding and core materials in the shakeout process. Castings are cleaned, inspected, and then shipped for delivery. Figure 1 is a schematic of a typical foundry process, showing both the finished product and the types of air emissions and wastes generated.

Potential variables in the generation process

One variable considered in this study was the type of molding process. The different types used in Indiana are:
- green sand
- chemically bonded
- shell mold
- semi permanent
- investment cast

Among these, semi-permanent and investment cast do not utilize sand for molding and therefore they are not discussed. In Indiana, the most commonly used process is green sand molding. "Green sand" indicates that the metal is poured into the molds when the sand is damp, as it is when the mold is made (Wendt, 1942). In the green sand process, bentonite is typically added as a binding agent, together with other additive like seacoal. In the chemically bonded process, different kinds of chemicals are used to bind the sand particles and the mold is air dried. Typically, furfuryl alcohol is used at a higher proportion, while resins are used at a lower proportion. "Shell mold" uses a mixture of sand and thermosetting resin (usually phenol-formaldehyde) to form the mold. When it touches a heated substance, the sand-resin mixture forms a thin shell due to polymerization of the resin, which binds the sand particles.

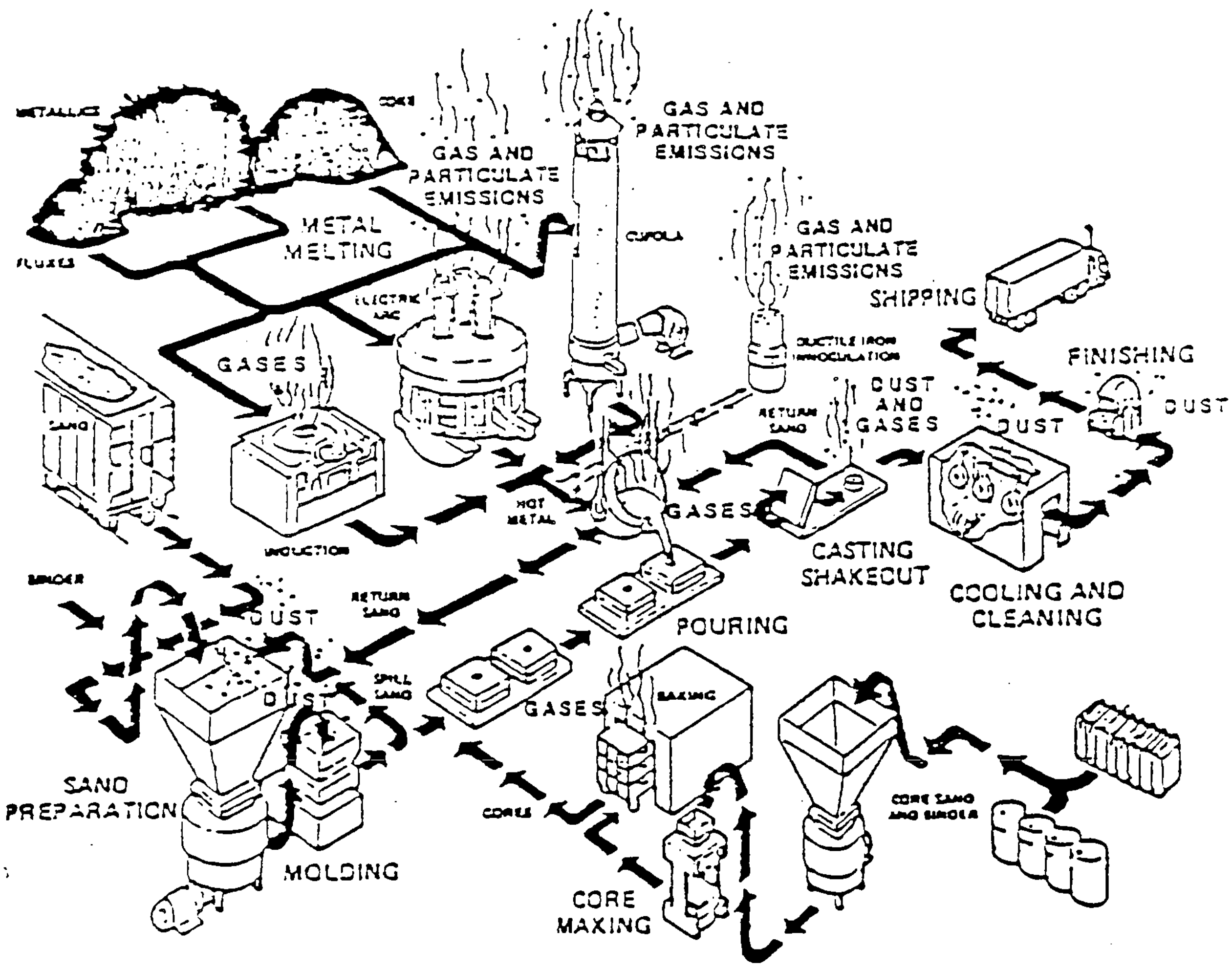
The ten samples which were collected for this study involved seven from the green sand process, two from chemically bonded processes and one from the shell molding process. Samples from green sand are designated as G, chemically bonded as C and shell mold as S.

Environmental concerns associated with beneficial uses

Before concluding that waste foundry sand is suitable for

FIGURE 1

GENERATION PROCESS OF WASTE FOUNDRY SAND (FROM KUNES T.P., 1987)



highway construction one should consider the regulations which govern its disposal.

The Office of Solid and Hazardous Waste Management of the Indiana Department of Environmental Management (IDEM) adopted final solid waste management regulations in August 1988 which became effective in February 1989. These regulations classify foundry wastes on the basis of result from the EP Toxicity Test and a modified EP Toxicity water leaching procedure (Indiana Leach Test) as shown in Table 1 (329 IAC 2-9, 1992). Wastes which leach parameters in concentration at or below the concentration shown are classified as wastes suitable for disposal in Types I, II, III, or IV sites. Wastes that are documented to pass the Type IV criteria have minimal requirements for disposal and are not subject to the provisions of the regulations (329 IAC 2, 1992). The regulations also provide for beneficial use of foundary sand meeting the Type III category,if the use is "legitimate", including the use as a base for road building (329 IAC 2-3-1, 1992). Other uses may be approved if they determined to be legitimate uses that do not pose a threat to public health or the enviroment.

For sites Type I, II, and III, additonal siting restrictions apply. These facilities must also have a soil barrier between the solid waste and any aquifer. The thickness of the soil barrier depends on the waste type and permeability,the physical and chemical properties of the soil, the nature of groundwater resources in the area and the use of alternative liner technology (329 IAC 2-10-4, 1992). Facilities accepting waste Type I, II or III require formal detailed permit applications,with Type III being somewhat less restrictive. For example, Type III sites only require additional cover and do not require ground water monitoring.

Laboratory and experimental observations

Extensive work on the chemical/environmental properties of ferrous foundry wastes has been carried out in Wisconsin. Laboratory study was conducted at the Univeristy of Wisconsin, Madison. Waste foundry sand was collected from three ferrous foundries and also natural soils were tested for comparison purposes. The results of that study indicated that none of the foundry sands tested would be classified as hazardous by the Resource Conservation and Recovery Act (RCRA) definition of 100 times the primary drinking water standard in the Toxicity Characteristic Leaching Procedure (TCLP) test. The parameter of highest concern for sands from all three foundries in that study was iron. The concentration of iron did not meet the drinking water standard in the TCLP leachates of the foundry sands from the three foundries, and it was significantly higher than in leachates from the Wisconsin soils tested.

Leaching characteristics of the foundry wastes under actual field conditions was measured through the use of lysimeters to collect portion of the leachate under test piles, and groundwater monitoring wells.

Two field leaching sites were selected. Test fill cells of

Table 1 (from 329 IAC 2-9-3, 1992)

IDEM FOUNDRY WASTE CLASSIFICATION GUIDELINES CONCENTRATIONS (MG/L)

(1) For Parameters Using the EP Toxicity Test:

Parameter	Concentrations (Milligrams per liter)			
	Type IV	Type III	Type II	Type I
Arsenic	≤ 0.05	≤ 0.50	≤ 1.25	< 5.00
Barium	≤ 1.00	≤ 10.00	≤ 25.00	< 100.00
Cadmium	≤ 0.01	≤ 0.10	≤ 0.25	1.00
Chromium	≤ 0.05	≤ 0.50	≤ 1.25	< 5.0
Lead	≤ 0.05	≤ 0.50	≤ 1.25	< 5.0
Mercury	≤ 0.002	≤ 0.02	≤ 0.05	< 0.20
Selenium	≤ 0.01	≤ 0.10	≤ 0.25	< 1.0
Silver	≤ 0.05	≤ 0.50	≤ 1.25	< 5.0

(2) For Parameters Using the Leaching Method Test:

Barium	≤ 1.00	≤ 10.00	≤ 25.00	*
Boron	≤ 2.00	≤ 20.00	≤ 50.00	*
Chlorides	≤ 250.00	≤ 2.50	≤ 6.25	*
Copper	≤ 0.25	≤ 2.50	≤ 6.25	*
Cyanide, Total	≤ 0.20	≤ 2.00	≤ 5.00	*
Fluoride	≤ 1.40	≤ 14.00	≤ 35.00	*
Iron	≤ 1.50	≤ 15.00	*	*
Manganese	≤ 0.05	≤ 0.50	*	*
Nickel	≤ 0.20	≤ 2.00	≤ 5.00	*
Phenols	≤ 0.30	≤ 3.00	≤ 7.50	*
Sodium	≤ 250.00	≤ 2.50	≤ 6.25	*
Sulfate	≤ 250.00	≤ 2.50	≤ 6.25	*
Sulfide, Total	≤ 1.00**	≤ 5.00	≤ 12.50	*
Total Dissolved Solids	≤ 500.00	≤ 5.00	≤ 12.50	*
Zinc	≤ 2.50	≤ 25	≤ 62.50	*

Parameter	Acceptable Range (Standard Units)			
pH	6-9	5-10	4-11	*

* Testing is not required

** If detection limit problems exist, please consult the office of solid and hazardous waste for guidance.

5,10 and 15 feet heights were constructed. Each site had two fills of these heights, one of foundry sand and one of native soil. Managanese, arsenic and total dissolved solids (TDS) were found at significantly higher concentrations than the drinking water standard levels, both in the foundry sand test piles and in the natural soil test piles at both sites. Since they were found in both types of materials, the source of these parameters may not be from waste foundry sand alone. A source common to both piles such as rainfall or lysimeter construction materials and practices could be responsible for the high concentration of arsenic, manganese and TDS found in the lysimeter leachate. There was no evidence that the waste foundry sand tested would pose a threat to groundwater when compared to natural soils.

Potential use of waste foundry sands in highway construction.

In the following sections, the applications of waste foundry sand as a fine aggreate supplement for portland cement concrete products and asphalt concrete pavements, as a silica additive for cement, as a snow and ice abrasive agent and as a fill material are discussed.

Portland cement concrete and asphalt concrete products

Concrete is composed of binding agent, water and aggregates of various sizes. In Portland cement concrete, Portland cement is the binding agent. Although extensive testing is desired to evaluate concrete mixture performance, some initial testing has been carried out for partial replacement of fine aggregate in the concrete mixture. The tests included: particle size distribution, dust content, organics content, clay lumps and friable particles, soundness, and absorption.

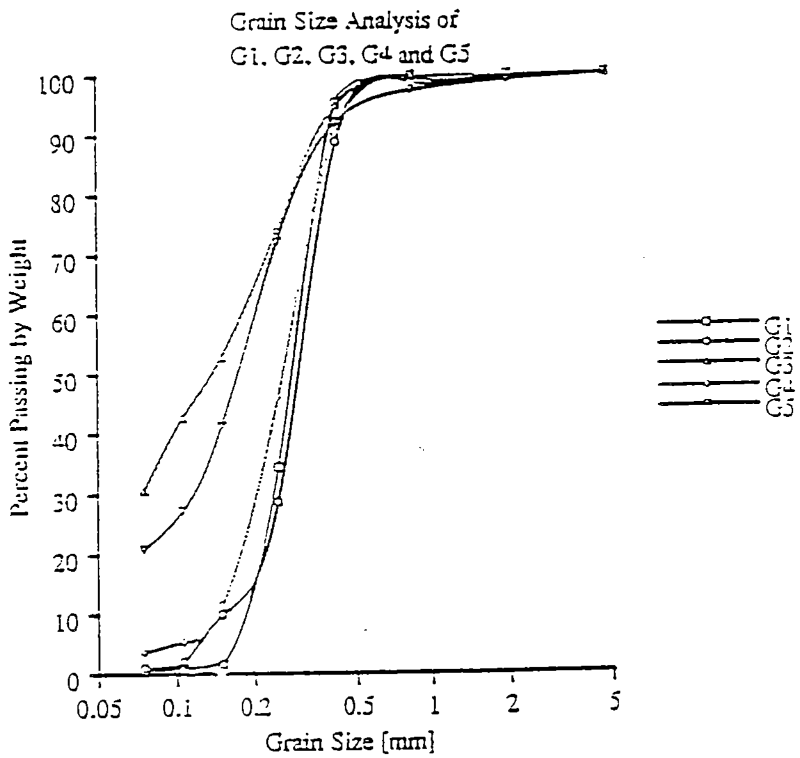
Particle size distribution

The particle size distributions of the ten test samples are shown in Figure 2. Table 2 shows typical distribution of a waste foundry sand sample and the gradation requirements of fine aggregate in Portland cement concrete and a particular surface mixture of asphalt concrete (asphalt sand) as per Indiana highway specifications of 1988.

Comparing the gradations, foundry sands are found to be too fine to satisfy the specifications for fine aggregate in Portland cement concrete.

FIGURE 2

GRAIN SIZE ANALYSIS OF THE TEN SAMPLES TESTED



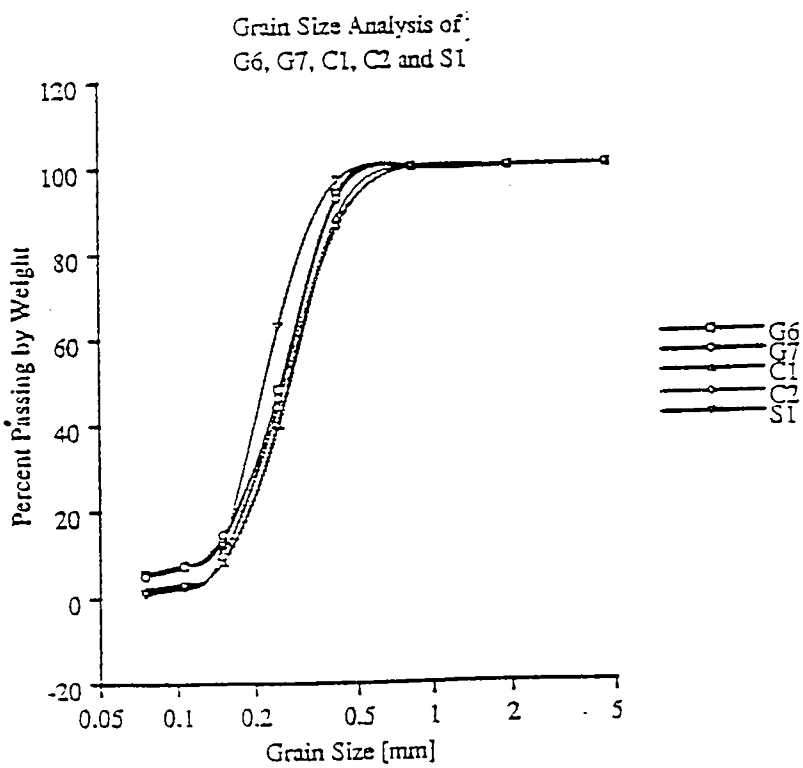


Table 2

Comparison of foundry sand gradation with Portland cement concrete and asphalt sand (From IDOH, 1988)

Sieve Sizes	Sizes (percents Passing)				
	Portland Cement Concrete			Asphalt Sand	Foundry Sand
	23	24	15		
3/8"	100	100		100	100
#4	95-100	95-100		95-100	100
#6	-	-	100	-	100
#8	80-100	70-100	90-100	70-90	100
#16	50-85	40-80	-	40-68	99
#30	25-60	20-60	50-75	20-50	97
#50	5-30	7-40	15-40	7-30	55
#80	-	-	-	-	20
#100	0-10	1-20	0-10	1-20	14
#200	0-3	0-6	0-3	0-4	4

Dust content

In Portland cement concrete, clays and other fines can increase the water demand which increases the amount of cement to maintain the strength. ASTM C33 allows for 5% by weight of -200 mesh materials (smaller than 0.075mm). In asphalt concrete an increase of fines result in decrease of percentage voids. Two out of ten samples (G3 and G5) failed this requirement, as can be seen from particle size distibution curves (Figure 2).

Organics content

Loss on Ignition (LOI) is a measure of the unburned carbon content. From an engineering point of view unburned carbon is considered as a contaminant. In Portland cement concrete, high carbon content increases water demand due to high porosity of carbon particles and it also reduces fineness and pozzolanic activity. Lightweight, fine aggreggate specifications require a LOI of less than 5% and concrete masonary units allow up to 12% LOI for coal and related substances (AFS report, 1991). The results are summarized in Table 3. Three samples had LOI greater than 5% while one had LOI greater than 12%.

Clay lumps and friable particles

Clay lumps are lumps of clay and silt which remain cohesive during processing. Friable particles are characterized by a poor bond between the grains, hence they break down easily into many smaller pieces. Large quantities of friable particles in the aggregate cause a reduction in the concrete strength and small quantities downgrade the abrasion resistance of concrete significantly (Dolar-Mantuani, 1978). The results are summarized in Table 3.

Soundness

The soundness of aggregate is considered as a measure of the material's resistance to disintegration caused by weathering actions such as alternate freezing and thawing, wetting and drying, heating and cooling and action of aggressive waters (Yoder et al., 1975). According to Indiana highway specifications for fine aggregate, soundness loss should not be more than 10%. Six of our ten samples failed this requirement.

Absorption

Absorption values are used to calculate the change in the weight of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it is deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential. The recommended absorption values according to Illinois Department of Transportation is a maximum of 5% (AFS report, 1991). This test was carried in accordance with ASTM C128 and the values on the ten samples are reported in Table 3.

Table 3

Results of soundness, clay lumps and friable particles, loss on ignition and absorption of the ten samples tested.

Sample #	Soundness %	Clay Lumps & Friable Particles, %	LOI %	Absorption %
G1	9	1.35	3.16	3.18
G2	6	1.72	2.11	2.29
G3	25	44.33	12.06	5.07
G4	45	2.59	3.75	3.62
G5	9	0.62	5.32	4.87
G6	17	2.26	5.43	3.40
G7	47	23.22	6.31	2.27
C1	12	100	1.61	3.00
C2	21	0	1.01	3.24
S1	10	10.64	0.35	3.20

From these initial testing, it is inferred that although not all of the waste foundry sands satisfied all of the specifications which were studied, some waste foundry sands may be used for Portland cement concrete and asphalt concrete. Moreover, they may only be used as a partial replacement in fine aggregates for concretes rather than as a complete substitution for fine aggregates, as can be inferred from gradation requirements.

Silica additive for Portland cement

It is known that waste foundry sand contains a major portion of silica (AFS Report, 1991), an essential raw material for manufacture of Portland cement. Conventionally, shale is used to provide for this basic ingredient. However, sometimes deficiency in this basic requirement requires supplemental additions of silica. Waste foundry sand, rich in silica, may serve to eliminate this deficiency. An additional benefit of using fine grained waste foundry sand will be the reduction in grinding energy, since conventional materials require crushing before being sent to kilns for manufacture of Portland cement.

Snow and ice abrasives

According to the Indiana Department of Highways, 1988, Section 903.01 (f), such abrasives are required to conform to the gradation shown in Table 4. For comparison, typical gradation of foundry sand is also included. Some blending of coarse sand is obviously needed to meet the requirements for snow and ice abrasives.

Table 4

Comparison of the waste foundry gradation with the gradation requirement for snow and ice abrasives (From IDOH, 1988)

Sieve Size	Snow & Ice Abrasives % Passing	Foundry Sand % Passing
3/8"	100	100
#4		100
#8		100
#16		99
#30		97
#50	0-30	55
#100		14
#200	0-7	4

Fill material

To evaluate the use of fill material, selected samples were tested for geotechnical properties including unit weight, California Bearing Ratio (CBR), shear strength, compressibility and permeability.

Maximum and minimum index density

These are helpful in evaluating the relative density of soil which in turn is used to determine the state of compactness of a given soil mass. These were carried out in accordance with ASTM D 4253 and D 4254. The determination of maximum index density utilizes vibration by using a vibrating table. The results are reported in Table 5.

Moisture density relationships

This was carried out in accordance with ASTM D 698 and the results are shown in Figure 3. Uniformly graded soils, consisting of a narrow range of particle sizes, give a flatter compaction curve as opposed to clay soils, well-graded sandy or silty soils which show a clearly defined peak to the compaction curve (Head, 1980). This was observed particularly for C2 which was found to be cohesionless. Figures 3, 4 and 5 show moisture density curves for the three samples. Table 5 compares unit weights and optimum moisture contents (OMC) of the foundry sand samples with conventional materials. For cohesionless soils, maximum density is observed using vibration and this was verified for C2.

Table 5

Results of the unit weights and OMC for foundry sand samples and their comparison with the conventional materials

Sample #	Min. Index Density (pcf)	Max. Index Density (pcf)	Density ASTM D 698 (pcf)	Opt. Moisture Content, %
G1 (SP)[1]	87	103	110	11.9
G3 (SM)[1]	69	83	99	20.2
C2 (SP)[1]	94	111	103.3	12.5
SP[2]	100-120			12-21
SM[2]	110-125			11-16

[1] Classification according to Unified Soil Classification
[2] Typical properties from NAVFAC Manual DM 7 (1982)

California Bearing Ratio (CBR)

The CBR test is a small scale penetration test and is a ratio

FIGURE 3

MOISTURE DENSITY RELATIONSHIP AND CBR FOR G1

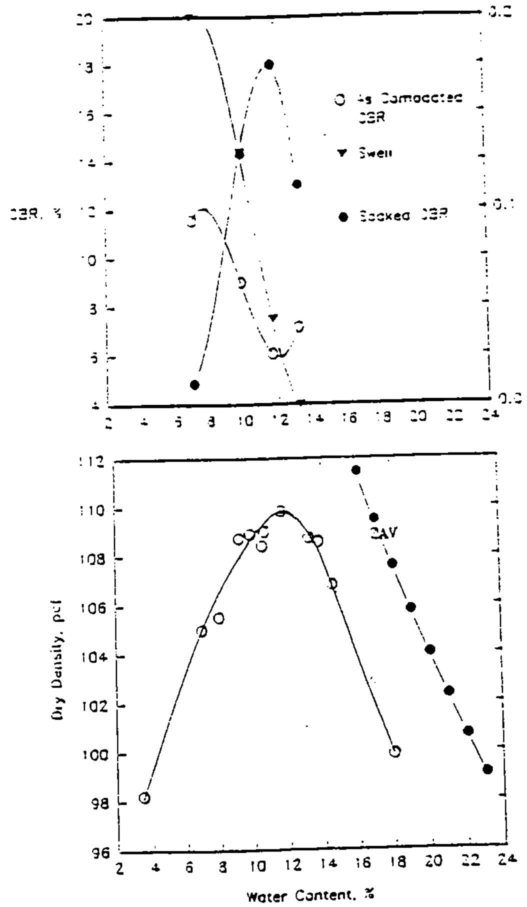


FIGURE 4

MOISTURE DENSITY RELATIONSHIP AND CBR FOR G2

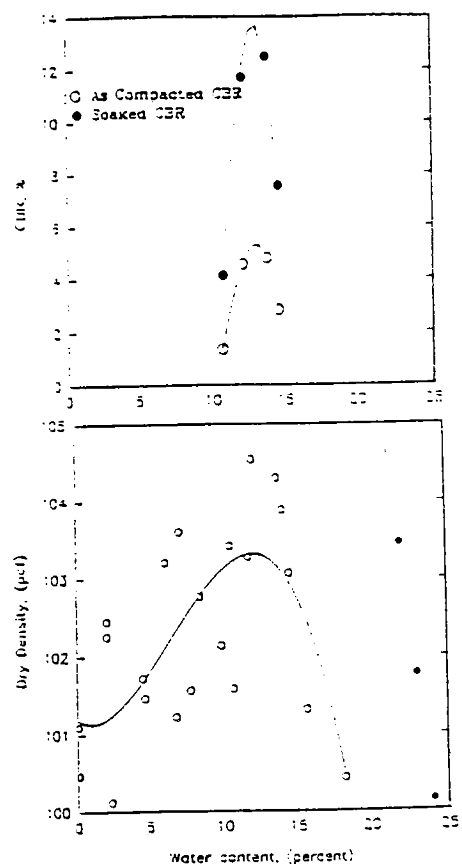
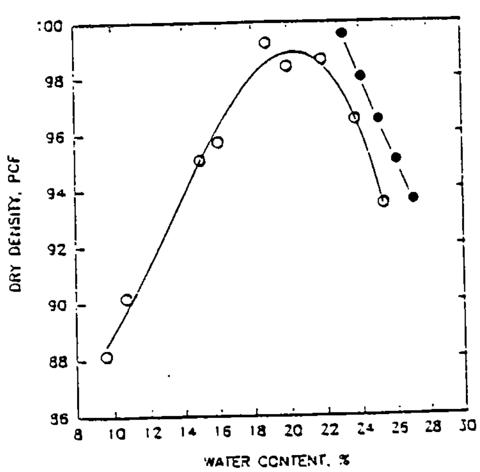


FIGURE 5

MOISTURE DENSITY RELATIONSHIP FOR G3



between the test load to the standard load expressed as a percentage. Based upon CBR values, thickness of pavements are designed. The results are plotted in Fig. 3 and Fig. 4. Typical CBR values for SP material is between 15 and 25 (Yoder and Witczak, 1975).

Shear Strength

The shear strength parameters were determined by direct shear tests. The normal stress used in the test ranged from 5 psi to 19 psi. The results are summarized in Table 6. The observed values compared very well with the reported values for conventional materials (NAVFAC DM 7, 1982).

Table 6

Results of shear strength parameters and their comparison with the conventional materials

Sample #	Relative Density, %	c intercept psi	Angle of shearing resistance, degrees
G1 (SP)[1]	90	1.44	36.6
G3 (SM)[1]	98	1.04	40.9
C2 (SP)[1]	94	1.82	34.9
SP[2]			37
SM[2]			34

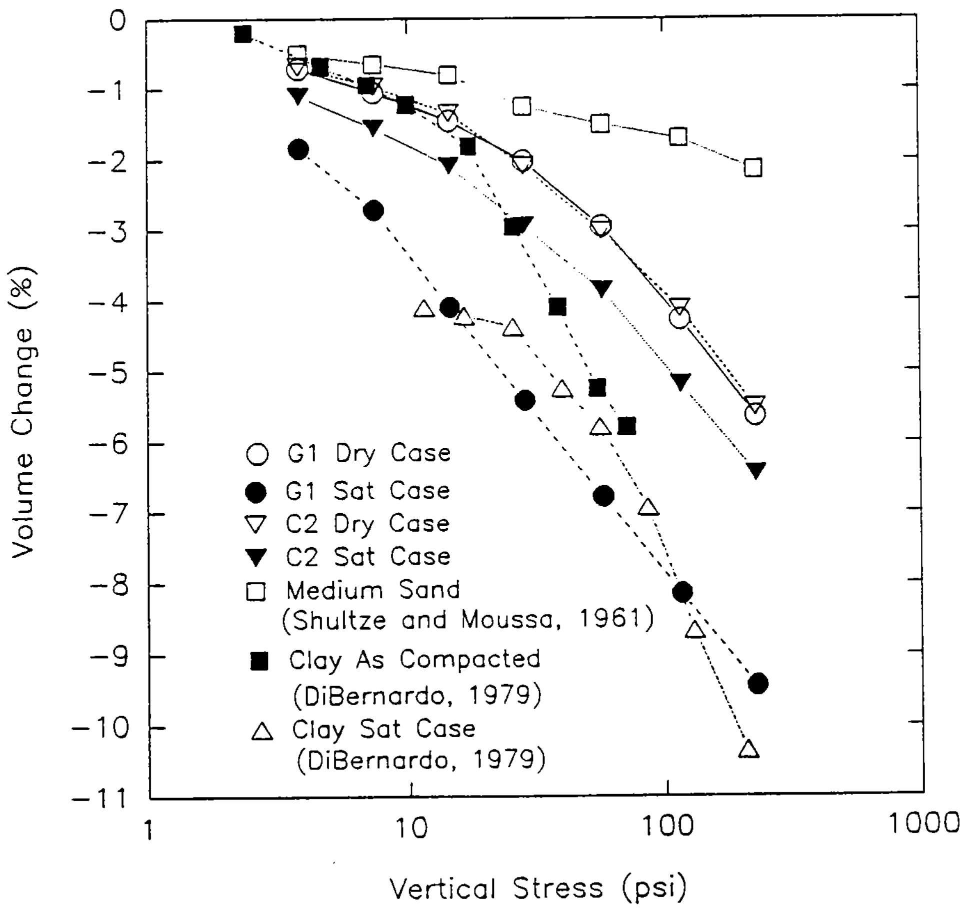
[1] Classification according to Unified Soil Classification
[2] Typical properties from NAVFAC Manual DM 7 (1982)

Compressibility

The compressibility of a fill determines the magnitude of vertical deformation at the surface of the fill. Initially, dry materials were placed in the consolidometer and compressed until they yielded 95% of maximum dry density determined by either the standard compaction test (ASTM D 698) or by vibration method. Two samples of each foundry sand were tested, one in a dry condition and the other one in a saturated condition. The results are compared with a highly plastic clay and a medium fine sand (Figure 6). Our samples seem to be more compressible than medium fine sand. However, they are comparable to clay at low stresses but are less compressible at high stresses under both as-compacted and saturated conditions. The foundry sand G1 is found to be more compresible than C2 because C2 is more rounded than G1. Sands with angular particles are known to be more compressible than well-rounded sands, because the sharp edges become overstressed during movement and reorientation of the particles and hence break to produce compression (Roberts and DeSouza, 1958).

FIGURE 6

COMPRESSIBILITY OF G1 & C2 WITH
COMPARISON OF REPORTED VALUES



Permeability

Permeability was measured on two samples using a flexible wall permeameter. The samples were first compacted near the optimum moisture content and then placed in the permeameter. Saturation was accomplished using backsaturation techniques and the permeability of the samples was then measured using the falling head method. Permeability of G1 at 97% of maximum dry density was found to be 3.36×10^{-5} cm/sec, while permeability of C2 at 98% of maximum dry density was 1.02×10^{-3} cm/sec. Typical permeability of a uniform fine sand is reported to be 4×10^{-3} cm/sec (Hough, 1969). Because chemically bonded samples do not use bentonite, the permeability of C2 was found to be in close agreement with the reported value of the natural sand. The permeability of G1 was found to be typical of natural silt.

Considering the above factors , waste foundry sand is considered suitable for embankment. Since CBR and permeability values were found to be low compared with conventional materials, their use in pavements is not promising. The moisture density relationship for G1 suggests that such material should be compacted on the wet side as this will result in minimum swelling and increased CBR. The CBR curves also suggest that it will be advantageous to compact on the wet side. Chemically bonded sands may be compacted even in flushed conditions as inferred from CBR and moisture-density relationships. Although, the compaction curve shows a slight decrease in density at the highest water contents, field compaction data reported on cohesionless materials indicate that field compaction curves do not show such a decrease (Highway Research Board, 1952)

Conclusions

From the results of the study reported herein, the following conclusions can be drawn:
- Large volume applications of waste foundry sand can be used in embankments, provided these wastes fall in Type III or Type IV as previously discussed.
- With some blending of coarse sand , WFS has potential to be used as snow and ice abrasives.
- Although physical properties of the foundry sands tested do not conform to ASTM specifications for fine aggregate in Portland cement concrete and asphalt concrete they may be used when blended with conventional aggregates.

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EFFECT OF FLY ASH QUALITY ON CONCRETE DURABILITY

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ABSTRACT

Three class F fly ashes collected from the same power plant, at different times, were investigated to determine the effects of variable loading conditions on the properties of the collected ash and the durability of fly ash concrete. The chemical, mineralogical and physical properties of the as-received ashes were studied using selective dissolution, quantitative x-ray diffraction, x-ray sedimentation and chemical analysis. The impact of varying the as-received ash properties on hardened concrete quality were assessed through monitoring the corrosion potential of steel bars embedded in fly ash concrete.

The results indicate that variation in the loading conditions affected the pozzolanic activity of the ashes. The pozzolanic activity was found to be a function of the fly ash oil content, glass content and fineness. Open-circuit potential measurements for the bar steel embedded in fly ash concrete showed that durability of fly ash concrete is affected by the quality of fly ash.

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Three class F fly ashes collected from the same power plant, at different time intervals, were investigated to determine the effects of variable loading conditions on the properties of the collected ash and the durability of fly ash concrete. The chemical, mineralogical and physical properties of the as-received ashes were studied using selective dissolution, quantitative x-ray diffraction, x-ray sedimentation and chemical analysis. The impact of varying the as-received ash properties on hardened concrete durability were assessed through monitoring the corrosion potential of steel bars embedded in fly ash concrete.

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INTRODUCTION

Less than six percent of the total fly ash produced in the United States is currently used in cement and concrete products [1]. In order to increase the utilization of ash in concrete structures, a better understanding of the technical requirements for a satisfactory materials performance is needed. There are several parameters that affect the quality of a particular ash; namely, furnace design, burning procedure, pulverization process of coal, furnace load condition (variable loading as opposed to base of full load condition), collection system and coal source [2].

For a particular power plant unit, all of the above variables are maintained constant with the exception of the loading condition. Variation in the loading condition results from several factors the most important of which are maintenance processes and variations in power demand. This study was initiated to address the effects of variable loading condition on the collected ash properties. In addition, the effect of the collected ash quality on blended concrete durability was also examined.

EXPERIMENTAL TECHNIQUES

Chemical Analysis

The oxide content for the as received ashes and portland cement (Type II) were determined using atomic spectrophotometer and wet chemical analysis (Table 1). The total organic carbon content was determined using an automated total organic carbon analyzer (Doharmann Model DC-80) in which the carbon content is measured through a nondispersive infrared detector (Horiba, dual beam Model PIR-2000). Oil content was determined using a freon extraction procedure outlined in [3].

Mineralogical Analysis

A quantitative analysis of the mineralogical composition of fly ash was done using an x-ray diffractometer (GE XRD-700) equipped with a goniometer. The operating voltage and current were 45 KV and 30 mA. The scan rate used to identify the primary crystalline compounds in the ashes was 2°/min over an angle range of 6 to 65°. The amount of each crystalline phase was quantified using sythentically produced phases and calcium fluoride as an internal standard. The scan rate used in this case was 0.4°/ min. and the mass of each phase in the fly ash was determined through the integrated intensity technique [4,5] and comparing the integrated intensity in the fly ash sample with that obtained from the synthetic phase.

Physical Properties

The fineness and particle size distribution of the as-received ash were determined using wet sieve analysis (ASTM C430) and a Sedigraph particle size analyzer respectively.

In the latter, the sedimentation rate of the ultrasonically dispersed fly ash particles in an inert hydrocarbon oil is measured through a collimated x-ray beam. The concentration of the particles remaining in suspension is determined as a function of sedimentation depth and time. The distribution of particle mass at various points in the cell affects the number of x-ray pulses reaching the detector which in turn is correlated to the particle diameter distribution and the percent mass at a given particle diameter. The logarithm of the transmitted intensity is electronically generated, scaled and presented as mass percent fines in terms of Stokes' equivalent spherical diameter.

Pozzolanic Activity

In evaluating the pozzolanic reactivity of the fly ashes, selective dissolution and mortar cube strength were used. As the mortar cube strength is a function of porosity, which in turn is a function of water content, mortar cubes used for assessing the pozzolanic potential of the fly ashes were prepared using a constant water to cement ratio of 0.484, cement content of 500 grams and a sand to cementitious ratio of 2.75. 20% portland cement replacement level was used for the fly ash mortars. The cubes were demolded after 24 hours from casting and placed in a saturated calcium hydroxide solution maintained at room temperature until tested at the selected times.

Selective dissolution was performed on plain and blended paste using picric acid and methanol [6,7]. In this method, picric acid dissolves all the hydration products and unreacted cement while leaving the unreacted pozzolana undissolved. A detailed description of the method and its validity is presented in [6,7]. The cement/fly ash mixtures were prepared in a weight ratio of 80/20 with a water to cementitious ratio of 0.45. The paste was casted in PVC bottles, demolded after 24 hours and placed in a saturated calcium hydroxide solution. At the selected testing age, the paste was placed in ethanol/acetone mixture for 2 hours to cease hydration. The paste was then placed in an oven set at 50°C for 24 hours. A roller mill was used to grind the paste to a fine particle size. The amount of unreacted fly ash was then determined for selected hydration times.

Durability

Monitoring of corrosion potential of the steel reinforcement embedded in concrete as a function of exposure time, to a sodium chloride solution, was used to determine the effect of fly ash quality on the durability of concrete. The specimens were prepared using sand blasted steel bars (0.5 in. diameter) embedded at the center of plain and fly ash concrete cylinders (4x6 in.), maintaining a clearance of 2 in. from the concrete bottom. Each steel bar was covered with an epoxy layer 0.25 in. in length at lower end of the reinforcement embedded in the concrete and 1 in. in length at the concrete-air interface (0.5 in. below and 0.5 in. above the interface). The epoxy layer was applied in order to eliminate the possibility of crevice corrosion at those locations. The properties of the materials used and the mix design are given in Table 2 and 3. The specimens were demolded 24 hours from casting and were then placed in a saturated calcium hydroxide solution for seven days. They were then submerged in a 5 w/o sodium chloride solution. The solution level was at an elevation of 2.25 in. (measured from the bottom of the concrete cylinder). The potential was measured using a high impedance voltmeter and a saturated calomel electrode (SCE).

RESULTS AND DISCUSSION

Chemical Analysis

The oxide content, carbon content, oil content and loss on ignition (LOI) for the three fly ashes are given in Table 1. The ashes satisfy the chemical requirements of Class F fly ash [8]. Variation in loading conditions did not seem to have significant effects on the oxide content of the ashes. This finding was expected due to the fact that all of the ashes came from the same power plant burning unit and therefore the same coal source. The impact of variable loading conditions was more significant on the carbon content, oil content and loss on ignition. FA#1 and FA#2 had similar carbon content and LOI, while FA#3 was higher in both properties. In addition, FA#2 showed a substantially higher oil content than the other two ashes. Although the loss on ignition of fly ashes is generally considered to be closely related to the carbon content, higher LOI values are possible due to the loss of chemically combined water and carbonate decomposition at the LOI test temperature. High carbon content has adverse effects on workability and air-entrainement capabilities in concrete mixes incorporating fly ashes [9].

Mineralogical Properties

Identification and quantification of the crystalline content of the fly ashes using quantitative x-ray diffraction showed that the major crystalline phases are alpha quartz, mullite, hematite and magnetite, with mullite and quartz being more abundant than the crystalline forms of iron oxide. Difficulties were encountered in quantifying the hematite content due to the fact that the total iron oxide content, as determined from chemical analysis, was low (5.26 w/o and 6.6 w/o). Therefore, it was decided to quantify the hematite phase by taking the difference between the total iron oxide content from the chemical analysis and the magnetite content determined by QXRD. The validity of this approach is based on the assumption that iron oxide exists only in these two crystalline phases. The total crystalline phases content varied between 33 to 40 w/o. Since these crystalline minerals are nonreactive at ordinary temperatures, their presence in large quantities, at the expense of amorphous phases, tend to reduce the pozzolanic reactivity of fly ash. The reactive or amorphous content of the three fly ashes are given in Table 4. The results indicate that variable loading conditions had moderate effects on the reactive content of the fly ashes.

The effect of fly ash quality on concrete durability is directly related to the reactivity of the fly ash with the cement paste. The amorphous content (aluminosilicate glass), reacts with the cement paste hydration product, specifically calcium hydroxide, to form denser hydrates of calcium silicates and aluminates. Such a reaction has the effect of reducing concrete permeability. On the other hand, the crystalline phases in the fly ash act as a filler material and do not participate in the hydration process or pore refinement.

In addition to the effect of the pozzolanic reaction on the pore refinement of blended concretes, there are other advantages to the lime consuming reaction. The strength contributing potential of calcium hydroxide is low. This is because calcium hydroxide crystals are large in size and more susceptible to leaching. In addition, calcium hydroxide has adverse effects on concrete chemical durability to acidic environments. If the pozzolanic activity of a fly ash is governed solely by the reactive amorphous content, it would therefore be expected that FA#1 and FA#3 have a better strength and durability performance than FA#2.

Physical Properties

The percentage of particles finer than three arbitrary chosen diameters (45μ, 20μ and

10μ) obtained from x-ray sedimentation curves are presented in Table 5. The percentage of particles retained on 45μ sieve obtained from wet sieve analysis are also included for comparison purposes. Data from both techniques show the same trends; however, results from wet sieve analysis for particles greater than 45μ are generally higher than those from x-ray sedimentation. In addition, differences in particle sizes revealed from wet sieve analysis for different fly ashes are smaller than from x-ray sedimentation. For the three particle diameters considered in Table 5, FA#2 showed the lowest percent fines. Increasing the coarseness of a fly ash has an adverse effect on water requirements in a concrete mix, if a specific workability is desired. Increasing the water content would increase the concrete permeability and therefore affect concrete durability. In addition, since the hydration processes of cementitious particles occur on the surface of solid phases, through a process involving diffusion and dissolution, it then implies that smaller particles would react at a faster rate.

Selective Dissolution

Variation of the reacted fraction of fly ashes as a function of hydration time are shown in Figure 1 where it can be seen that FA#1 and FA#3 had higher reactivity than FA#2. The data presented in Figure 1 are determined on ignited basis. This explains why FA#3 had a high reacted fraction though it had the highest carbon content. The relative rates of hydration of pozzolanic and cementitious materials depend in general on their particle size distribution and composition/content of the amorphous phase, both of which control the activation energy available for the hydration reaction. Since both FA#1 and 3 had similar glass content and particle fines, it is therefore to be expected that they would have similar reacted fractions as evident through the results presented in this section.

Mortar Cube Strength

The compressive strength of blended mortar cubes (presented as a ratio with respect to control mortar cube strength) at different hydration times is shown in Figure 2 where it can be seen that cement mortars had a higher compressive strength up to 28 days of hydration. Among the blended mortars, FA#2 showed a consistently lower strength ratio than FA#1 or FA#3 at all ages. These results are in agreement with the selective dissolution results, where FA#2 had the lowest reacted fraction.

Durability

Variation of the open-circuit potential of reinforcing steel in concrete as a function of exposure time is presented in Figure 3. Each set of data has four specimens representing the same condition. The time to the reinforcement corrosion initiation was evaluated in accordance to the criterion set in ASTM C876 [10]. In accordance to this criterion, the probability of reinforcement corrosion is greater than 90% if the half-cell potential is greater than -293 mV (vs SCE). The open-circuit potential data showed that FA#1 and 2 crossed the threshold potential for corrosion initiation at longer times than plain concrete specimens. FA#3 experienced a drop in potential 50 to 100 days earlier than the control. In addition to longer exposure times prior to experiencing a potential drop, fewer specimens in FA#1 and 2 mixes experienced such a drop compared to specimens in the control and FA#3 sets. Displaying the fraction of specimens in each set, experiencing the potential transition, as a function of exposure time is shown in Figure 4. The better performance of FA#1 is evident from Figure 4 where only one specimen in this set showed a potential transition over a period of 550 days. All specimens in the control mix and FA#3 experienced a drop in potential by 500 and 350 days respectively. Potential measurements therefore indicate that the quality of fly ash has an effect on the time required for corrosion initiation. Longer exposure times are required as to assess the performance of blended concrete in terms of corrosion propagation.

SUMMARY AND CONCLUSIONS

Summarizing the effect of variable loading in a power plant on the as-received fly ash quality, the results of this study indicate that the oxide content and glass content of the ashes did not show significant changes. Oil content, carbon content, particle fineness and LOI were sensitive to variable loading conditions. Pozzolanic reactivity of the collected ashes was affected primarily by the oil content and fineness as evident from the low reacted fraction and strength ratio of FA#2. It is postulated that oil might engulf the fly ash particles thus providing a barrier towards the fly ash particles participating in the pozzolanic reaction.

Corrosion measurements showed that the carbon content affected the time to corrosion initiation in reinforced concrete. Reinforced concrete made with FA#3 (carbon content of 5.83 w/o) experienced transition in potential, below the threshold level, at exposure times shorter than those reported for plain reinforced concrete.

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TABLE 1 Chemical Analysis Of Fly Ash And Portland Cement

Compound	Fly Ash #1	Fly Ash #2	Fly Ash #3	Cement
SiO_2	50.02	50.40	48.34	19.88
Al_2O_3	31.42	29.30	31.00	5.05
Fe_2O_3	6.60	6.82	5.26	3.68
MgO	1.05	0.65	0.74	0.66
CaO	2.12	1.84	1.84	62.98
Na_2O	0.06	0.08	0.11	0.20
K_2O	1.61	1.51	1.52	0.53
SO_3	0.59	0.58	0.64	2.52
LOI(800°C)	3.74	4.09	6.70	1.37
Carbon*	2.81	3.92	5.83	___
Oil (ppm) Content	180	980	260	___

TABLE 2 Material Properties Used in Blended and Plain Concrete

CEMENT	ASTM Type II Portland cement (sp. gr. = 3.15)
FLY ASH	Three different fly ashes were used for 20 w/o replacement of the cement. The specific gravities are as follows: FA#1 = 2.22 FA#2 = 2.21 FA#3 = 2.19
COARSE AGGREGATE	Crushed Florida Limestone with a nominal maximum size of 1/2 inch. Grade size #7. BSG(SSD) = 2.50. AC=3.0%. DRUW = 90 lb/cy.
FINE AGGREGATE	Florida lake sand. FM = 2.35. BSG(SSD) = 2.63. AC = 0.5%.

TABLE 3 Mix Design For Plain and Fly Ash Concrete

Materials	Control (lb/cy)	Fly Ash #1 (lb/cy)	Fly Ash #2 (lb/cy)	Fly Ash #3 (lb/cy)
Water	275	275	275	275
Cement	611	489	489	489
Fly Ash	---	122	122	122
Coarse aggregate	1489	1489	1489	1489
Fine Aggregate	1519	1477	1476	1475

TABLE 4 Glass Content of Fly Ashes

FLY ASH Number	Total Crystalline Phases Content [TCP] (w/o)	LOI (w/o)	Glass Content (100-TCP-LOI) (w/o)
FA#1	33.08	3.74	63
FA#2	39.88	4.09	56
FA#3	34.34	6.7	59

TABLE 5 Particle Fineness from X-Ray Sedimentation and Wet Sieve Analysis

Specimen Number	X-Ray Sedimentation			Wet Sieve Analysis
	% Finer			% Retained on Sieve No.325
	<45μ	<20μ	<10μ	(<45 μ)
FA#1	95	70	43	16.95
FA#2	89	65	37	19.18
FA#3	93	70	40	17.87

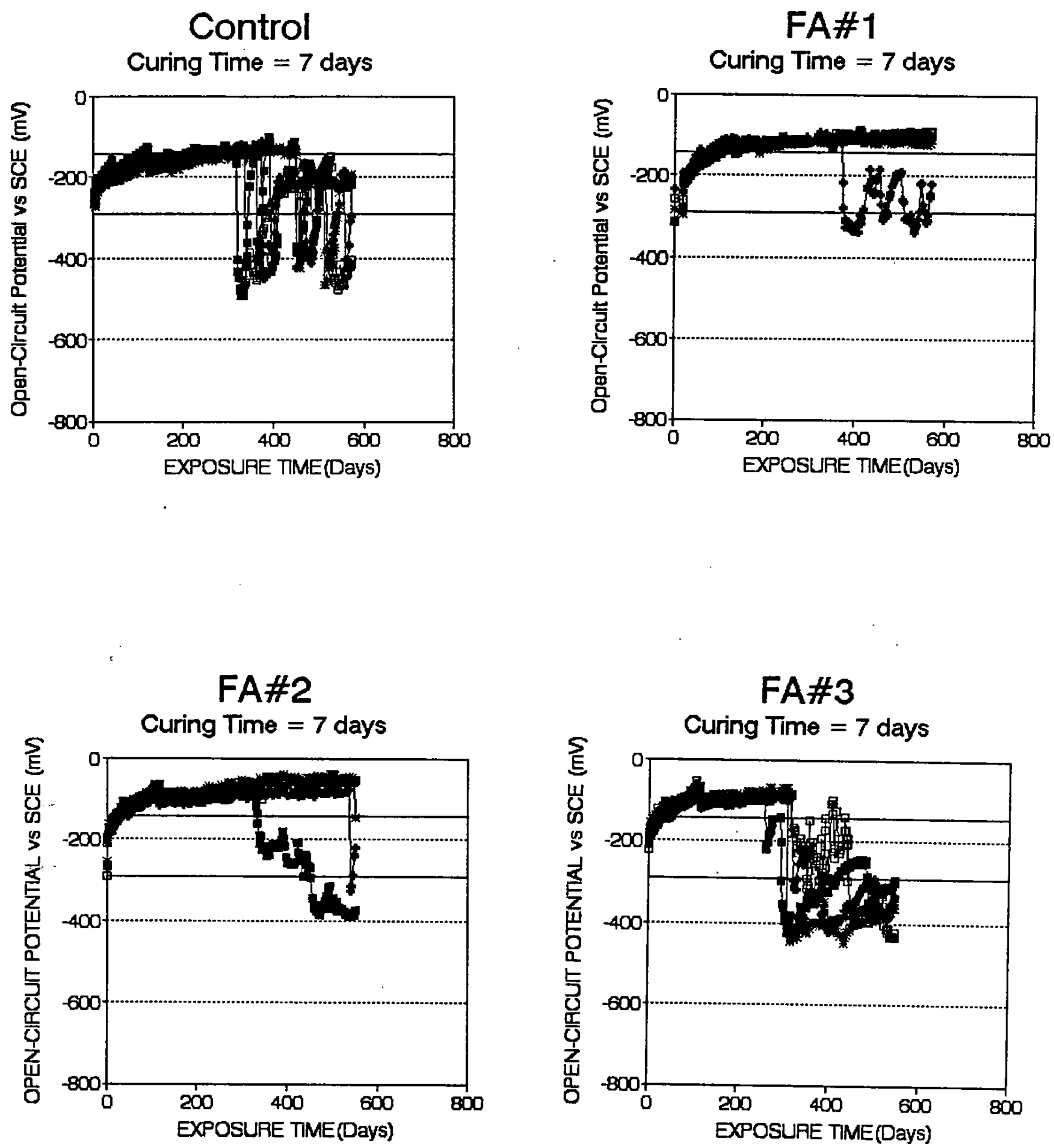

Figure 3- Open-circuit potential measurements of reinforcing steel in plain and fly ash concrete as a function of exposure time to a sodium chloride solution.

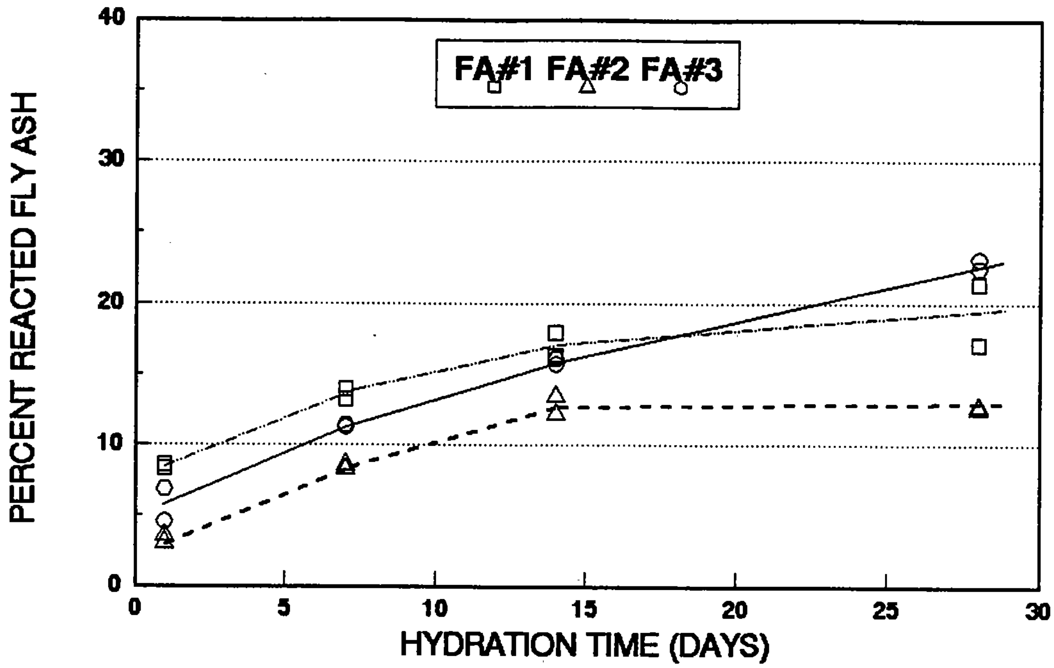

Figure 1- Percent reacted fly ash as a function of hydration time.

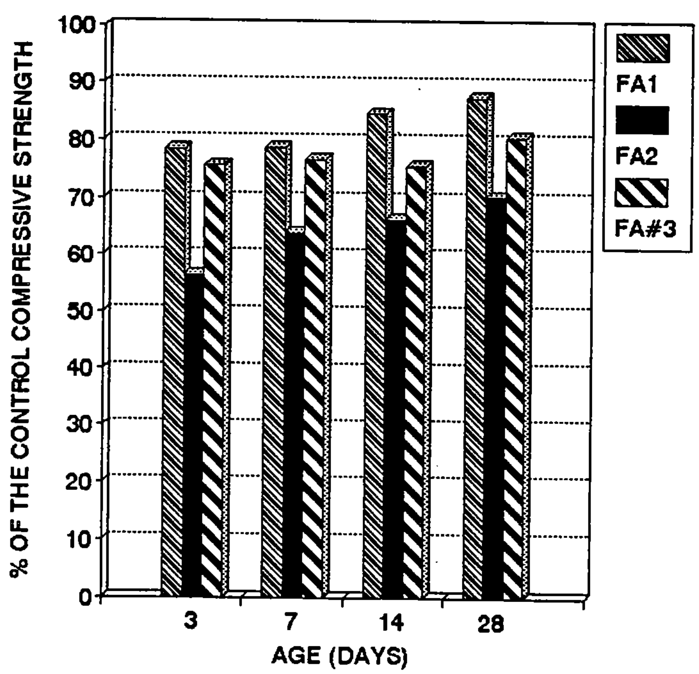

Figure 2- Compressive strength ratio of mortar cubes as a function of hydration time.

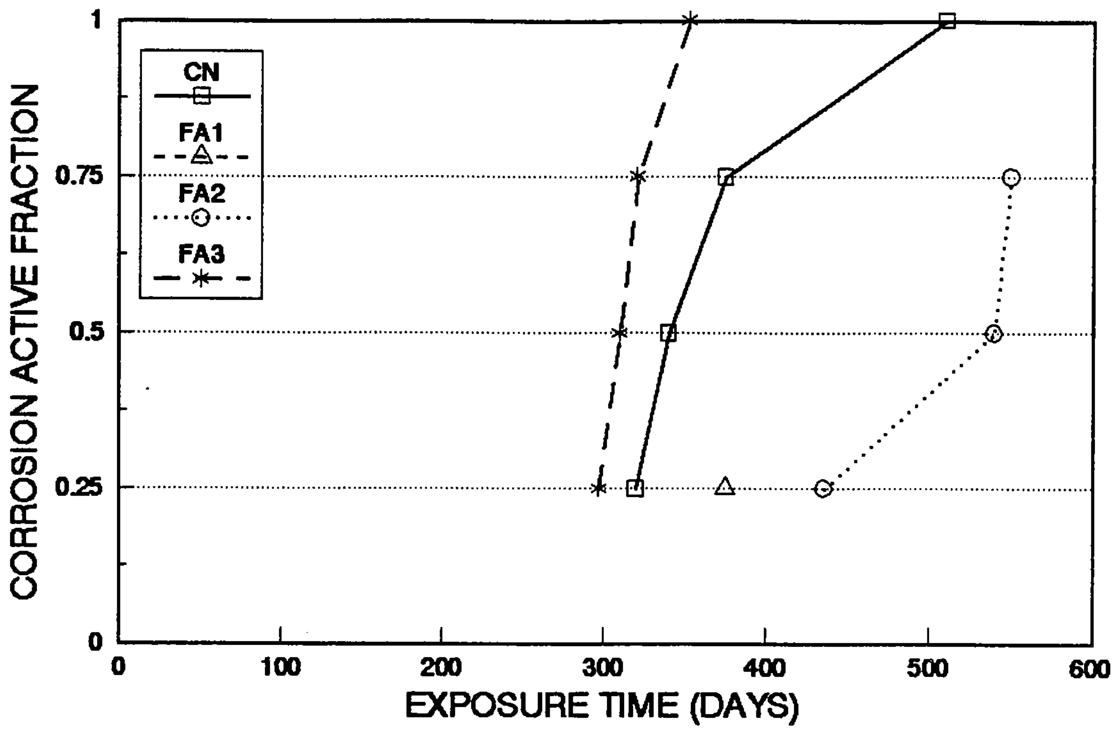

Figure 4- Corrosion active fraction of reinforcement in plain and fly ash concrete as a function of exposure time to sodium chloride solution.

A Roadway Problem In A Cavernous Karst Environment at the Florida Caverns State Park

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ABSTRACT

The Florida Caverns State Park, in Jackson County, Florida, has within its limits a unique network of caves, one of which was thought to run under a paved park road. The road has been used for several years. Recently, some cave authorities expressed concerns about the structural integrity of China Cave, and consequently, the safety of the road above it.

The potential for catastrophic collapse of the roadway led to an investigation of a network of cave passageways in November and December, 1992. The survey, intended to verify and update a 1973 survey, located four cave entrances and possibly as many as four vertical shafts to the surface. There is a topographic depression - believed to be a collapsed cave - in a lightly wooded area on the north side of the road. Surveyors mapped the main passageway of China Cave, and found that not only does it underlie the roadway, but as little as seven feet of rock, base-coarse material, and asphalt lie between the cave ceiling and the road.

Remediation may call for fill material in areas showing signs of settling, and for sealing the asphalt roadway to help prevent surface water from percolating downward through the limestone into the cavern. The Park could reroute the road, or limit the weight of vehicles using the road. Lastly, and perhaps most important, China Cave, and the roadway above it warrant close monitoring for any physical changes.

Introduction

Of the more than 113 State Parks in Florida, only Florida Caverns State Park in Jackson County, is dedicated to the conservation and preservation of its unique network of caves (Figure 1). China Cave, one of the many karst features found in the Park, underlies a paved Park road. Recently, after touring China Cave, a cave authority with the National Park Service expressed concern about the structural integrity of the cave and the road above it.

The Florida Park Service initiated an inspection of the cave because of their concern about a potential catastrophic collapse of the roadway. During November and December, 1992, a team of cavers from the Florida Speleological Society, operating under a permit to survey issued by the Florida Department of Natural Resources (DNR), mapped the passageways of China Cave. Their primary goal was to locate China Cave in relationship to the road, and determine the thickness of the limestone above the cave. The new cave map would replace an older (1973), less accurate map. In addition, Park officials hoped for reassurances that the limestone overlying the cave was structurally sound. They wanted to know if the limestone could withstand the forces exerted on it from the vehicular traffic on the roadway above.

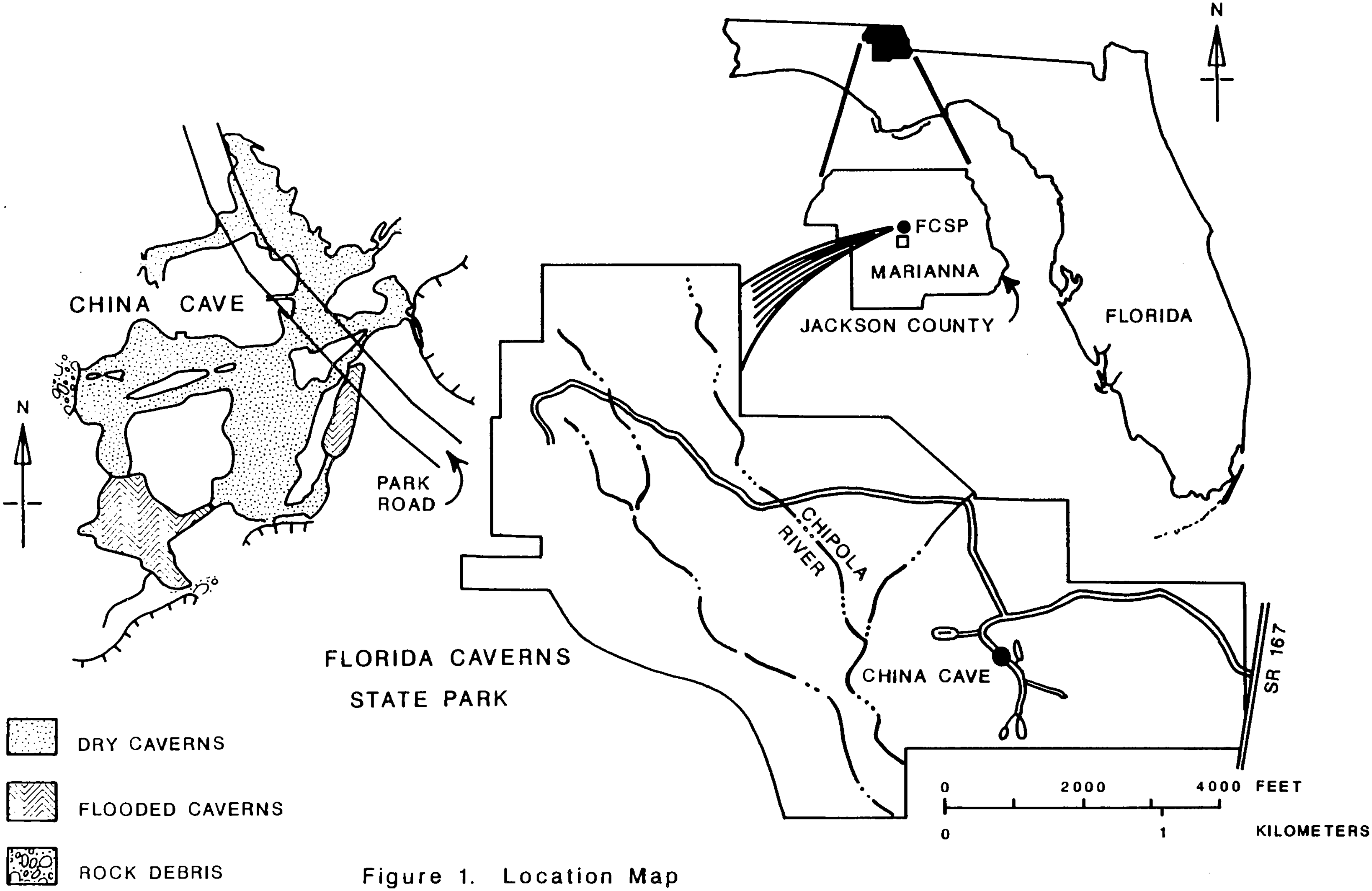

Figure 1. Location Map
(modified from Krause, 1992)

Geomorphology

The tri-county region of Holmes, Washington, and Jackson Counties have lower topographic elevations than other neighboring counties. Most of this region has been lowered by karst dissolution in the underlying limestone. Puri and Vernon (1964) termed this geomorphically distinct area, which extends north into Alabama and Georgia, the Marianna Lowlands (Figure 2). Topographic elevation of the lowlands, with some exceptions, are lower than those of the New Hope Ridge and Grand Ridge to the south towards the Gulf of Mexico (Puri and Vernon, 1964).

Amid the Marianna Lowlands are a number of hills, remnants of the Northern Highlands Physiographic Zone (Puri and Vernon, 1964). The highlands are essentially a continuation of each other; and they span the entire northern half of the Florida panhandle. They are separated by the elevationally lower Marianna Lowlands. The once high, broad elevations of the region have, over time, been lowered by karst dissolution and by erosion from rivers and streams.

The New Hope Ridge and Grand Ridge, which are separated by the valley of the Chipola River, form the southern boundary of the Marianna Lowlands. At one time, the two ridges were joined together to form one higher ridge in southern Jackson and adjoining counties (Puri and Vernon, 1964). However, over eons of time the ridge succumbed to the erosional processes of a large river system (Moore,1955).

The Florida Caverns State Park encompasses 1,280 acres of land in sections 20-22, and 27-29, of Township 5 North, Range 10 West in the Marianna Lowlands. Karst development is extensive throughout the region, including the Park. The maximum elevation in the Park is 180 feet above mean sea level (MSL). A minimum elevation of 70 feet above MSL occurs in the southern part of the park along the Chipola River. Over time, the Chipola has eroded and dissolved its way through the limestone bedrock. Natural bridges have formed where the river siphons underground. The river flows in a northwest to southeast direction through the Park following the natural joint pattern of the limestone. Much of the groundwater from higher elevations drains through a network of subterranean caverns into the river. Today, there are both dry and wet caves located in and outside the Park boundaries. The karstic nature of the environment of the Park and surrounding region is evident in the landforms that are present throughout the area.

Geologic Structure

Florida Caverns State Park is located on the southern flank of the Chattahoochee Anticline (Veach and Stephenson, 1911; Puri and Vernon, 1964). This positive structure, which extends into Georgia and Alabama, trends in a southwest-northeast direction. The structure plunges to the southwest (Puri and Vernon, 1964; Schmidt, 1984). The crest of the anticline exposes the Eocene and Oligocene carbonates, and younger siliciclastic sediments in the Washington, Holmes, and Jackson tri-county area. Younger sediments pinch out or are truncated along the flanks of the arch (Schmidt, 1984).

Geology

The oldest formation to crop out in the Florida Caverns State Park and the one in which caverns are developed, is the Eocene Ocala Limestone. Moore (1955) reported the thickness of the Ocala Limestone in Jackson County to be about 220 feet. In the Park, approximately the upper 15 feet of Ocala Limestone has some distinguishing characteristics. Here, the uppermost Ocala is soft and friable. It has a granular texture, and is very porous and permeable due to the abundance of bryozoa and foraminifera, including *Lepidocyclina chaperi,* which comprise the rock. The Ocala Limestone is white to tan in color, and dips to the south in the central and western parts of the county (Moore, 1955). It forms an integral part of the Floridan aquifer system.

The Oligocene Marianna Limestone crops out on the hills bordering the Park, and sporadically on some hilltops in the Park. The Marianna lies unconformably above the Ocala Limestone and below the Oligocene Suwannee Limestone, where that formation is present. Where the Suwannee is absent the, Marianna is overlain by younger undifferentiated siliciclastic sediments. The Marianna Limestone is light

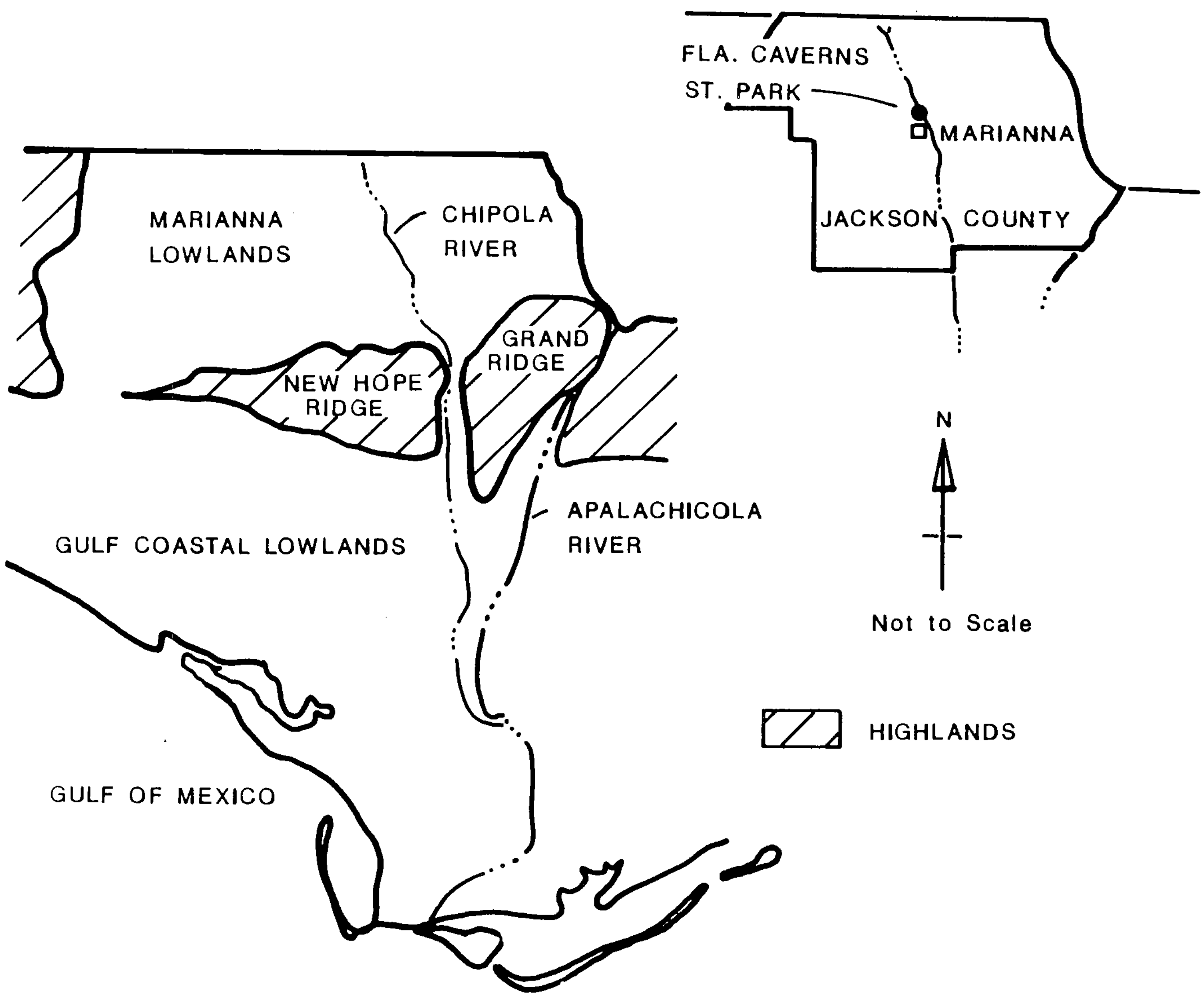

Figure 2. Geomorphology

tan to white in color, usually quite massive, and less permeable than the underlying Ocala Limestone. It can, however, be very fossiliferous, containing abundant large foraminifera such as *Lepidocyclina mantelli*. Also present in the limestone are large numbers are *Operculinoides dius,* and *Pecten* sp. Moore (1955) reports the thickness of the formation is up to about 40 feet in outcrop. He further states that this limestone formation dips to the south about 13 feet per mile, then plunges to 64 feet per mile at roughly the south side of Township 4 North. The Marianna Limestone forms a less permeable part of the Floridan aquifer system in the panhandle region of Florida.

In the Park, the Ocala Limestone is often overlain by a residuum of quartz sand, clayey sand, and organics. Thicknesses of these sediments range from non-existent up to at least 80 inches in some locations (Soil Conservation Service (SCS), 1979). The SCS samples sediments to depths no greater than 80 inches. Algae and lichens often cover exposures of limestone.

The Problem

Groundwater, flowing through the joints and fractures in the limestone, has led to the formation of a system of cave passageways in the Park. There are two zones of cave development, the upper zone is 3- to 15-feet above the present mean stage of the Chipola River. The second zone of passage development (wet caves) is about 25- to 30-feet below the mean river stage (Al Krause, personal communication, 1992). The dry caves developed under shallow phreatic conditions mostly, when the water levels were higher. Evidence for a phreatic origin include solution pockets on the walls, wall partitions, and mazes, all of which are common in the caves in the Park (Bretz, 1942). With lower water levels, wet caverns developed to provide drainage to the river. Occasional vertical shafts breach the limestone, thereby connecting dry caves with submerged caverns. A network of Park roads overlie some of these caverns.

China Cave, last mapped in 1973, underlies one of the Park's main roadways (Figure 3). The entrance to the cave is about one-half mile northeast of the Chipola River in the river's floodplain. China Cave, with its maze of passages developed along bedding planes, formed along primary and secondary fractures. There are four entrances to the cave and two- to three hundred feet of passageway. The Florida Speleological Society's unpublished cave records show that vehicles operating on the road above China Cave generate a rumbling noise heard inside the cave.

Mr. Ron Kerbo, a cave specialist with the National Parks Service, toured some of the Park's caves during a visit in 1992. He questioned the structural integrity of the roof in China Cave, particularly the area under the road (Figure 4). He thought the potential for collapse of the limestone under the roadway was high. This, in turn, raised questions by Park authorities about the caves structural integrity. The overriding question remained: is the limestone overlying China Cave strong enough and thick enough to carry vehicular traffic safely? If it is, then there really isn't a problem. If not, is the site a geologic hazard to park patrons who use the road? And if it is a hazard, then how does the Park resolve the situation and still hold to the premise of preservation of its natural resources?

The Project

Initially, the Park service tried to define the system of caverns underlying the road with ground penetrating radar (GPR). A review of the U.S. Department of Agriculture's Soil Conservation Survey of Jackson County (1979) shows the soils near China Cave to be about 30- to 45-percent clay and silt, and often wet. Wet, clayey and silty soils tend to cancel the GPR's effectiveness. Results of the GPR survey were negative -- it did not show the presence of any caverns. To confirm the extent of the cave, officials needed an alternate methodology.

With entrances to the cave so accessible from the road, Park management, on the advice of Mr. Ron Kerbo and others, decided to inspect China Cave and have it resurveyed. A new survey, it was thought, would verify the existing 1973 survey. If any discrepancies in the earlier survey were found, they would be corrected. The primary goal for Mr. Al Krause and his survey team was to determine the exact location of China Cave, both horizontally and vertically, relative to the overlying road. Next, they were to

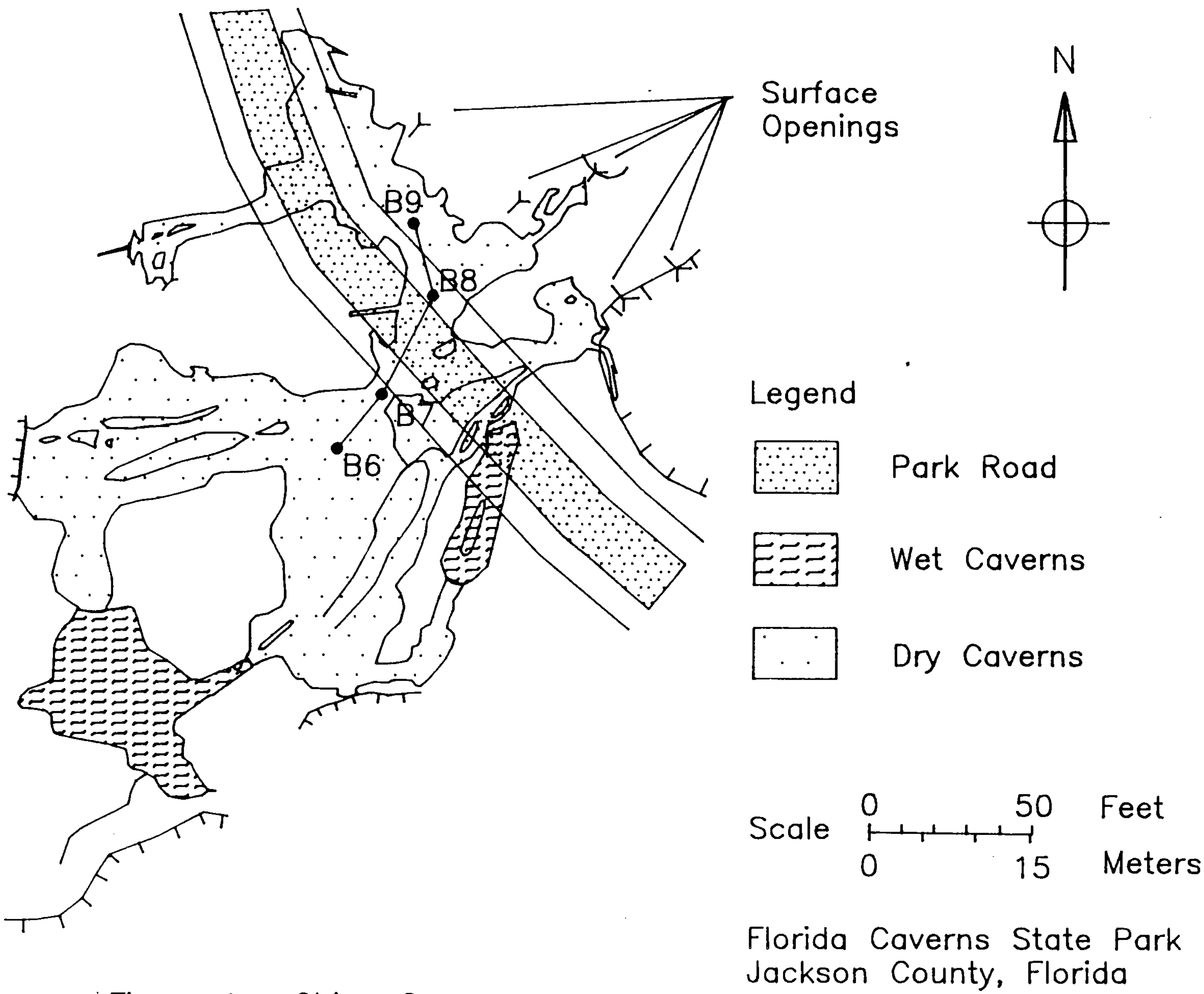

Figure 3. China Cave

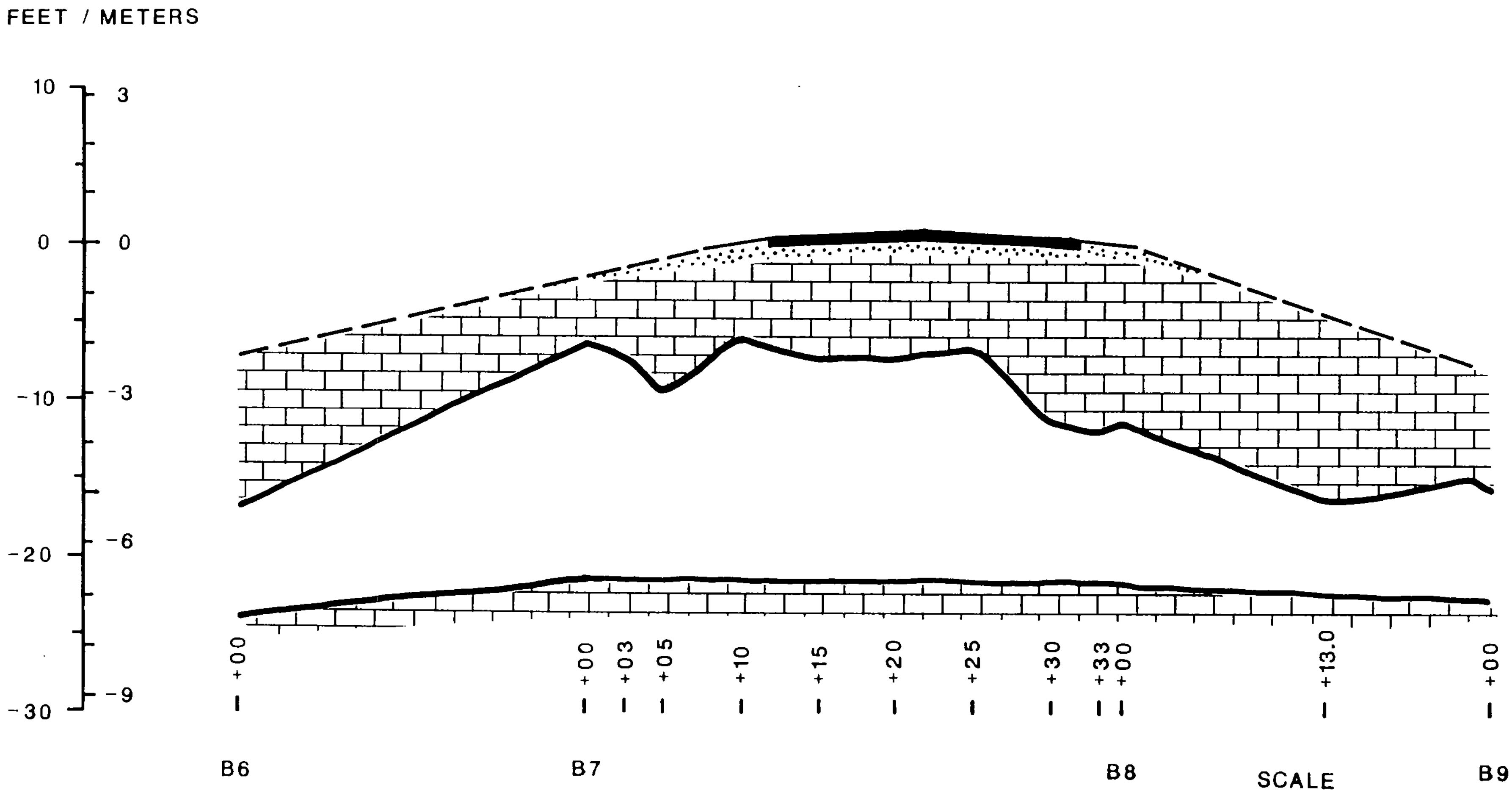

Figure 4. Cross-Section B6 to B9

attempt to determine the thickness and the integrity of the limestone above the cave.

On November 7 and 8, 1992, a survey team began mapping China Cave. Their equipment included Sisteco compasses and clinometers (accurate to within 0.5 degrees), Abney levels, Jacob's staffs and fiberglass engineers tapes. The team leader began by setting a temporary survey marker into the edge of the roadway to serve as a point of origin. Using an clinometer (degrees and percent), a compass (azimuth), and the measuring tape, the survey team located all geomorphic features on either side of the road. They surveyed closed loops using fore- and back-sights to insure accuracy. The survey then proceeded southwest through a lightly wooded area into the cave entrance.

Inside the cave, the team set stations along the transect line with reflective markers. On-station, width measurements of the cave were taken at 90 degrees left and right of the transect, and height measurements at one-foot and four-feet above the cave floor. Surveyors also recorded the bearings of primary and secondary joint patterns. While one team member drew profiles of the cave at various stations, other members of the team mapped the cave. Surveyors used a measuring tape attached to four-feet extensions of PVC pipe (Jacobs' staffs) to measure and record the height of the cave at several stations. Again, accuracy of the survey was insured by closing loops and making fore- and back-sights. After this surveying trip the data was compiled and analyzed for completeness. A second surveying trip on December 19 and 20, supplemented information obtained during the first trip and helped to clarify any remaining questions.

Results

Results of the survey show that the road does, indeed, overlie the cave, and that the thickness of the cave ceiling is greater than previously thought. There is at least 6.9 feet of rock and fill between the cave ceiling and the road. Collapse of what was probably an adjacent cave occurred sometime in the distant past. Visual evidence of that collapse is present northeast of the road. A new map of the cave shows greater detail and is more accurate than the old map. Visual inspection of the cave ceiling did not show signs of deterioration from the roadway (Krause, 1992). However, there is an asphalt patch in the road which indicates, perhaps, that some settling has taken place.

Discussion

There are several potential remedies to the problem, although most run counter to the mission of the Park. Some remedies might include building supports in the cave or filling the cave. Either of these solutions would destroy the cave. Other answers to the problem may include rerouting the road or building a bridge structure over the cave area. Sealing the road surface will help reduce the amount of surface water penetrating the road and reaching the limestone underneath (AL Krause, personal communication, 1992). The result of sealing the roadway would be to lessen the amount of dissolution of the rock directly under the road. Whatever the answer, careful consideration must be given to the site geology.

Until a determination is made as to what remedies should be undertaken, the roadway will continue to be used. Perhaps the Park Service should limit the weight of vehicles using the road. In the interim, the site should be closely monitored for settling or cracking of the pavement, or any other indications of cave ceiling failure.

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Cover-Subsidence Sinkhole Evaluation
State Road 434 - Longwood, Florida

By Jon Foshee[1] and Brian Bixler[2]

ABSTRACT: This paper presents the results of a cover-subsidence sinkhole evaluation. In this case, the sinkhole activity caused a slow, gradual settlement of the State Road 434/Harbour Isle Way intersection in Longwood, Florida. The subsoil conditions were explored with numerous Cone Penetrometer Tests and permanent piezometers. The study indicated that the settlement was caused by internal soil erosion and ravelling. Piezometric elevation contour maps revealed a well defined depression which coincided with the observed surface settlement. The piezometer monitoring results suggest this may be an effective technique for economically identifying potential sinkhole locations.

INTRODUCTION

In October 1989, Florida Department of Transportation (FDOT) maintenance engineers noticed some minor pavement settlement at the intersection of State Road 434 and Harbour Isle Way in Longwood, Florida. This intersection is located in Seminole County, about 8 miles north of Orlando and 1½ miles east of Interstate 4. Figure 1 shows the general site location.

FIG. 1. General Site Location

About this same time, five small sinkholes occurred North of State Road 434 and two developed to the South. Most of these sinkholes began as shallow surface depressions which gradually deepened. They eventually caused building settlements and damaged several streets, yards and driveways. Figure 2 shows the approximate locations of these seven sinkholes as well as another one which damaged a Shopping Center parking lot in August 1992. Because of the intense nearby sinkhole activity, it was decided that close monitoring and subsurface explorations should be performed to determine the cause of the State Road 434 pavement settlement.

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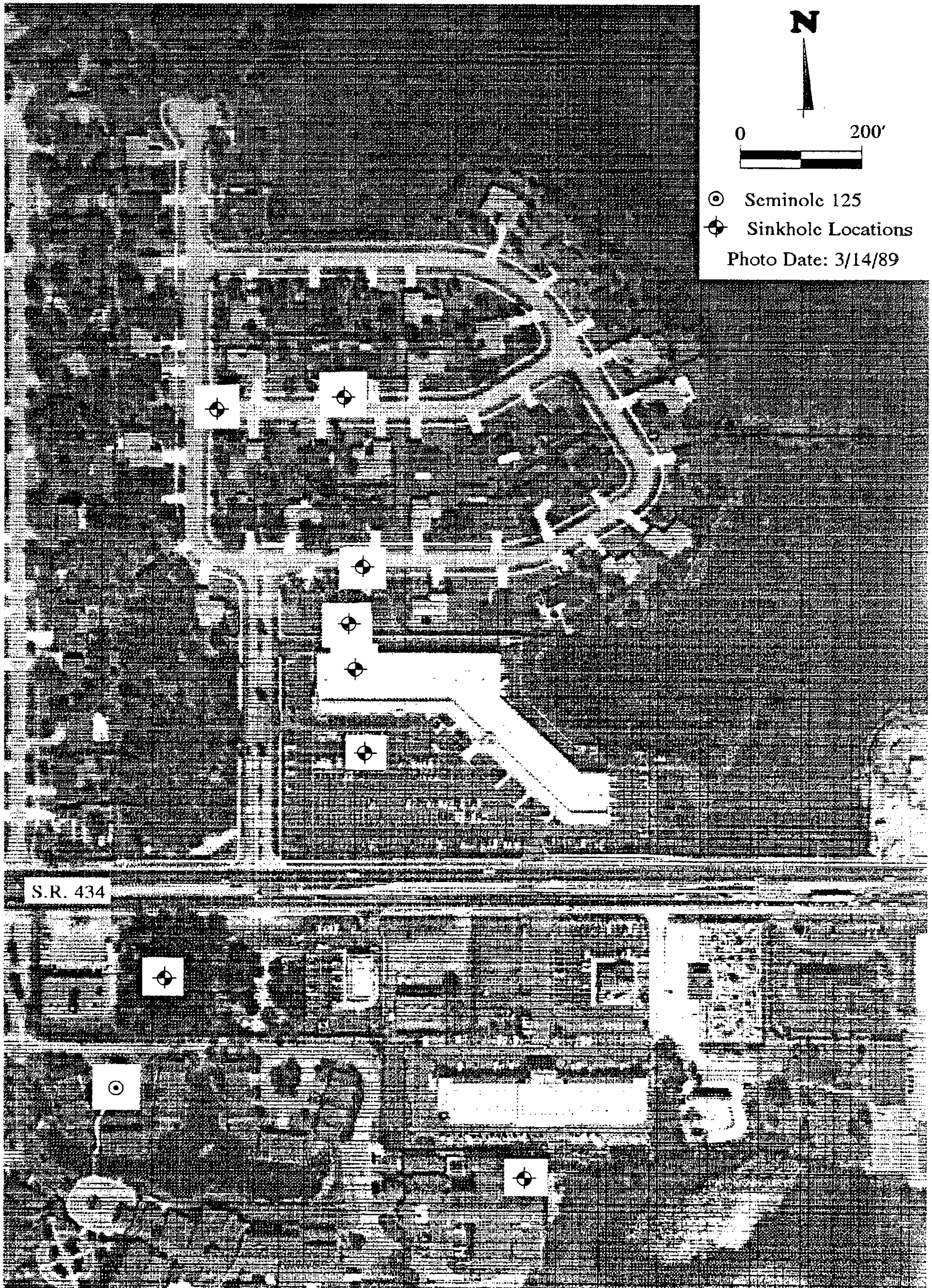


FIG. 2. Vicinity Sinkhole Activity

SITE DESCRIPTION

This site is in the Orlando Ridge physiographic region of Central Florida. Figure 3 shows the existing area topography and urban development. In general, ground surface elevations vary from about +87 to +90 feet (NGVD). Local surface drainage is predominately towards the lowlying areas to the northeast and south. According to the USDA Soil Conservation Service(SCS), the near surface soils consist of moderately well drained Tavares-Millhopper fine sands.

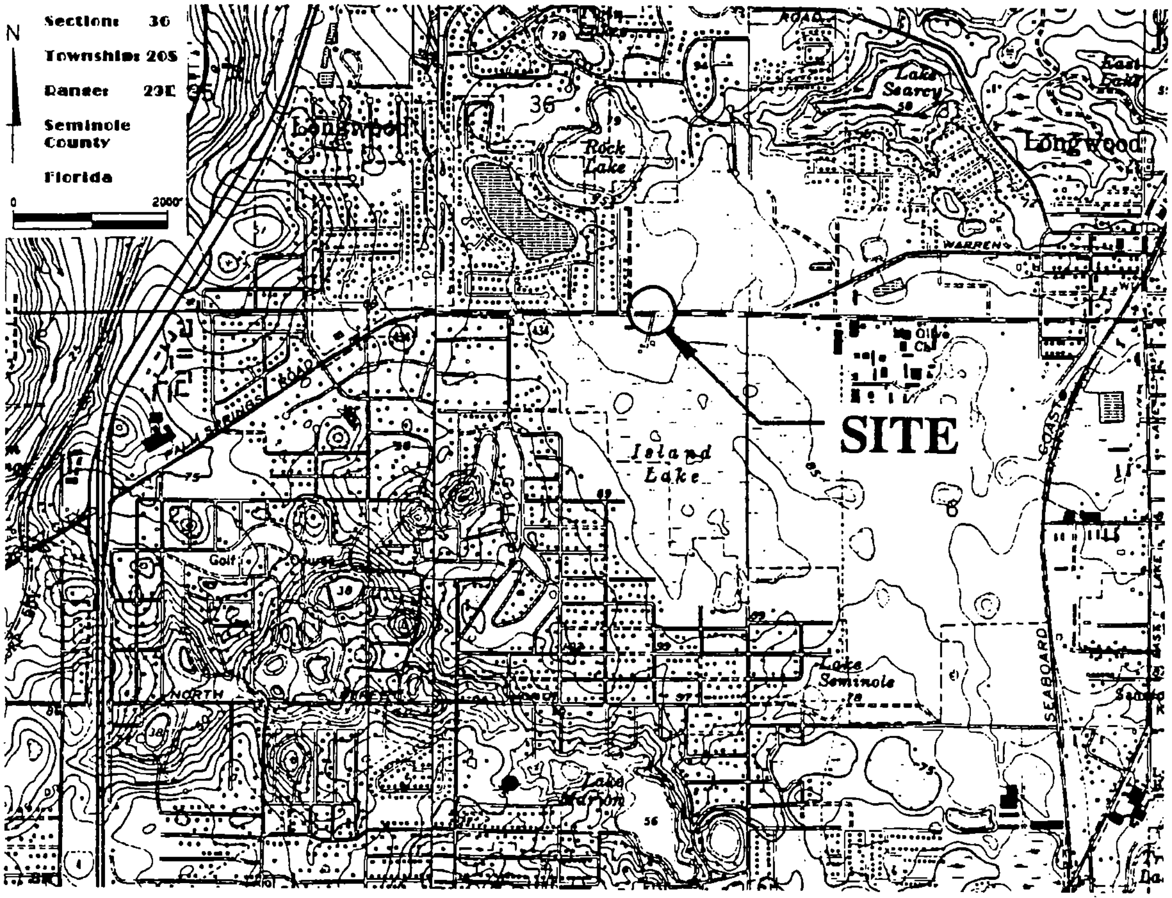


FIG. 3. Area topography (ref: USGS Casselberry & Forrest City, Fla. 7.5 minute series quadrangle maps, 1962 - photorevised 1980)

GEOLOGIC SETTING

In this part of Seminole County, the upper layer of the geologic profile typically consists of about 25 to 50 feet of Pleistocene and Recent sandy soil deposits. These deposits are underlain to depths of approximately 100 to 150 feet by the Miocene Age Hawthorn Formation which consists of alternating beds of clays and sandy phosphatic limestone. The Hawthorn Formation is underlain to depths of about 700 feet by the porous Ocala Group and Avon Park Limestone Formations.

According to Barraclough(1962), the Ocala Group and Avon Park Formations are the most productive sources of fresh water in Seminole County. Together with the permeable portions of the Hawthorn Formation, they form the upper part of the artesian Floridan Aquifer. Figure 4 is a graph of aquifer piezometric levels recorded in a well (i.e. Seminole 125) located about 250 feet west and 350 feet south of this intersection (see FIG. 2). This well is 146 feet deep and is cased to 63 feet.

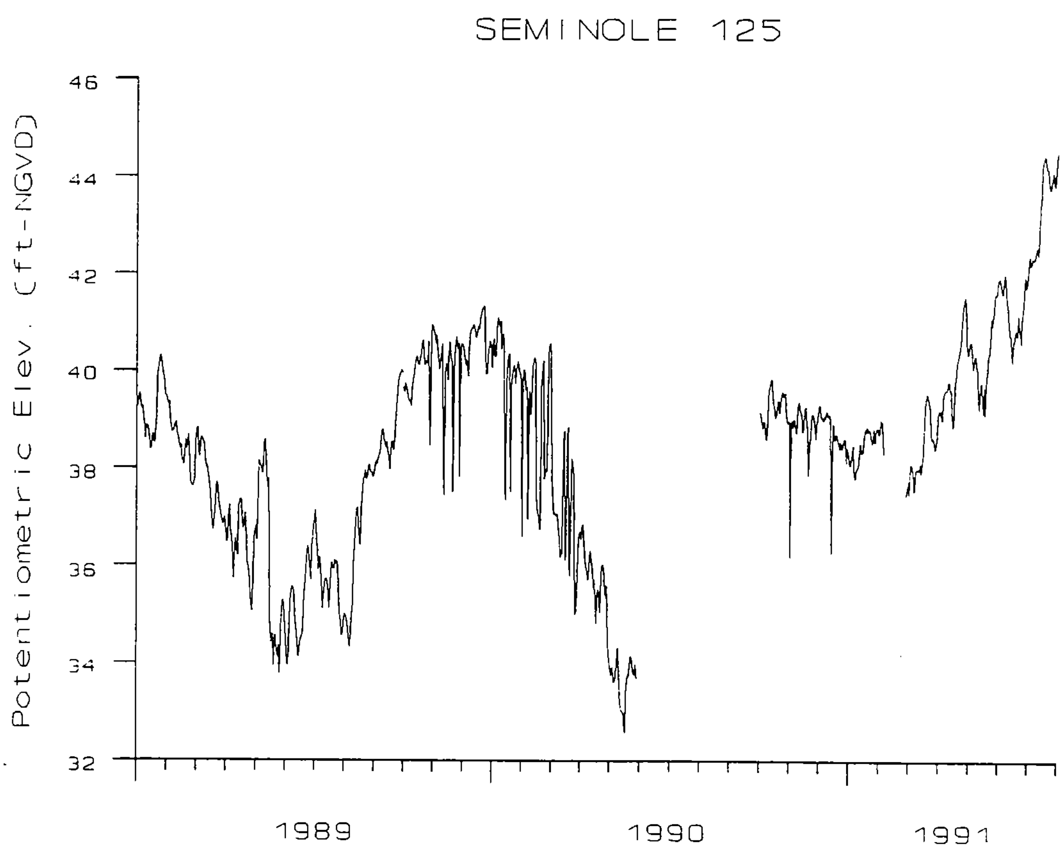


FIG. 4. Seminole 125 piezometric levels recorded by the U.S. Geological Survey.

Clay beds of the Hawthorn Formation generally confine the Floridan Aquifer and seperate it from the overlying non-artesian shallow groundwater table. The aquifer is recharged by downward leakage of groundwater through breaches in the confining clay beds. Stewart(1980) classified this portion of Seminole County as an area of high recharge(i.e. 10 to 20 inches/year).

FIELD EXPLORATION

The field exploration for this study consisted of settlement monitoring, drilling 1 Standard Penetration Test(ASTM D-1586) boring, performing 22 Cone Penetration Tests (ASTM D-3441), installing 20 permanent piezometers, and pore pressure monitoring. A Ground Penetrating Radar survey was also made, but the results were inconclusive.

Based on visual observations and surface elevation surveys, we were able to identify three primary areas where pavement settlement had occurred. These areas are indicated by cross hatching in Figure 5. The greatest settlement (i.e. ≈ 6 inches) indicated by our January 8, 1991 survey was located on the center-line of S.R. 434 near the intersection with Harbour Isle Way. Figure 6 presents the results of this survey.

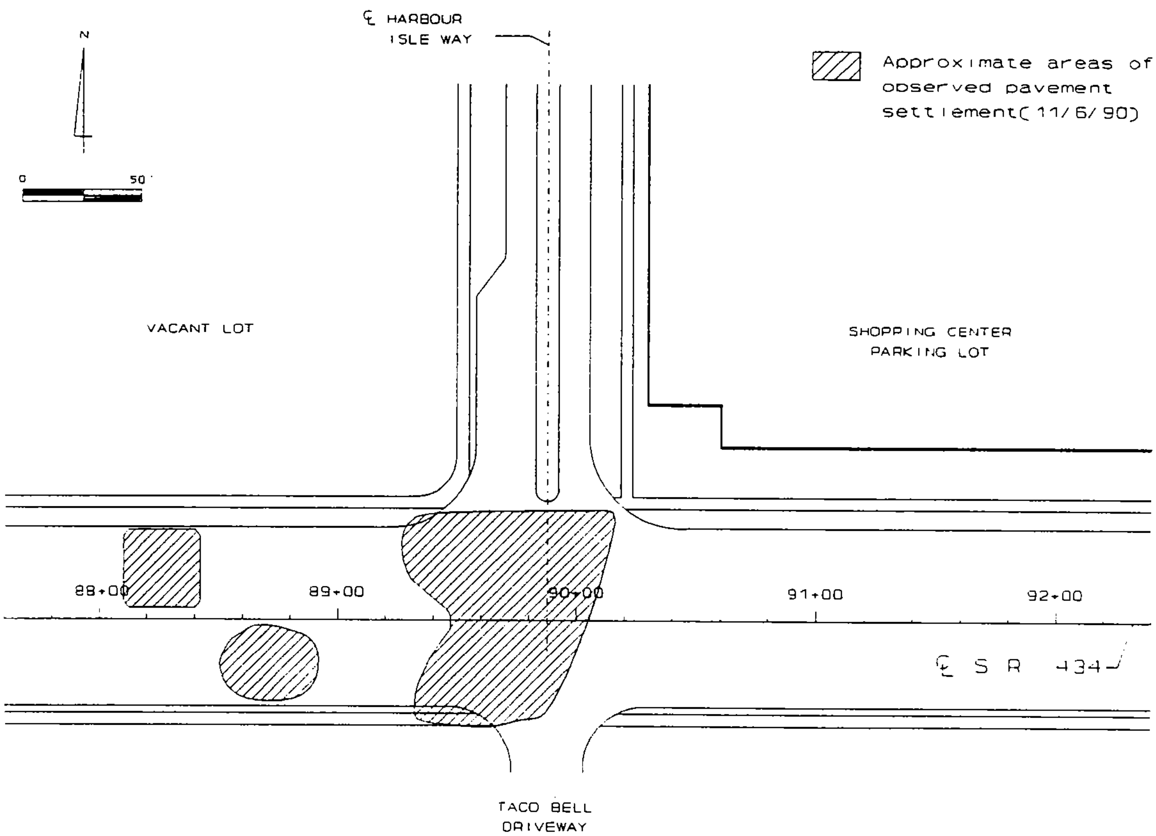


FIG. 5. Pavement settlement observed on November 6, 1990

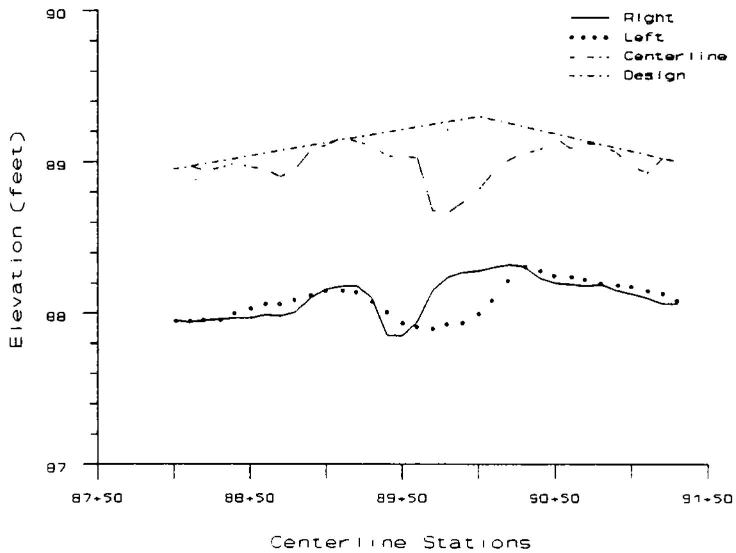


FIG. 6. Surface elevation survey results - January 8, 1991

The locations of the Standard Penetration Test(SPT) boring and the Cone Penetration Test(CPT) soundings are indicated on Figure 7. The results of the SPT boring are presented in Figure 8. The results of the CPT soundings are presented in Figures 9 and 10.

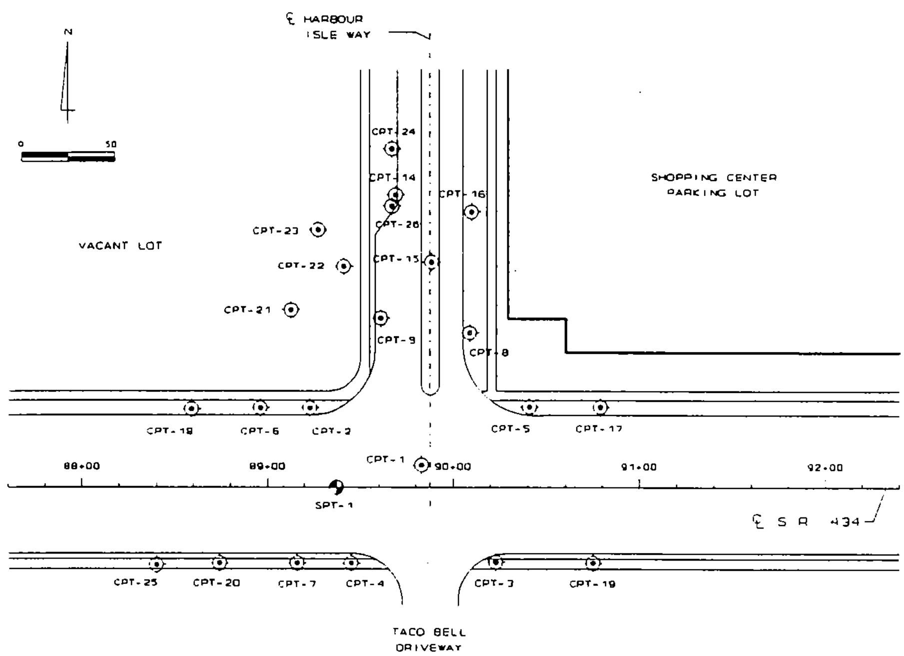


FIG. 7. Boring and sounding locations

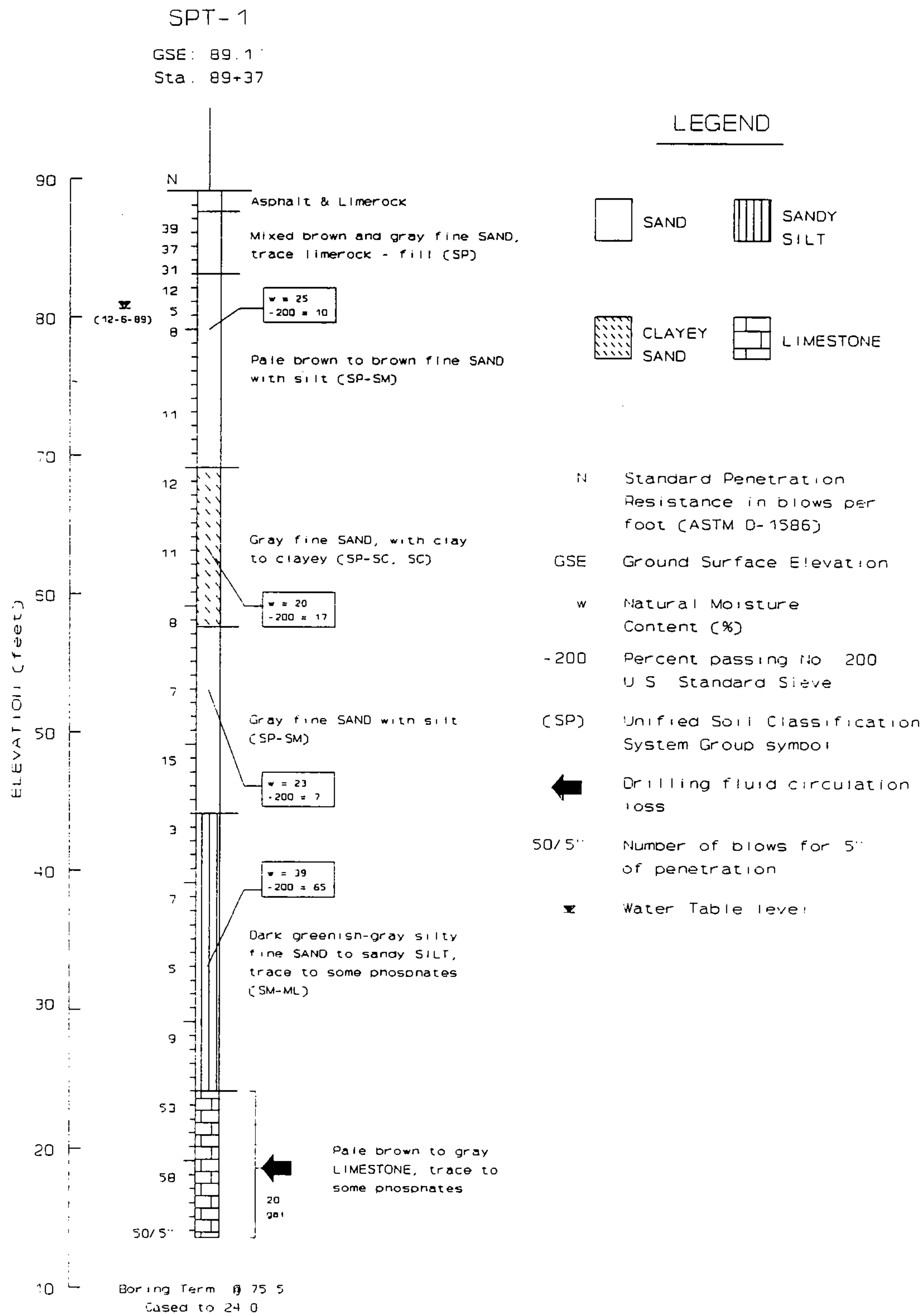


FIG. 8. Standard Penetration Test boring results.

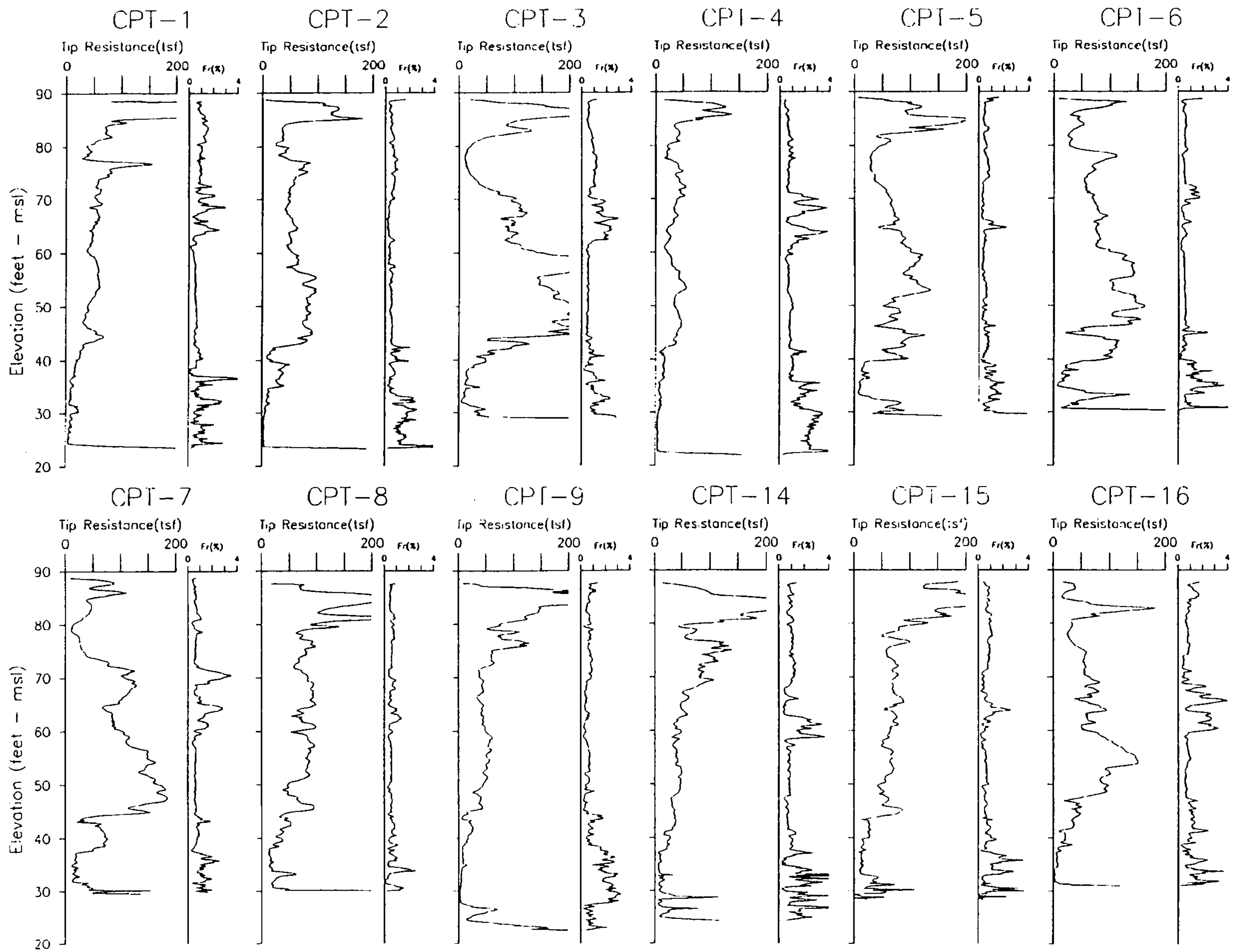


FIG 9 Cone Penetrometer Sounding test results

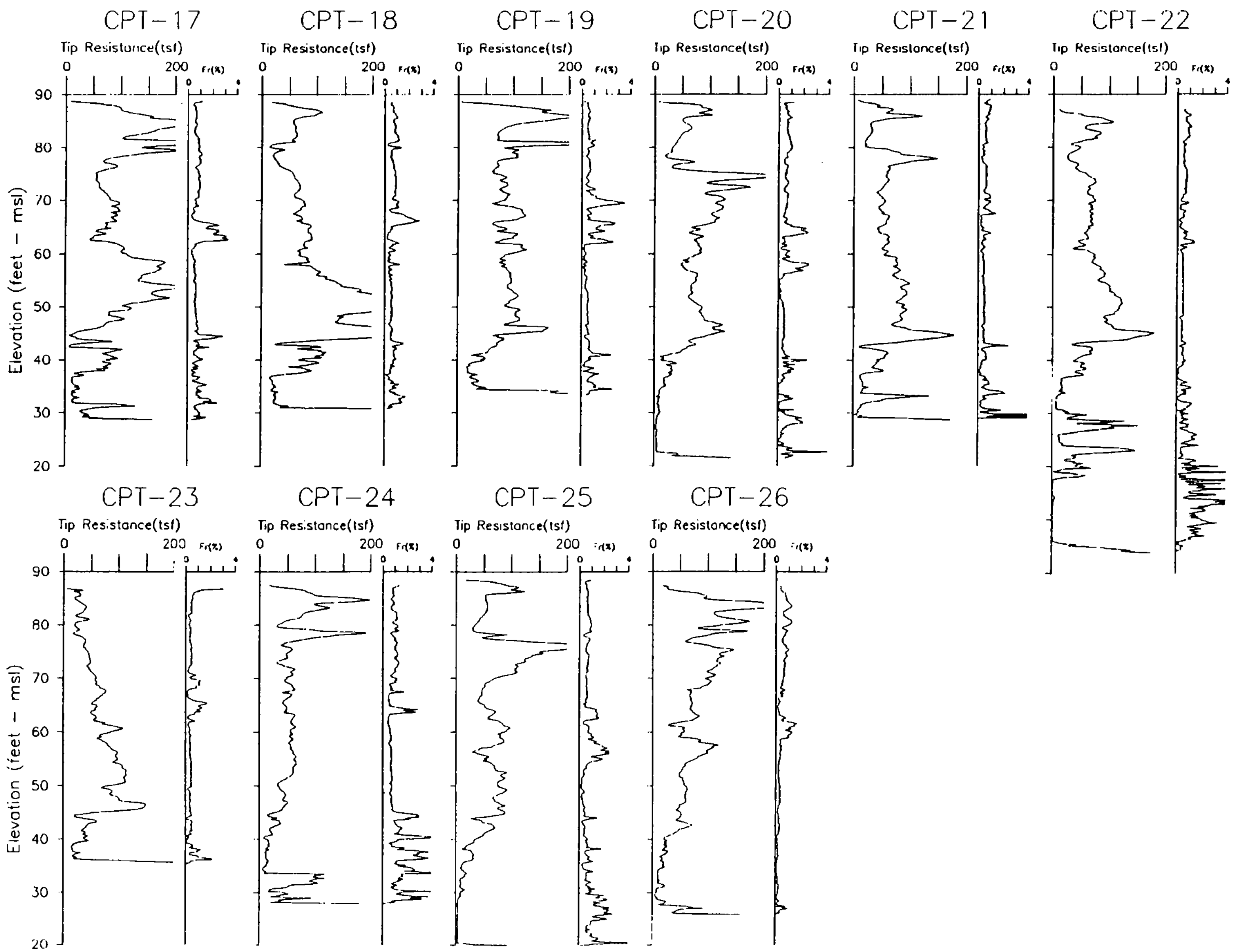


FIG 10 Cone Penetrometer Sounding test results

The piezometers used for this study were PETUR Model P-100 general purpose pneumatic piezometers. Because of the small size of this piezometer(i.e. 0.625 in. O.D. x 2.45 in. long) we were able to install it using our Cone Penetrometer truck. This was done by fabricating a special adapter for the Mechanical Cone rod. The adapter is large enough to hold the P-100 piezometer wrapped in a sand filled filter sock followed by a small sack of bentonite pellets. The adapter was fitted with a disposable steel cone tip to facilitate installation. Cone tip resistance measurements gave a general indication of the soil condition at each piezometer location. Figure 11 is a sketch of the piezometer installation device. The average piezometer installation time for a one man crew was less than 2 hours. The results of pore pressure readings taken through September 1991 are presented in Figure 12. The piezometer tip elevations are indicated in the legend of Figure 12. The piezometer locations are shown on Figure 13.

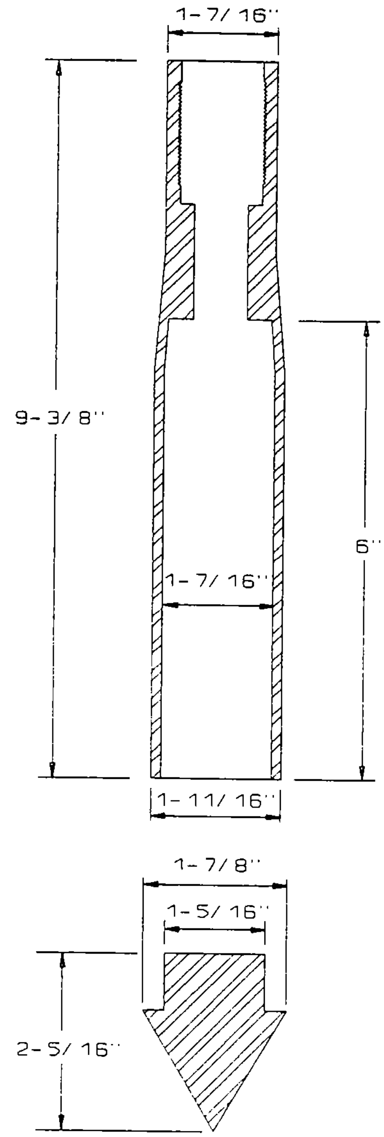


FIG. 11. Piezometer installation device

DATA EVALUATION

The boring and soundings performed for this evaluation did not encounter any near surface compressible soils or buried objects which could be causing the observed pavement settlement. The boring and soundings did however, encounter deep very loose soils which appear to be affected by internal soil erosion.

Internal erosion is a slow process by which the subsoil structure unravels upward due to migration of soil into underlying porous limestone. In an advanced stage, this ravelling will result in a subsidence of the ground commonly referred to as sinkhole activity. Figure 14 presents an illustration of the ravelling sinkhole concept. Sinkholes come in a wide variety of types and sizes. In this area, the surface expression usually consists of either a sudden collapse of the ground(i.e. a cover-collapse sinkhole) or a gradual surface subsidence(i.e. a cover-subsidence sinkhole).

Internal soil erosion is caused by downward flow of groundwater into limestone formations (i.e. recharge). Recharge occurs when there is a hydraulic connection through the confining Hawthorn Formation and a difference in piezometric level exists between the non-artesian shallow groundwater table and the limestone aquifer. As indicated on Figure 4, aquifer piezometric levels measured in well "Seminole 125" between January 1, 1989 and September 30, 1991 varied between elevations +32.57 and +44.55 feet(NGVD). On December 6, 1989 boring SPT-1 encountered the non-artesian shallow groundwater table at elevation +80.6 feet(NGVD).

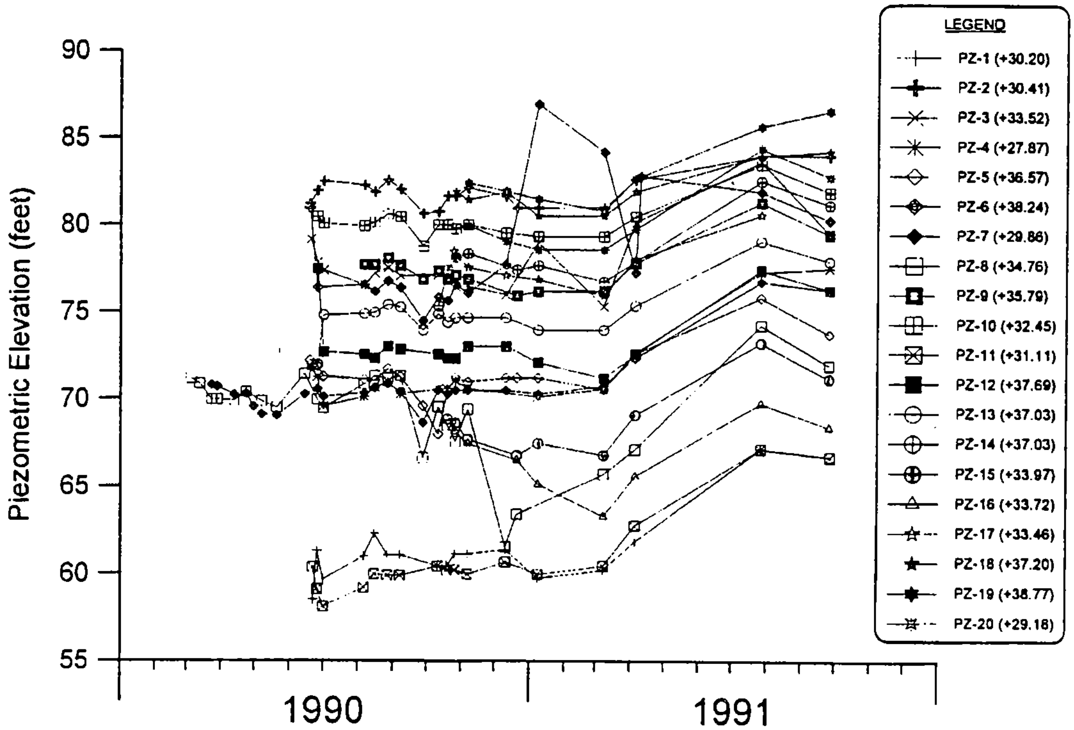


FIG. 12. Piezometer pore pressure readings

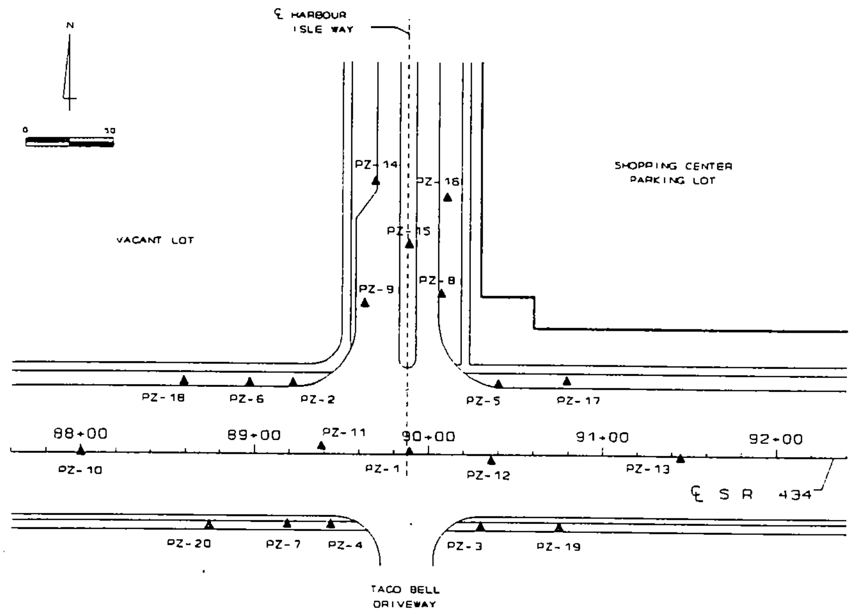


FIG. 13. Piezometer locations

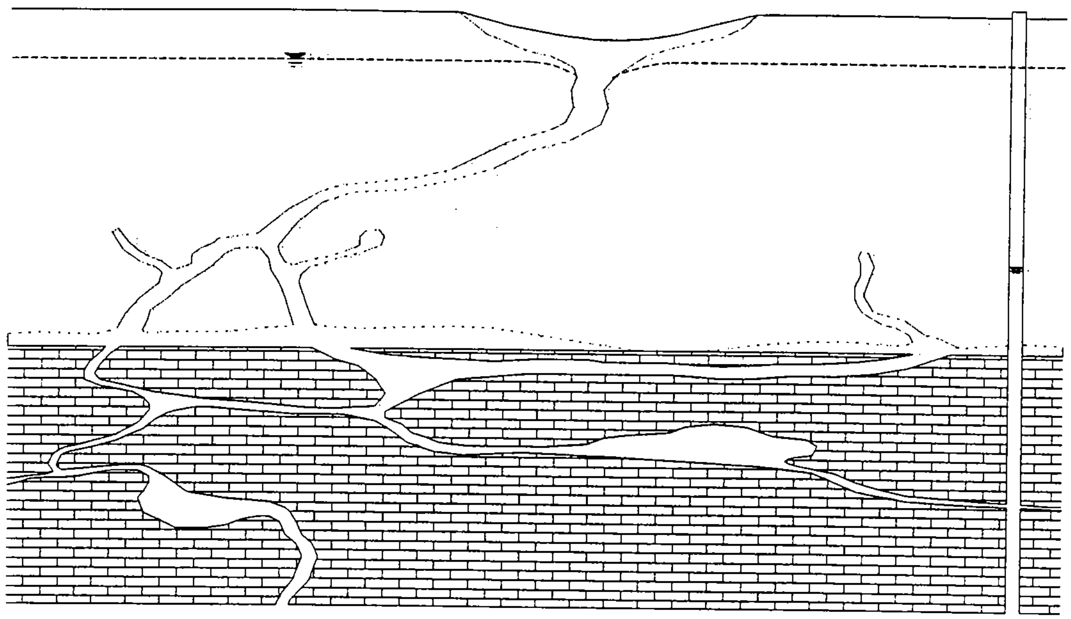


FIG. 14. Ravelling sinkhole conceptual illustration

Another indicator of ongoing recharge in this area is the piezometer readings presented previously. If piezometric elevation head readings for a single time are plotted on a site map and contoured, it is possible to determine subsurface groundwater flow patterns. The flow will be from high piezometric head to low piezometric head. Figure 15 presents this type of contour map based on piezometric readings recorded on November 6, 1990. This map indicates that there is a well defined piezometric head depression in the vicinity of the worst pavement settlement. A similar pattern exists when plots are done for other dates.

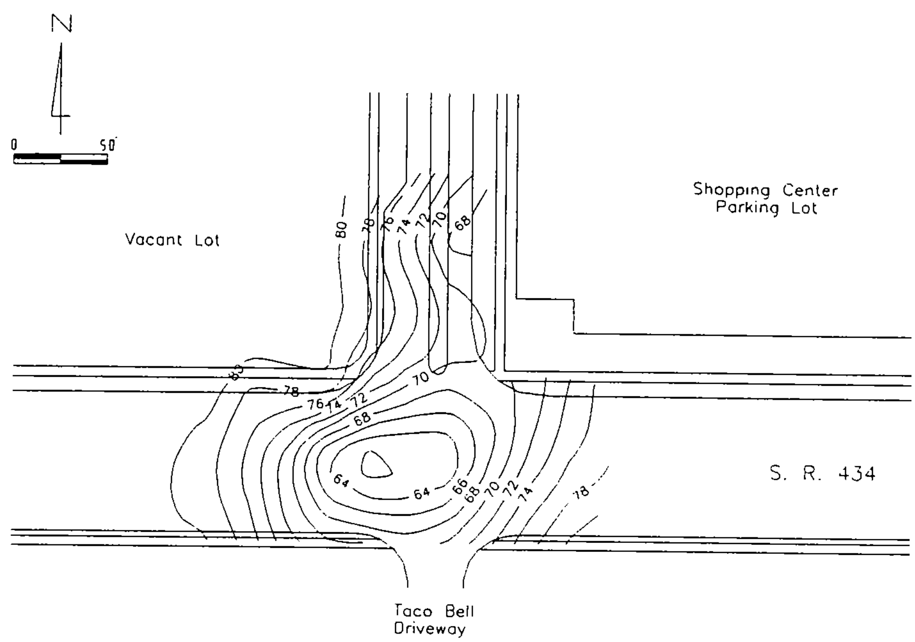


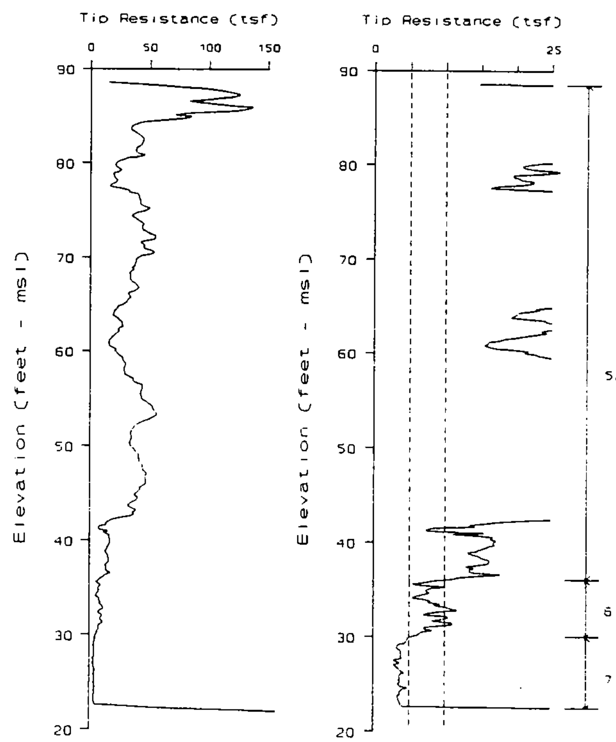
FIG. 15. Piezometric head contours based on November 6, 1990 readings.

The CPT soundings indicate that the ravelled subsoil conditions in this area are primarily confined to a 75 to 100 foot wide strip which runs essentially north and south through the S.R. 434/Harbour Isle Way intersection. The ravelling is not dramatic, but it can be determined by closely inspecting the cone results. The ravelled soils can be identified by their abnormally low cone tip resistances(i.e. $q_c \leq 10$ tsf). Since these are predominately sandy soils which were deposited in a prehistoric marine environment, they should have tip resistances greater than 10 tsf. The most reasonable explanation for their abnormally low shear strength is that they have been disturbed and loosened by downward erosion into limestone solution channels/cavities. One method of determining the degree of internal erosion is by calculating the Ravelling Index(RI). The Ravelling Index is an empirical ratio of ravelled zone thickness to depth to top of the ravelled zone. The higher the index, the worse the condition. Figure 16 presents the Ravelling Index calculation for CPT-4.

Figure 17 indicates the Ravelling Index calculated for each Cone Penetrometer Location. As indicated in Figure 18, the ravelled areas appear to coincide with a trough shaped depression of the limerock surface.

CONCLUSIONS

The results of this study indicate that the observed pavement settlement is being caused by cover-subsidence type sinkhole activity. The encountered soils are not severely ravelled. However, the deeper strata are moderately disturbed and they are probably being eroded into underlying limestone solution channels and cavities. This slow deterioration of the subsoil profile appears to be the cause of the continuing pavement settlement problems. If this area is not stabilized, localized pavement settlement similar to that which has already been documented, will probably continue.



$$\text{Ravelling Index(RI)} = \frac{\text{Thickness of Ravelled Zone}}{\text{Depth to top of Ravelled Zone}}$$

RI = (6')(0.75) + 7 = 11.5/52 = 0.22

FIG. 16. Ravelling Index (RI) calculation for CPT-4.

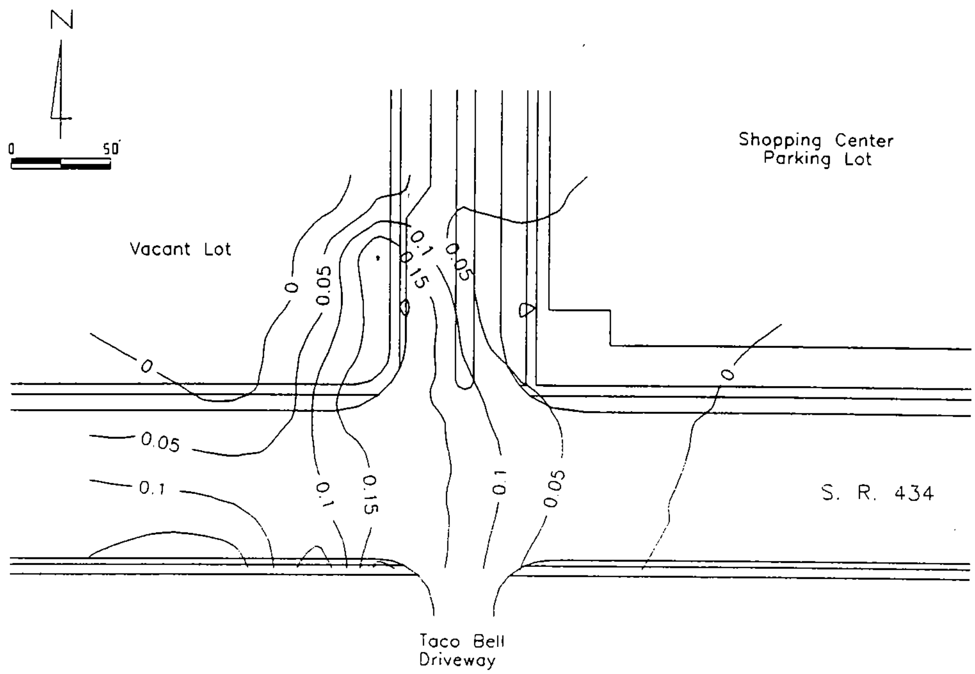


FIG. 17. Ravelling Index (RI) contours based on CPT results.

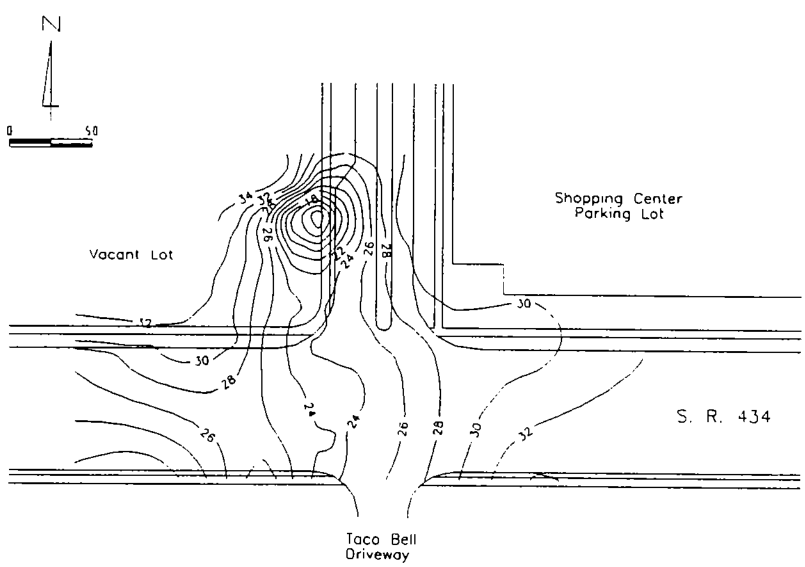


FIG. 18. "Rock" surface elevation contours based on CPT results.

One of the most valuable results of this study was the success of the permanent piezometers. The piezometers appear to be a very useful tool for economically identifying potential sinkhole locations. By using the modified Cone Penetrometer equipment, piezometers can be quickly and efficiently installed. In most soil conditions and depending on site accessibility, a two man crew should be able to install 4 or 5 piezometers to depths of 60 feet in less than one day. This should result in a total cost(i.e. piezometer + installation) of less than $300/piezometer. At this low cost, it should be economically feasible to install a closely spaced grid of piezometers for most proposed building areas. In this manner, potentially active sinkhole areas can be identified and explored thoroughly with Cone Penetration Tests and Standard Penetration Test Borings.

Use of the pore pressure contouring technique for identifying potential sinkhole locations needs to be studied further to confirm its reliability in a variety of subsoil conditions. The technique appears to have worked well at this site where the soils are predominately fine sands and silty/clayey fine sands. It may not perform as well at sites which have different subsoil conditions such as a predominately clayey profile.

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APPENDIX I. REFERENCES

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TECHNICAL RELATED ANALYSIS, DESIGN AND CONSTRUCTION 4 LANE HIGHWAY OVER 8 TO 20 FEET OF PEAT

by

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ABSTRACT

Prior to the development of several thousand acres of residential communities during the late 1980's in the City of Oviedo, Mitchell Hammock Road West consisted of a 1 to 2-lane unimproved roadway. The most westerly portion was comprised of a filled road through a muck farm. As part of the agreements for the adjacent land developments, Mitchell Hammock Road West was to be improved to a 4-lane highway.

During the geotechnical field exploration by Michael D. Sims & Associates, Inc. surficial organic soils varying from 4 to more than 20 feet in thickness were encountered beneath approximately 1/2 mile of the roadway alignment. Furthermore, the subsurface conditions within the initial alignment were highly variable due to the placement of the farm road which included internal ditches to facilitate the farming operations. Based on careful consideration of the highly variable subsurface soil conditions together with the need for providing an economical alternative for construction of the roadway, the engineering analyses employed an adjusted roadway alignment together with removal of relatively shallow organic soils and the implementation of surcharge program over the deep deposits of organic material.

Development of the surcharge program design required and extensive field program to explore possible roadway alignments with less variable subsurface conditions. Approximately 8 lateral ditches existed in the selected right-of-way which were constructed as part of the farming operations. The use of geogrids together with selective backfilling were employed as part of the roadway design over the variable surface conditions (open ditches). In addition, careful consideration was given to the design and construction of the transition areas where the roadway was completely demucked adjacent to the surcharge area. Analyses procedures for structural stability of the pavement section, lateral drainage to assist in the consolidation of the compressible soils below the surcharge, as well as a detailed presentation of the construction procedures and results will be provided.

Introduction

Organic soils present a particular problem for the support of roadways in that the materials have low strength and generally are highly compressible. There are primarily three (3) methods that can be employed in constructing in areas where organic soils exist:

1. Complete removal of the organic soil/peat, followed by backfilling with suitable fill to the final grades.

2. Construction of a Land Bridge consisting of structural slabs supported on a pile foundation system.

3. Building directly on the organics following the implementation of the surcharge program.

Primarily due to cost, complete removal of the organic soils is generally not feasible to depths greater than about 8 feet. The use of a structural slab supported on piles is a viable yet still costly alternative in deep organic soils. In general, the most economical alternative for constructing over deep deposits of organic material is to surcharge the soils above the maximum expected post development load.

Organic soils are highly compressible. This engineering property can be attributed to their high void ratio (ratio of the volume of the voids to the volume of the solids). In Florida, organic matter is almost always found submerged below the groundwater table. Water fills the voids in the material, resulting in a high moisture content. During surcharging, the moisture is expelled from the voids and the soil particles are rearranged to a more compact configuration. The surcharged material is less compressible and has increased strength. Also, it has been found that surcharging to higher effective stress levels acts to reduce long term settlement due to secondary compression (a phenomena similar to creep). Consequently, the main purpose of the preload/surcharge program is to maximize the primary consolidation prior to construction over the organic material.

In summary, a surcharge program entails the construction of an embankment directly on the organic soils to a height generally 5 to 7 feet above the final roadway grade. Settlement plates are set and monitored throughout the filling process as a design tool to control the rate of fill placement. When the surcharge has reached its design height, monitoring of the settlement is continued for a period generally ranging from 3 to 6 months. The results of the time verses settlement curves are used to indicate when the surcharge load may be removed and to predict final long term settlement.

Basically, the design of a surcharge program includes the determination of the thicknesses of the organic soil deposit, evaluation of the surcharge fill height, and monitoring the settlement of the compressible soils during and following the placement of the fill to the design height. Key elements in the design of the Mitchell Hammock Road West roadway over the organic soils which will be discussed herein include:

- Selection of the alignment.
- Preparation of the subgrade soils over existing ditches.
- Evaluation of subsurface drainage to drain excess pore pressures within the organic soils and surcharge embankment fill.
- Total removal of the shallower organic materials followed by replacement with suitable compacted structural fill.
- Design of transition areas between the completely demucked and surcharged areas.
- Recommendations for the careful control and placement of the surcharge embankment.

SUBSOIL CONDITIONS AND ROADWAY ALIGNMENT EVALUATION

Mitchell Hammock Road West is approximately 1.1 miles in length and intersects with State Road 426 and State Road 434 in the City of Oviedo, Florida (Figure 1). The roadway consists of four (4) lanes with two (2) traffic lanes in either direction. A typical cross-section showing the roadway and embankment is shown of Figure 2.

The roadway alignment traversed a large deposit of organic soil known as Mitchell Hammock that has been farmed for over 40 years. Prior to the improvements, a dirt roadway was constructed and used in conjunction with the farming operations. The dirt roadway was in existence some tens of years prior to the planned improvements. The thickness of the fill soils below the dirt road varied from 2-½ to 7 feet below the existing grades. Figure 3 illustrates a portion of the original alignment and depicts the location of the existing dirt road and ditches. Immediately adjacent to the dirt road were ditches and canals which were used to drain the muck fields as part of the farming operations. The north/south lateral ditches occurred across the entire alignment and the collector ditch was situated in an east/west direction along the existing dirt road.

Based upon the results of our investigation, organic soil thicknesses in excess of 15 feet existed below the proposed roadway alignment. The organic soils were in different stages of compression along the length and width of the alignment due to the presence of the fill below the dirt road. In addition, the excavation of the ditches created sudden changes in organic soil thicknesses that were not conducive to an effective surcharge program. Based on of the depth of the organics, the presence of the existing dirt road and ditches, the subsurface soil conditions were considered to be highly variable within the proposed alignment. In view of the highly variable subsurface conditions, the most assured means of adequately supporting the roadway along this original alignment was to completely remove the unsuitable organic soils, which was not considered economically feasible.

The review and evaluation for an alternative alignment included further investigation of the subsurface soil conditions. The additional field investigation performed included a series of muck probes in order to determine the approximate vertical extent of the organic soil present. In many instances, both due to fill and desiccation of the upper organic soil layer, bucket augering was used in the upper few feet of the profile followed by probing using a ½-inch diameter pipe and the weight of two men. The thickness of the organic soils were found to vary from 0 to 21 feet. More generally however, depths of the organic soils were in the range of 8 to 12 feet in thickness. The organic soils were encountered for a total distance of approximately 3,200 linear feet of the alternative roadway alignment.

In addition to the further field investigation, careful consideration was given to the presence of lateral ditches upon which the roadway would have to be constructed. It was also necessary to consider the stability of the soft organic soils along the collector ditch. The selected alternative alignment was located further to the south. A portion of the alternative and selected alignment with respect to the existing collector ditch, farm road, and lateral ditches is depicted on Figure 4.

SURCHARGE DESIGN

The design of the surcharge included: evaluating the surcharge height; evaluation for subsurface drainage within the filled embankment; design of backfill operations over the existing lateral ditches to minimize differential settlement; and the construction of transitional areas between the surcharged and completely demucked areas.

The height of material to be removed after the surcharge hold period was to be a minimum of 2.0 feet. Based upon the required fill removal height described above, settlement estimates, and our experience with surcharging organic soils, a surcharge height of 5 feet above the final grades was selected. The exterior side slopes of the surcharge were three (3) horizontal to one (1) vertical. In addition, the toe of slope was maintained a minimum distance of 20 feet from the collector ditch located along the north edge of the construction limits based upon stability considerations.

Several settlement related factors were evaluated for the design over the existing lateral ditches. In order to provide more uniform loading, the existing trenches/ditches were backfilled with peat soils. In addition to uniform load, the peat soils were to provide a compressibility more consistent to the in-situ peat soils. To further promote uniform settlement of the backfill peat soil and adjacent in-situ organic soils, geogrid reinforcement was placed across the ditches. A typical cross-section of this construction procedure is included on Figure 5.

In order to improve the rate at which water may be expelled from the underlying organic soils, (improve the rate of consolidation process) a drainage system was installed within the embankment fill. The drainage system was comprised of Drainage Net (Conwed XB-8210 with fabric on both sides) approximately 2.5 in width. The Drainage Net was spaced at 25 feet on center throughout the surcharge area. The placement of the geo-net within the surcharge embankment is shown of Figure 4.

As part of the construction, a portion of the roadway was completely demucked. Complete demucking was performed in areas where the peat soils were found to be 6 feet or less. These areas were located at the east and west ends of the organic soil deposit. Differential settlement between the completely demucked and surcharged area was minimized by creating a transition zone. The transition zone consisted of tapering the organic soils at 3 horizontal to 1 vertical beginning at the edge of the organic soils to remain and sloping upward toward the surcharge area. A typical cross-section through the transition area is included on Figure 6.

The monitoring of the settlement performance of the surcharge included measuring settlement of the embankment fill. The settlement monitoring program consisted of the installation of thirteen (13) settlement plates (Figure 8) placed at the center and edges of the embankment throughout the surcharge area. The elevations of the monitoring plates were measured at the end of each work day during the placement of the surcharge embankment. Once the filled area had reached its design height, the settlement was measured three times a week. A sample of the settlement data is included on Figure 9.

CONSTRUCTION PROCEDURES

It is critical to the success of a surcharge program that the geotechnical engineer be directly involved in controlling the surcharging operation (mainly the rate of placement of fill during construction). As part of the preparation of the construction procedures, careful consideration was given to the control of the height and rate of fill placement to prevent stability failures. A summary of the guidelines used for the construction of the Mitchell Hammock Road West surcharge were as follows:

1. Both the east and west ends were completely demucked in areas with 6.0 feet or less of organic soils. The transition areas were constructed as depicted on Figure 6.

2. Prior to initiating the surcharge fill placement, the ditches within the surcharge area were prepared using the method depicted of Figure 5.

3. Upon completion of preparing the lateral ditches, the initial working lift (approximately 2.5 feet in height) was placed over the surcharge area. The initial lift as well as the remaining lifts had to be carefully placed without creating unbalanced loads on the surface and to avoid bearing failures. To the extent practical, the placement of the initial lift was in a uniform manner across the roadway. Furthermore, only light weight operated equipment was allowed within the work area during the original lift placement and the haul roads were varied in order to insure uniform placement of fill.

4. Upon the completion of the first lift and prior to placement of the second lift, the Drainage Net and monitoring plates were installed. The Drainage Net was placed at 25 feet on center as shown on Figure 4. The settlement plates (Figure 8) were placed 18 inches above the organic subgrade soils. At the time that the settlement plates were installed, the thickness of the organic soils were probed. In additi3on, the elevation of the fill was measured.

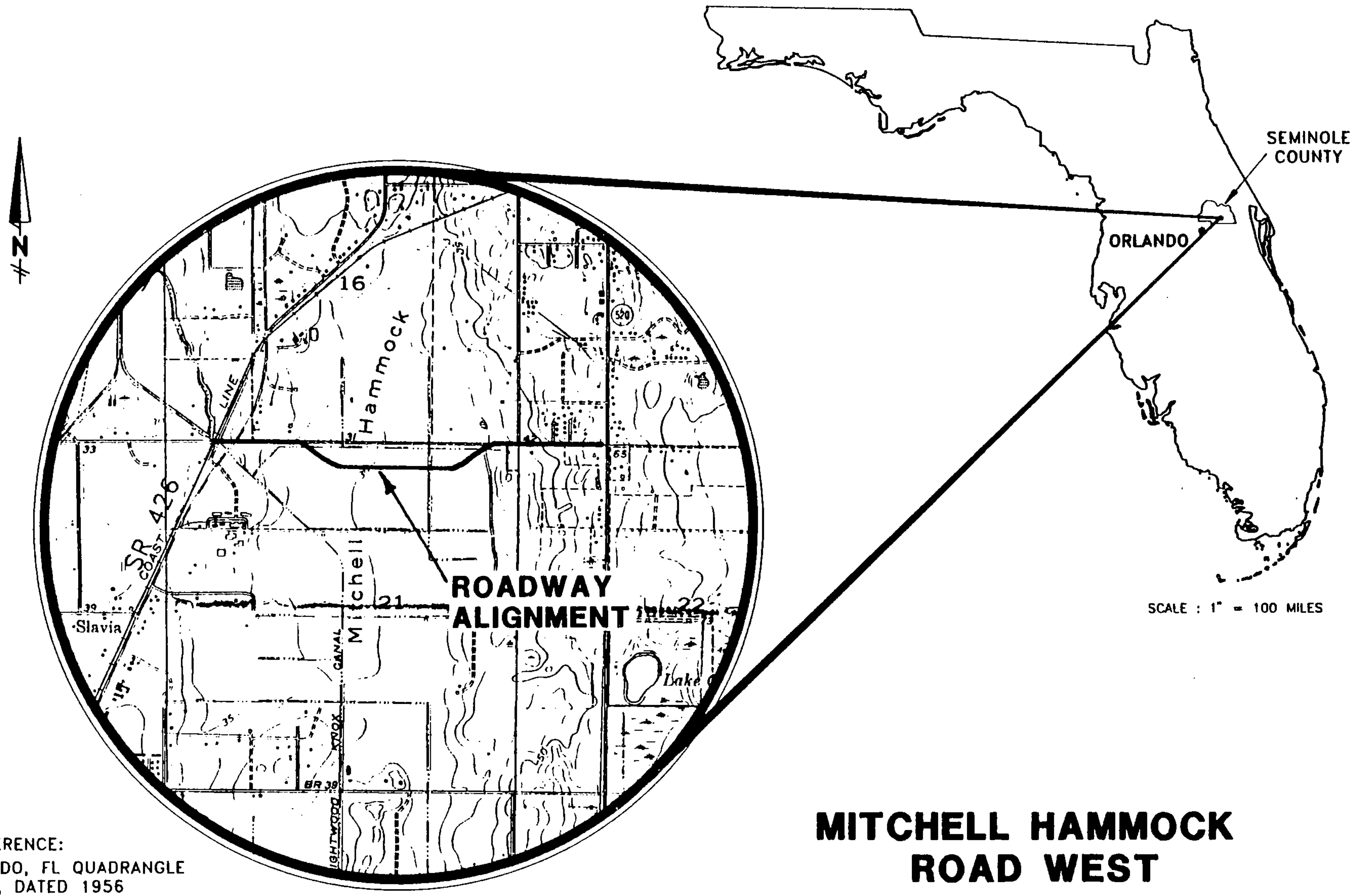
5. The elevations of the settlement plates were read daily while the filling progressed. Upon reaching the designed height, the settlement was monitored on a weekly basis. It is important to note that the maintenance of the settlement plate was the responsibility of the contractor and any settlement pipe knocked down during construction was to be replace immediately to provide continuity of the settlement data.

6. Additional fill required to obtain the design height was limited to 12 inch thick lifts and at the rate determined by the geotechnical engineer based upon settlement performance. The target rate of settlement was values less than 0.02 feet per day (¼ inch per day) before addition fill was allowed. The average hold period needed between lifts was generally on the order of 1 week.

7. The geotechnical engineer was provided settlement data which was later used to determine when the surcharge may be removed. A typical settlement curve is shown on Figure 8. Based upon review of settlement data at all thirteen locations, the required surcharge hold period was approximately 120 days.

Upon completion of the surcharge program, the roadway was constructed using normal construction procedures. The roadway pavement section consisted of 10 a inch compacted subgrade stabilized to a minimum Florida Bearing Value of 75 psi. The subgrade soils were then overlain with geogrid (Tensar SS-1) followed by the placement of the roadway base. The roadway base was comprised of 8 inches of limerock. The wearing surface of the pavement was comprised of 1-¾ inch Type S-1 asphaltic concrete.

The surcharge program and roadway construction were completed in July, 1989 at a significant cost savings when compared to conventional demucking and backfill alternative (savings on the order of 1 million dollars). The roadway has performed satisfactory in all aspects with no signs of settlement or pavement distress. Based upon our experience with both the design and construction of this type of surcharge program operation, we believe that this method provides an economically attractive alternative for roadway construction over compressible peat soils when compared to the conventional construction methods.



REFERENCE:
OVIEDO, FL QUADRANGLE
MAP, DATED 1956
PHOTOREVISED 1980
SEC. 21 & 22
T21S, R31E

VICINITY MAP

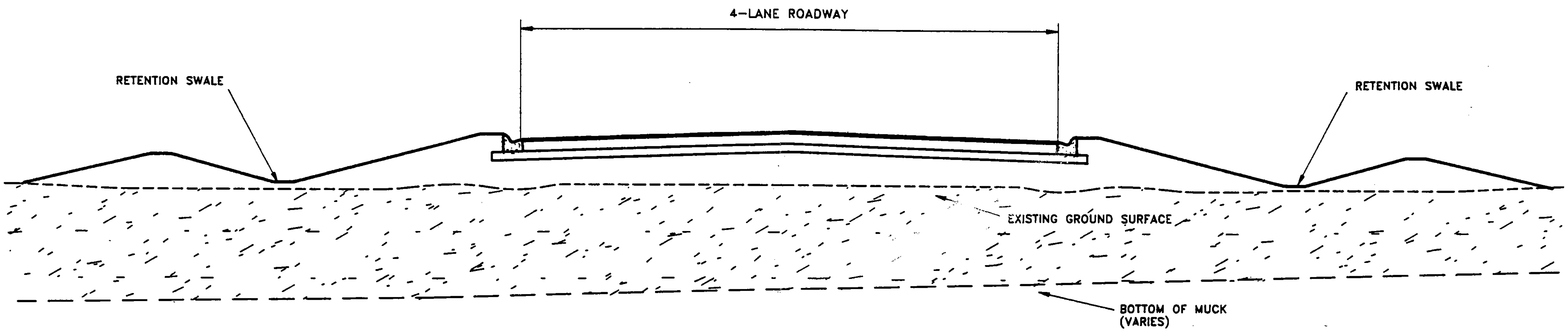
NTS

MITCHELL HAMMOCK ROAD WEST

CITY OF OVIEDO, FLORIDA

FIGURE 1

\DJY\PAP-F1



TYPICAL CROSS-SECTION

NTS

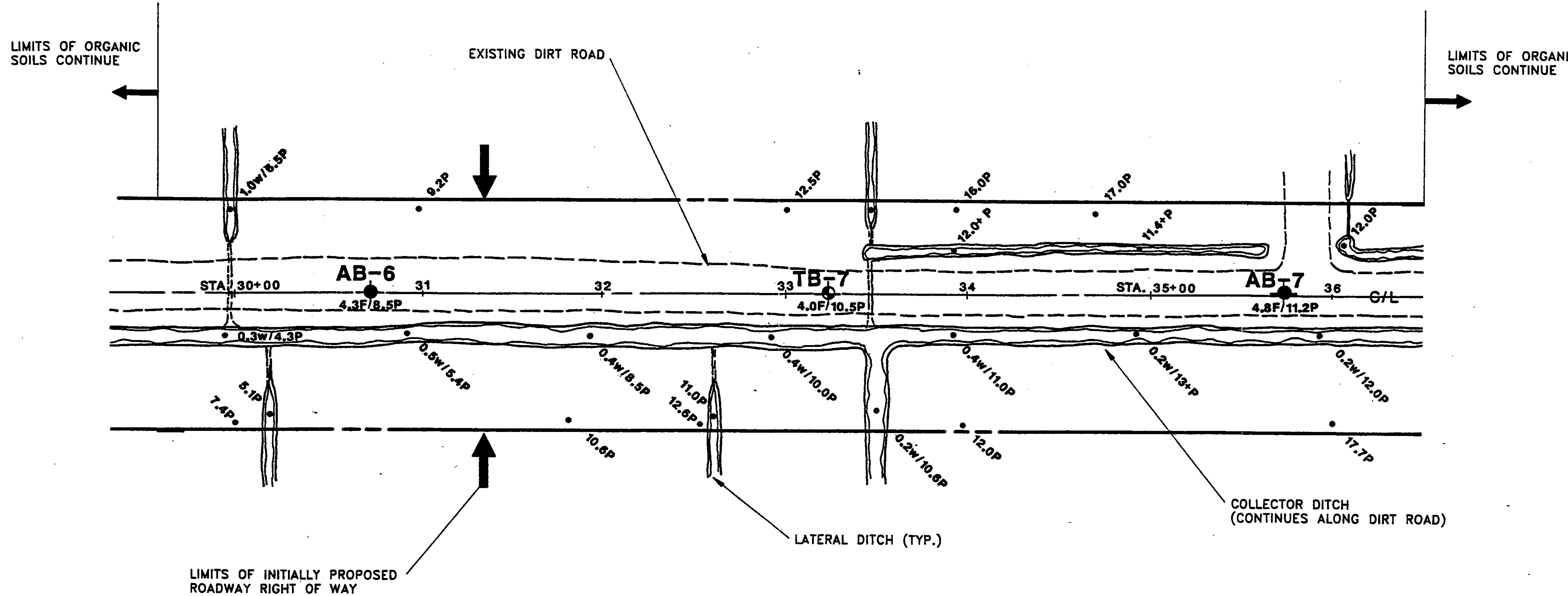
PROPOSED ROADWAY AND SWALES
(STATION 14+00 TO STATION 39+00)

MITCHELL HAMMOCK ROAD WEST

CITY OF OVIEDO, FLORIDA

\D... P-F2

FIGURE 2



PLAN VIEW

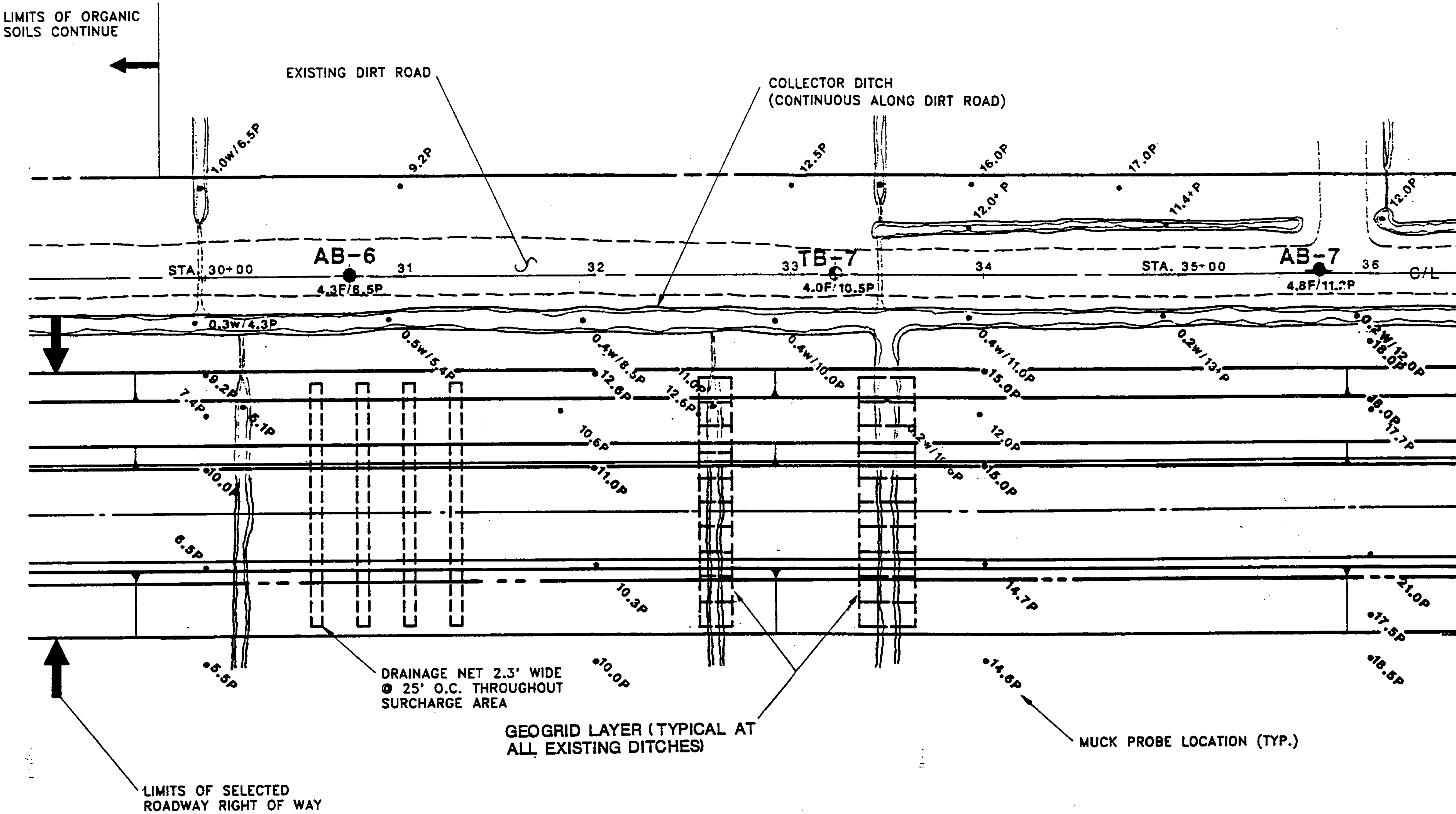
SCALE: 1" = 60'

FIRST ALIGNMENT
(STATION 30+00 TO STATION 36+00)

MITCHELL HAMMOCK ROAD WEST

CITY OF OVIEDO, FLORIDA

\DJY\PAP-F3



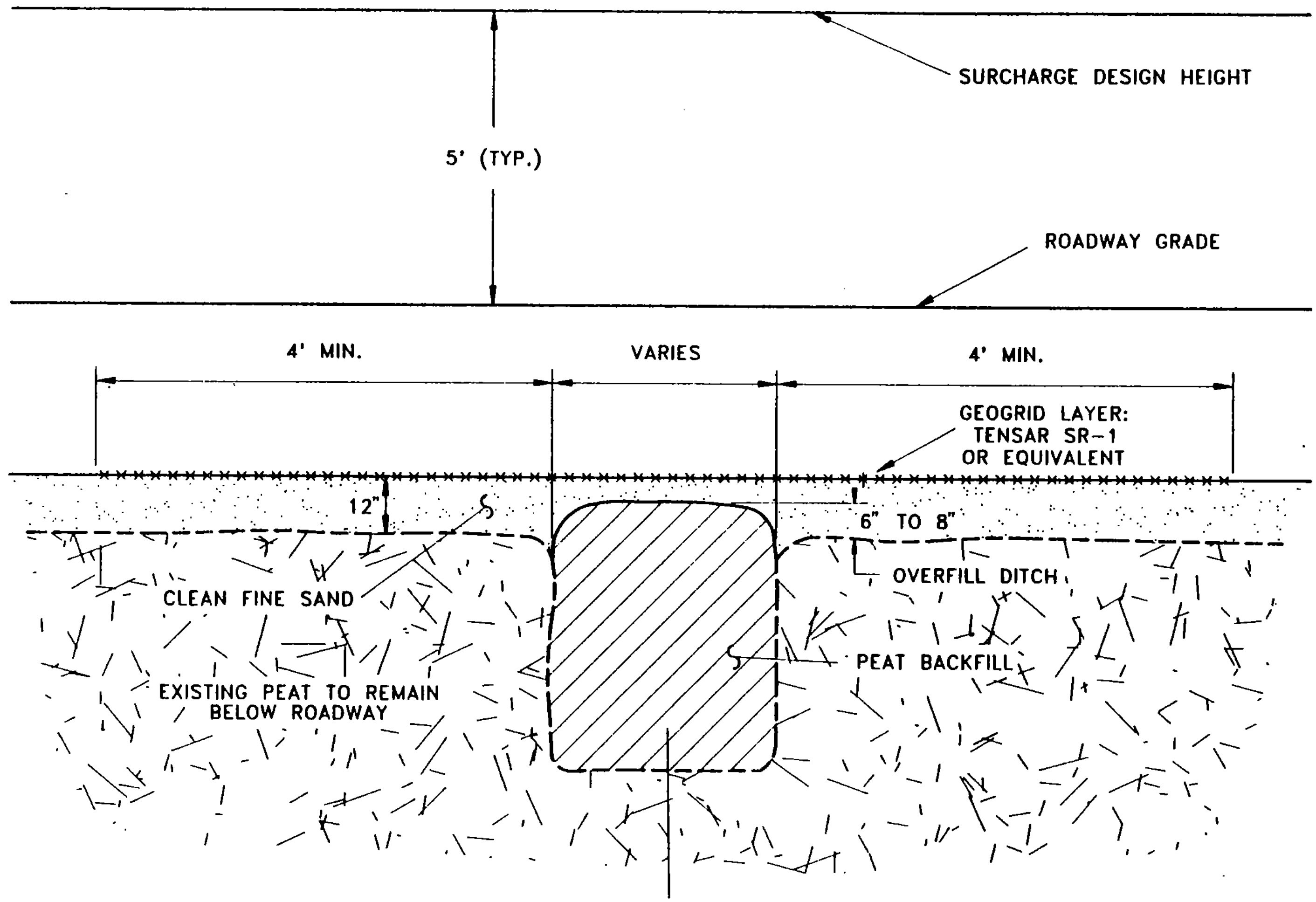
PLAN VIEW

SCALE: 1" = 60'

SELECTED ALIGNMENT
(STATION 30+00 TO STATION 36+00)
MITCHELL HAMMOCK ROAD WEST
CITY OF OVIEDO, FLORIDA

\DJY\PAP-F4

FIGURE 4



TYPICAL CROSS-SECTION

SCALE : 1" = 3'

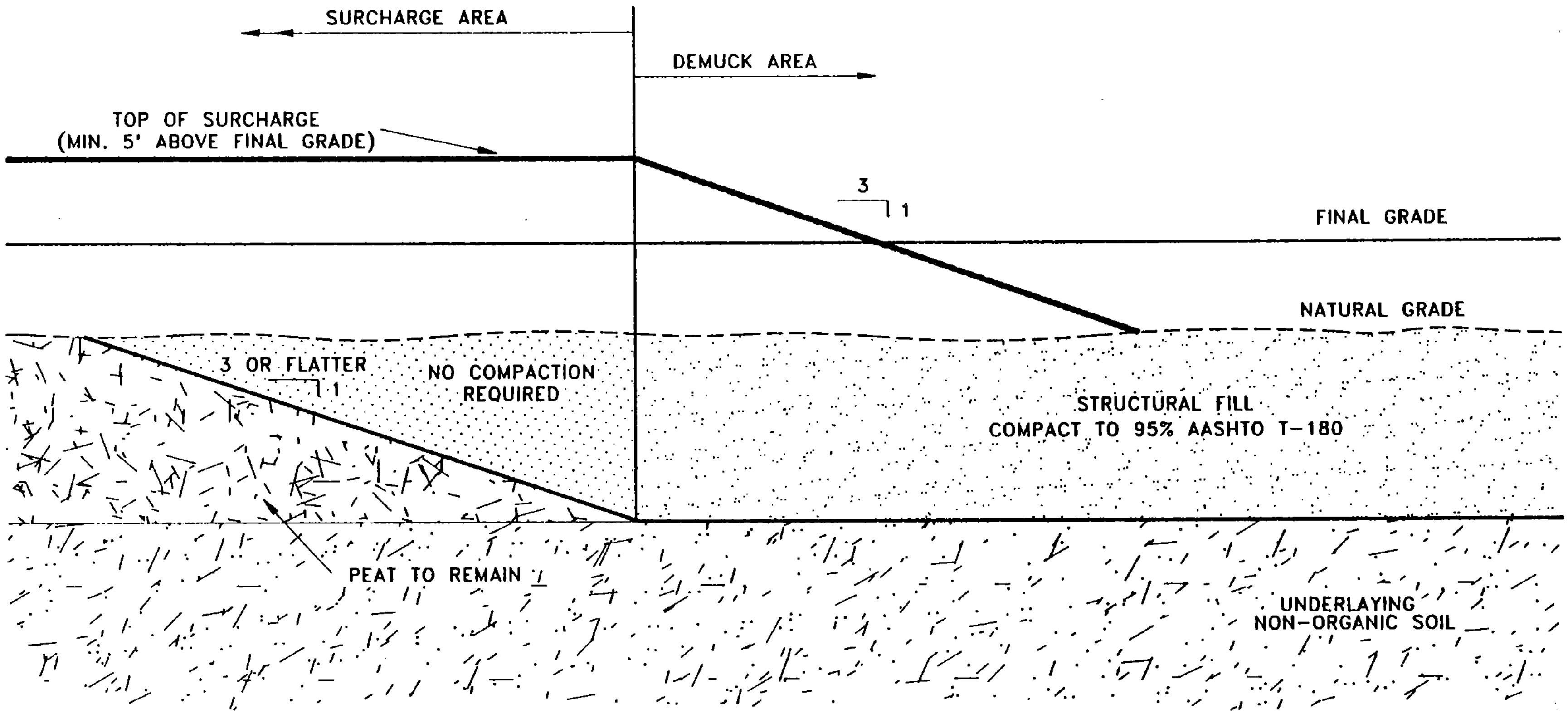
SUBGRADE PREPARATION OVER IRRIGATION DITCHES

MITCHELL HAMMOCK ROAD

CITY OF OVIEDO, FLORIDA

FIGURE 5

\DJY\PAP-F5



TYPICAL CROSS-SECTION

SCALE : 1" = 10 '

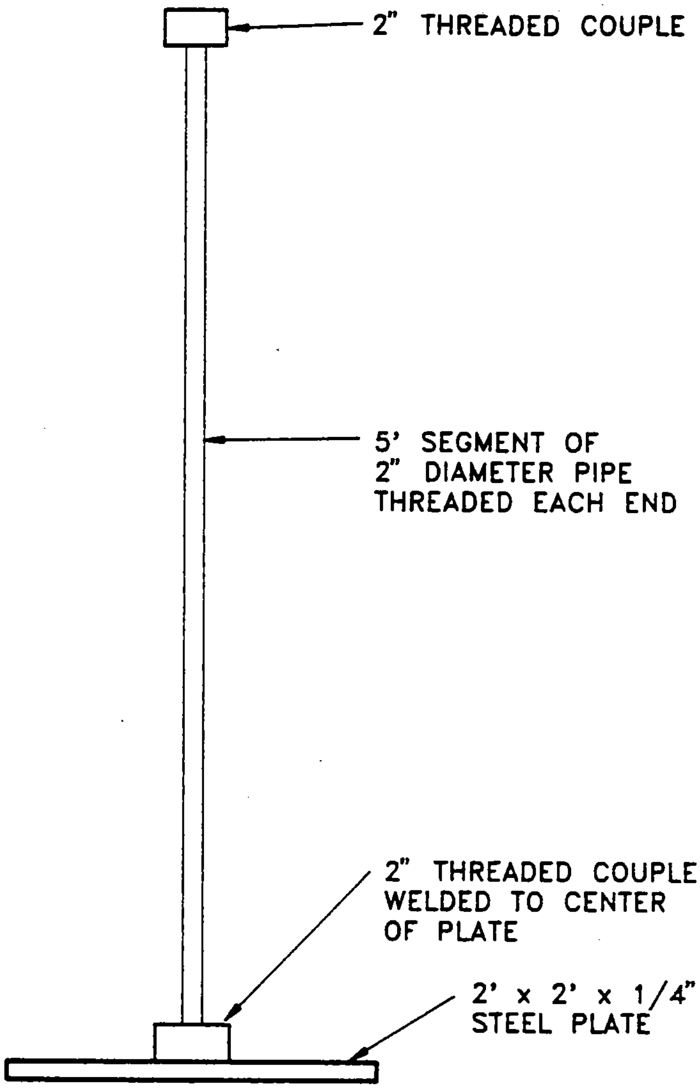
TRANSITION BETWEEN SURCHARGE AND DEMUCKED AREAS

MITCHELL HAMMOCK ROAD

CITY OF OVIEDO, FLORIDA

\DJY\PAP-F6

FIGURE 6

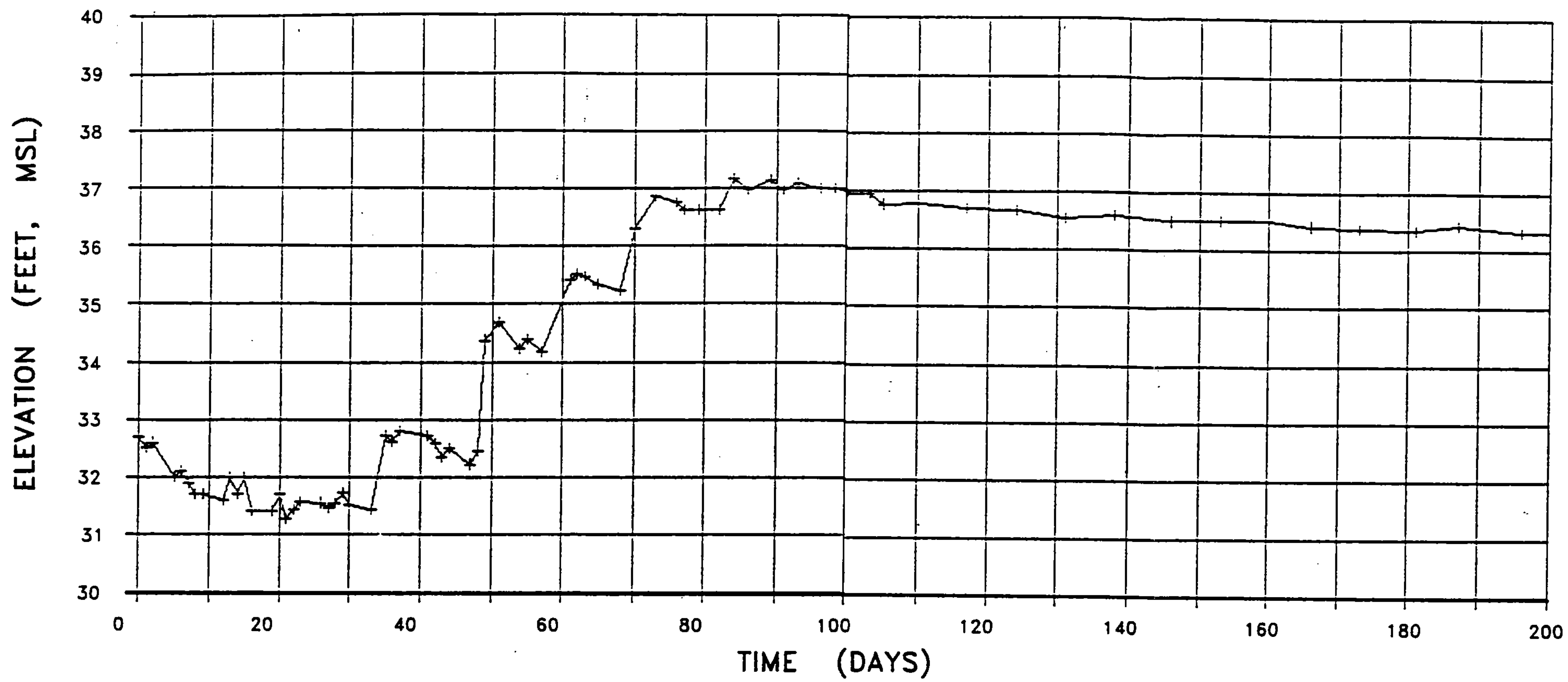


CONSTRUCTION DETAILS - SETTLEMENT MONITORING PLATE

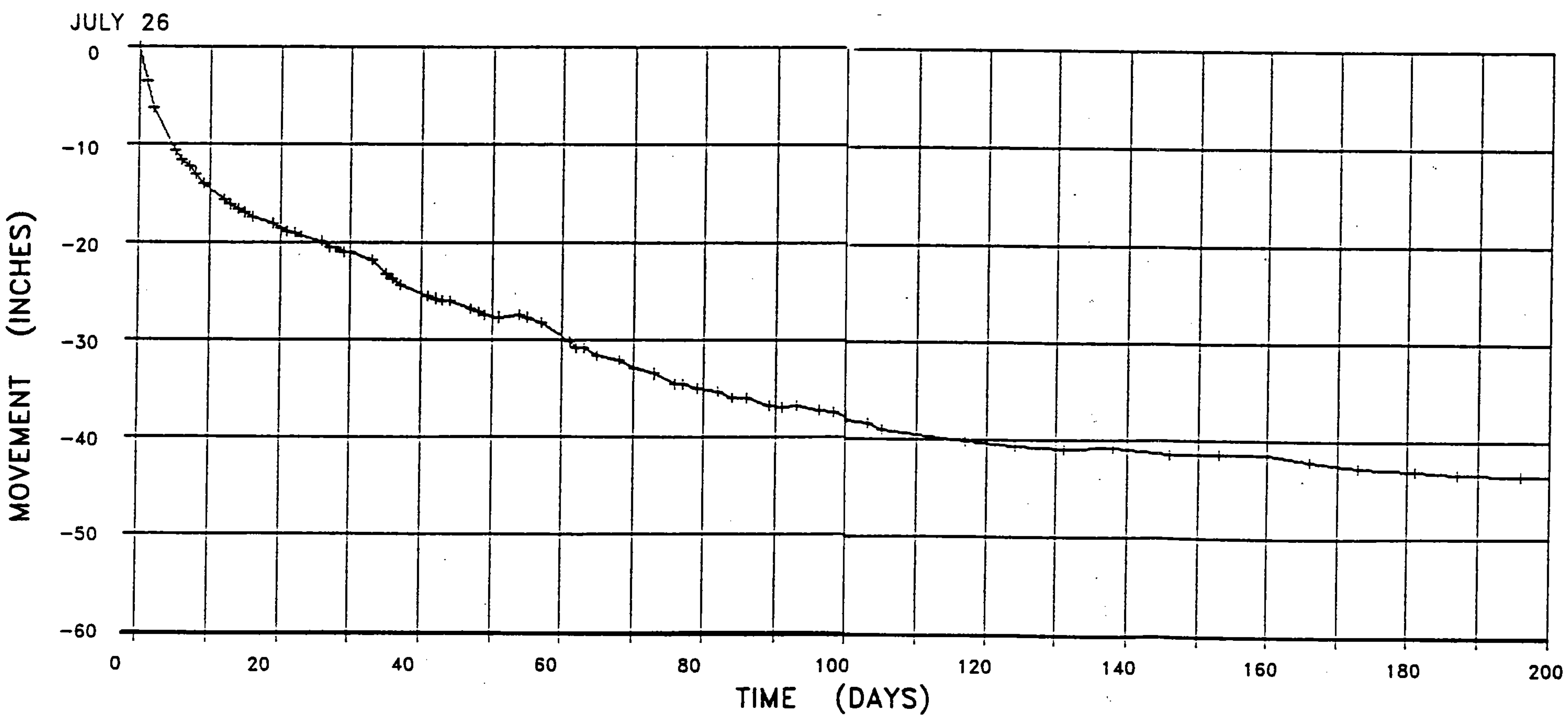
MITCHELL HAMMOCK ROAD

CITY OF OVIEDO, FLORIDA

FIGURE 7



FILL ELEVATION AT SETTLEMENT PLATE MP-5



SETTLEMENT PLATE MP-5 DATA

TYPICAL SETTLEMENT DATA

MITCHELL HAMMOCK ROAD

CITY OF OVIEDO, FLORIDA

\1046\310S1

FIGURE 8

Highway Reconstruction Over an Expansive Subgrade Incorporating A High Strength Geosynthetic Moisture Barrier

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Abstract

With the increased recognition that moisture variations are the basis for most road failures, several new methods have been proposed to solve moisture problems. For the problem of moisture variations in moisture sensitive soils, an innovative solution is the use of a moisture barrier. Moisture barriers can keep moisture out or can be used to maintain a constant moisture content, thereby eliminating the damaging shrinking and swelling of expansive soils associated with changing moisture contents of the soil.

The use of a geosynthetic moisture barrier is illustrated in a case history of the reconstruction of a 5-mile section of Cheyenne Avenue in North Las Vegas, Nevada. Local engineers had previously unsuccessfully tried several stabilization procedures on sections of this roadway, which led them to recommend an innovative alternative to stabilize the moisture sensitive clays in the roadway subgrade soils.

The new design approach consists of maintaining the subgrade at a constant moisture content to eliminate shrinking and swelling of these problem soils. The subgrade is encapsulated on three sides by placing a geosynthetic moisture barrier fabric, both horizontally and vertically, to a predetermined width and depth over and beside the expansive clay subgrade.

The geosynthetic moisture barrier used was a composite material with nonwoven geotextiles for strength and protection on both sides of an impermeable film. As it was placed beneath the base stone, the moisture barrier also provides the frictional, separation and soil stabilization effects of a nonwoven geotextile. Details are given on the project design and its implementation. Installation procedures followed on the project will be discussed. Seaming techniques used to join the moisture barrier fabric will be reviewed. Unanticipated construction problems encountered during construction and their resolution will be discussed.

Introduction

Cheyenne Avenue, a two-lane highway between Losee Road and Rancho Drive in North Las Vegas, Nevada was originally constructed in 1970. Within a few years, signs of pavement distress started to appear. In the late seventies, the pavement had deteriorated so badly due to swelling and shrinking subgrade that the road became extremely wavy. This became a major safety hazard with truck trailers reportedly becoming uncoupled from the tow vehicles. Severe alligator cracking was also evident. In 1980, the worst section was reconstructed by over-excavating up to 6 ft., replacing with select fill material, type 2 and 3 road base, and 4" of A.C. surfacing. However, deterioration due to shrinking and swelling continued. Between 1982 and 1988 the old and new sections of pavement were rehabilitated by repeatedly milling off humps and patching the badly alligatored areas. In 1987, the city of North Las Vegas contracted the services of G. C. Wallace, Inc., to redesign Cheyenne Avenue into a four-lane, modern highway with two paved parking lanes. In the reconstruction design, G. C. Wallace, Inc., explored various subgrade stabilization alternates. One system was a paving fabric saturated with asphalt to be used as a moisture barrier within the pavement. Due to anticipated installation problems in the vertical trenches this system was discarded. Lime treatment was not considered due to negative experiences with lime in Las Vegas on Stewart Avenue and Owens Street. The design engineers opted for a geosynthetic moisture barrier to be designed into the project. The moisture barrier selected was Petromat* MB, manufactured by Phillips Fibers Corporation and consisting of an impermeable membrane sandwiched between two layers of nonwoven geotextile. The design was reviewed and approved by Nevada Department of Transportation, Nevada Regional Transportation Council, city of Las Vegas and the city of North Las Vegas.

Site Geology And Site Condition

Natural soils consist of soft to moderately hard clays, gravelly clays, and silty clays and gypsum. Tests performed on soils by Western Technologies, Inc., Las Vegas, Nevada, indicated that the on-site clay soils range from having a low to very high expansion potential, and the gypsum is slightly soluble in water. In addition, some of the clay soils can exhibit large expansion pressures. Based on chemical tests, the on-site soils also contain a sufficient concentration of sodium sulfate to be susceptible to chemical expansion. In addition, Cheyenne Avenue crosses some adverse geologic features which are: compaction faults, area subsidence and fissure zones. There is a potential for vertical and horizontal displacements across the compaction faults. The conclusion of the subsurface investigation report conducted by Western Technologies Inc. was, "It is not economically feasible to overexcavate enough clay soils to allow the placement of enough non-expansive soils to provide the required surcharge. Also, the potential for displacement due to compaction faults cannot be eliminated. Pavement recommendations are intended to provide the best pavement life and service within practical economic consideration."

*Trademark Phillips Petroleum Company

Concept Of A Moisture Barrier

Moisture barriers are specialty geomembranes which, when installed properly, can solve expansive soil problems. Conservative estimates of the damage caused by the expansive soils in the United States exceed 10 billion dollars a year. More than half of these damages occur to roads and streets. The expansive soil subgrades which shrink and swell with variations in moisture content are described as being in the "active zone." An active zone may range from 3 to 10 feet deep beneath the ground surface. This zone is subject to surface water recharge and evaporative water discharge. For example, if the active zone beneath a road is 5 feet thick and experiencing 10% swelling, the road surface will actually move up 6 inches. Likewise, a 10% shrinkage will cause the road surface to move down 6 inches. The movement usually occurs locally, nonuniformly, creating a rolling road surface and road deterioration resulting in a need for premature rehabilitation. Figures 1 and 2 illustrate these conditions. Cheyenne Avenue road conditions fall into this category.

By incorporating a moisture barrier, on new projects or on rehabilitation projects, the moisture content of the subgrade is held constant and the subgrade soil remains inactive with no shrinking or swelling. Geosynthetic moisture barriers are deigned to maintain a constant moisture content by stopping infiltration into and evaporation out of the active zone.

Depending upon the actual site conditions, the geosynthetic moisture barrier can be used as part or as all of the moisture barrier system to shield the active subgrade soil zone from infiltration and evaporation as shown in Figures 3 & 4. In the existing road rehabilitation shown in Figure 3, a paving fabric membrane interlayer system is used as the horizontal moisture barrier, and the geomembrane moisture barrier is used vertically. In Figure 4, the geomembrane moisture barrier was used both horizontally and vertically such as applied to this Cheyenne Avenue project.

To seek ways to minimize the destruction caused by expansive soil conditions, Texas Department of Transportation has been using geomembrane moisture barriers since 1976. According to the Texas Department of Transportation, moisture barriers used both vertically and horizontally are cost effective. The active zone in some areas of Texas is at least 8 feet deep and to maintain constant moisture content under the roadway, they utilize a 9 feet deep vertical moisture barrier. In addition to its moisture barrier function, properly designed geosynthetic moisture barriers placed beneath the aggregate base function also as strong geotextiles and provide the following benefits:

1. Provides a separation layer to keep base aggregate from pushing into the subgrade and subgrade from intruding into the aggregate. This allows a clean, free-draining base aggregate to be used.

2. Stabilizes a weak subgrade allowing approximately a 1/3 reduction in structural section thickness or an increase in the overall performance factor of safety if structural section reduction is not realized.

3. Reduces overall project cost due to savings in structural materials both initially and by preventing aggregate loss by keeping the aggregate separated from the subgrade soil.

Selection Of The Moisture Barrier For Cheyenne Avenue

The city of Las Vegas had previous successful experience with the Petromat MB geomembrane used as a horizontal moisture barrier on city streets. It was used in a test installation to evaluate the ability of Petromat MB to keep subgrade moisture content constant, thereby improving pavement performance. The design engineers and local officials on the Cheyenne Avenue project requested further evidence that this unique product could withstand anticipated construction stresses and survive the installation without any damage.

The city of North Las Vegas performed a field test of their own to evaluate the survivability and waterproofing capabilities of the Petromat MB moisture barrier. The test consisted of placing a 10 ft X 20 ft piece of material on the ground, dumping 6 inches of crushed, hard angular rock on top of the membrane as a base course, and compacting the stone to simulate the harsh realities of field installation. After compaction, they scraped off the rock from the geosynthetic moisture barrier and visually inspected it for rupture or any other damage. No damage was observed. In order to test for water migration through the membrane after the survivability test, the membrane was placed on a wooden frame having a wire mesh screen as a support. Then water was poured onto the moisture barrier which was observed for 8 hours to monitor any leakage. They did not observe any leakage. This convinced the specifiers that Petromat MB could be successfully used for this project. The Phillips Petromat MB selected for this project is a composite of two layers of nonwoven needlepunched polypropylene fabric with a polyethylene film bonded between the two nonwoven fabric layers. This unique product offered the membrane protection and the proper frictional surface necessary for use as a horizontal moisture barrier. The typical properties of the moisture barrier supplied for the project are listed in Table 1.

Cheyenne Avenue Pavement Design

The pavement design consisted of installing the horizontal moisture barrier across the full width of the roadway including the median section and the vertical moisture barrier along each edge of the road underneath the curb and gutter section and the 5 feet wide sidewalk section on the road edges. The horizontal moisture barrier was covered with 18 inches of type-2 base having a maximum size of $1\frac{1}{2}$ inches aggregate. The base was overlain with

7 inches of asphalt binder mix which was placed in two lifts. The binder mix was overlain with a 3/4" thick open-graded mix to provide a high-skid resistance-riding surface. The actual typical section of the project is shown in Figure 5.

Moisture Barrier Installation

Before the start of construction, the representatives of Southern Nevada Paving Inc., the contractor; Phillips Fibers Corporation, the manufacturer; and G. C. Wallace Inc., project quality assurance engineers, had a preconstruction meeting to discuss the installation phase of the moisture barrier.

Since the project was a part reconstruction of an existing roadway and part widening with new construction, the construction of the roadway proceeded in two phases. Phase one was construction of the new half of the roadway so that the traffic could move without disruption on the existing roadway. Once that was completed, the old highway was excavated to subgrade level and reconstructed to the new design. The sequence allowed movement of traffic without disruption regardless of construction. The Petromat MB moisture barrier was supplied in two sizes, i.e., 12 feet wide by 300 feet long rolls and 6 feet wide by 300 feet long rolls due to construction logistics.

The installation of Petromat MB started from the outer, vertical trenches inwards. The first 12 feet wide roll was installed partially in the trench and partially on the roadway subgrade. Since the trench dimensions were 1 foot by 3 feet, the first roll of moisture barrier covered the bottom 1 foot of the trench, 3 feet upwards on the inside of trench, and 8 feet of the horizontal roadway subgrade. The outer edges of the trenches and top and bottom were lined with 4 NP Supac, a nonwoven needlepunched polypropylene geotextile specially engineered for drainage applications, to form an edgedrain. A 6-inch perforated pipe was installed in the roadway edgedrain to remove water quickly. The typical section of this installation is shown in Figure 5. The reason for the edgedrain is that although the North Las Vegas area is an arid environment, when it does rain, the run-off can cause localized flooding. The drain was provided to remove the seepage from the surface run-off quickly before it percolates into the subgrade. The edgedrain will also remove water from the road base aggregate layer over the moisture barrier.

In the first new construction phase of the project, four consecutive rows of Petromat MB were installed by shingling each successive row with an overlap of 6 inches. This procedure provided a 36.5-foot-wide membrane-lined roadway section. Each overlap was seamed by using Phillips Fibers Corporation's recommended sealing method which produced a watertight seam. The seaming method used involved placing a hot rubberized asphalt mastic material along the seam and then immediately rolling the seam in place. This system, developed by Phillips, results in a strong, waterproof seam which is easily performed just ahead of the aggregate placement over the moisture barrier.

As soon as the moisture barrier was in place, 18 inches of aggregate base meeting the Nevada Department of Transportation's specification of type-2 base material was placed over it. The base was compacted in two lifts at optimum moisture content to attain maximum density, and no damage was incurred by the moisture barrier. The base was overlain with 7 inches of asphalt concrete in two 3½-inch thick lifts. The surface will be covered later on with a 3/4-inch thick layer of open-graded asphalt mix, generally known as pop-corn mix, to provide surface drainage and high skid resistance. As soon as the first phase of new construction was completed, the roadway was opened to traffic. Then, the existing roadway was excavated and regraded to the final subgrade level. The second phase of moisture barrier was installed by following exactly the same sequence as followed in the first phase installation of new section. A gap of 11 feet wide was left in the middle of the roadway which was covered with a 12-foot-wide moisture barrier tying both halves of the moisture barrier system together. The aggregate base course and asphalt binder layers were also placed by following exactly the same procedure as defined earlier. At the completion of the project, two New Jersey-type traffic barriers were constructed with an 8 feet wide decorative median in the center as shown in Figure 6. To remove surface water quickly, storm drains were constructed every so often. Underdrains were connected to storm drains. At each storm drain drop inlet, Petromat MB was flashed (cut and sealed with mastic) to the drain similar to how roofing obstructions are flashed to keep the entire system waterproof.

Construction Problems

The project went very smoothly. Traffic was kept to a minimum on moisture barrier installation. There were minor folding problems in the first roll, which was partially placed in the trench, when stone was dumped in the trench. The dropping of the aggregate into the trench pulled the roll somewhat, thus causing the horizontal portion of the moisture barrier to fold. Any folding at seams was treated with extra seaming mastic to assume a good seam. Wind was a problem off and on, however, due to past installation experience under similar conditions, the contractor was advised to place some ballast on the membrane during installation regardless of the conditions. Also, a minimum amount of moisture barrier was placed ahead of the base aggregate placement. Otherwise, the project went more smoothly than actually expected. The contractor, the city of Las Vegas, and the design engineers responsible for the quality control and quality assurance were all pleased with the ease in which the geosynthetic moisture barrier system was installed.

Project Results

The project was completed in 1990 and to date the city has been very pleased with the results. This reconstruction has performed better than any other methodology they have tried in the past. On the 5-mile reconstruction, they have noticed only one small swell area which they suspect was due to a localized construction problem. Construction quality control is important when utilizing moisture barriers.

Other Moisture Barrier Applications

The objective of this paper was to present an example of the use of a moisture barrier as a geologic/geotechnical tool to help solve moisture related problems. Although the horizontal and vertical road base moisture barrier applications are probably the widest uses for the product, it has been used to address several other moisture problems as briefly discussed below.

- Moisture barriers are placed vertically in trenches along the sides of roads, i.e., along irrigated medians and city streets, and around green irrigated areas in parking lots to keep the irrigation water from getting beneath the pavement section and causing damage.
- Moisture barriers are used as covers on slopes to keep moisture out of a moisture sensitive or a failing slope.
- Moisture barriers are used as liners to line channels to prevent seepage and potential slope saturation and failure. For both the last two applications, Petromat MB with the nonwoven on either side of a film allows the stable placement of cover soil due to the good frictional characteristics of the fabric.
- Moisture barriers are used in lieu of a traditional permeable geotextile to help lower construction costs and extend construction seasons by being immediately placed on prepared, compacted subgrades or finish grades. This keeps the subgrade soil ready for base stone placement even if there is a rainstorm before the stone can be placed. This eliminates the costly waiting game or the removal, replacement, and recompaction of material which softened and lost its density due to rain.
- Moisture barriers are also used as environmental barriers, i.e., under parking lots in environmentally sensitive areas to keep salts and oils out of the groundwater.
- Depending on the soil and the construction conditions, it may be less expensive to place a moisture barrier horizontally to a designed distance from a pavement or structure to give the same effect as a more difficult to place vertical barrier adjacent to the structure.

Conclusion

Moisture barriers are extremely versatile geotechnical tools which should be more widely utilized. The structure of a moisture barrier such as Petromat MB by Phillips provides a cost effective solution to common moisture related problems while adding the extra dimension of serving as a strong nonwoven geotextile. Geologists or geotechnical engineers now have at their disposal many new geosynthetic materials such as these moisture barriers and such as composite drainage materials to help them deal with the greatest geotechnical problem——moisture.

Table 1

Moisture Barrier Fabric Specification

Moisture Barrier Fabric - Composite fabric shall consist of two layers of nonwoven needlepunched polypropylene fabric with a minimum layer of 4 mils thick polyethylene film bonded between the two nonwoven fabric layers. The moisture barrier shall meet the following minimum properties:

Property And Test Method	Value
Weight (oz/yd^2) ASTM D-3776	9.0
Thickness (mils) ASTM D-1777	50
Tensile Strength (lbs) ASTM D-4632	150
Elongation (%) ASTM D-4632	35
Mullen Burst (psi) ASTM D-4833	260
Puncture (lbs) ASTM D-4833	80
Trapezoidal Tear (lbs) ASTM D-4533	45
Permeability (cm/sec) ASTM D-4491	0
Resistance To Soil Burial ASTM D-3083	No Change
Abrasion Resistance (% strength retained) ASTM D-4886 (can be abraded on either side of fabric)	60

Note:

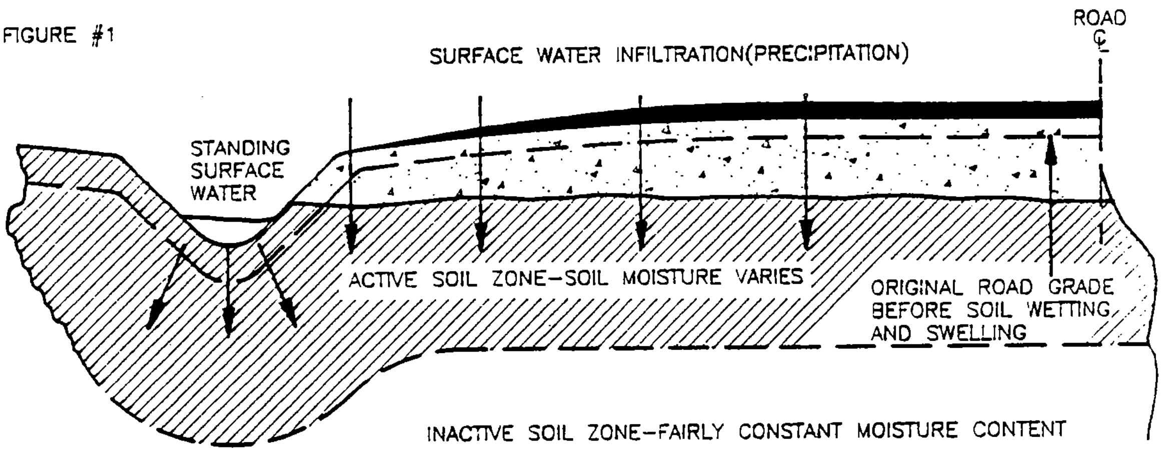
1. The nonwoven needlepunched fabric on both sides of the membrane is to provide a high degree of survivability, i.e., during installation and construction it provides a cushion against sharp objects that is not available in single-sided membranes. Also, without a film exposed, the product has very good friction against both underlying soil and against the cover material.

2. Seaming of the moisture barrier fabric shall be done in accordance with manufacturer's recommendation.

3. Petromat MB, as manufactured by Phillips Fibers Corporation of Greenville, South Carolina, meets the properties listed above.

SOURCES OF SOIL WETTING(RECHARGE)

FIGURE #1



SOURCES OF SOIL DRYING(EVAPORATIVE DISCHARGE)

FIGURE #2

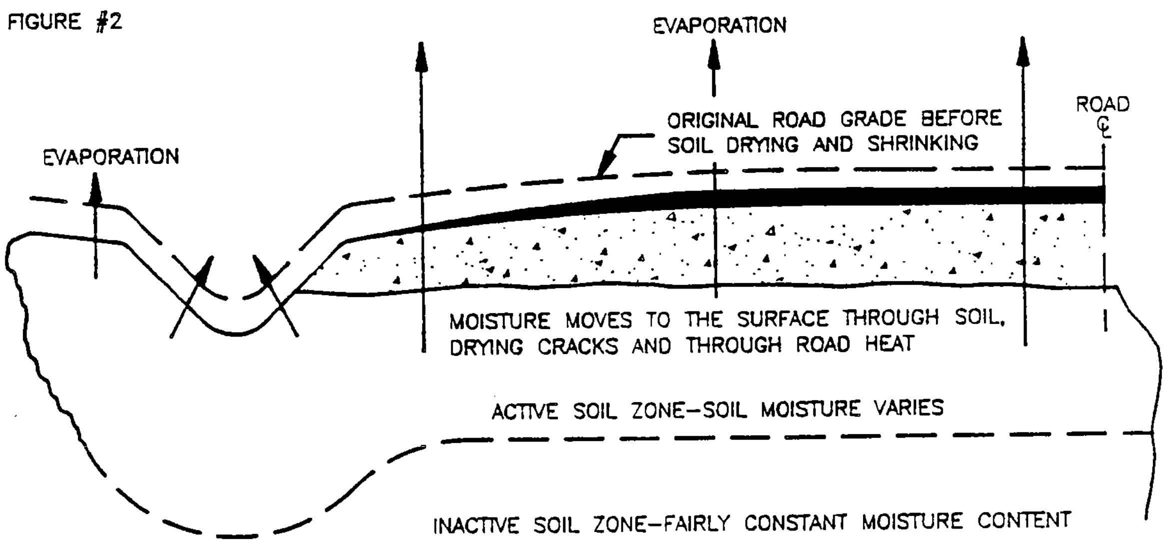


FIGURE 3 1) As a vertical moisture barrier

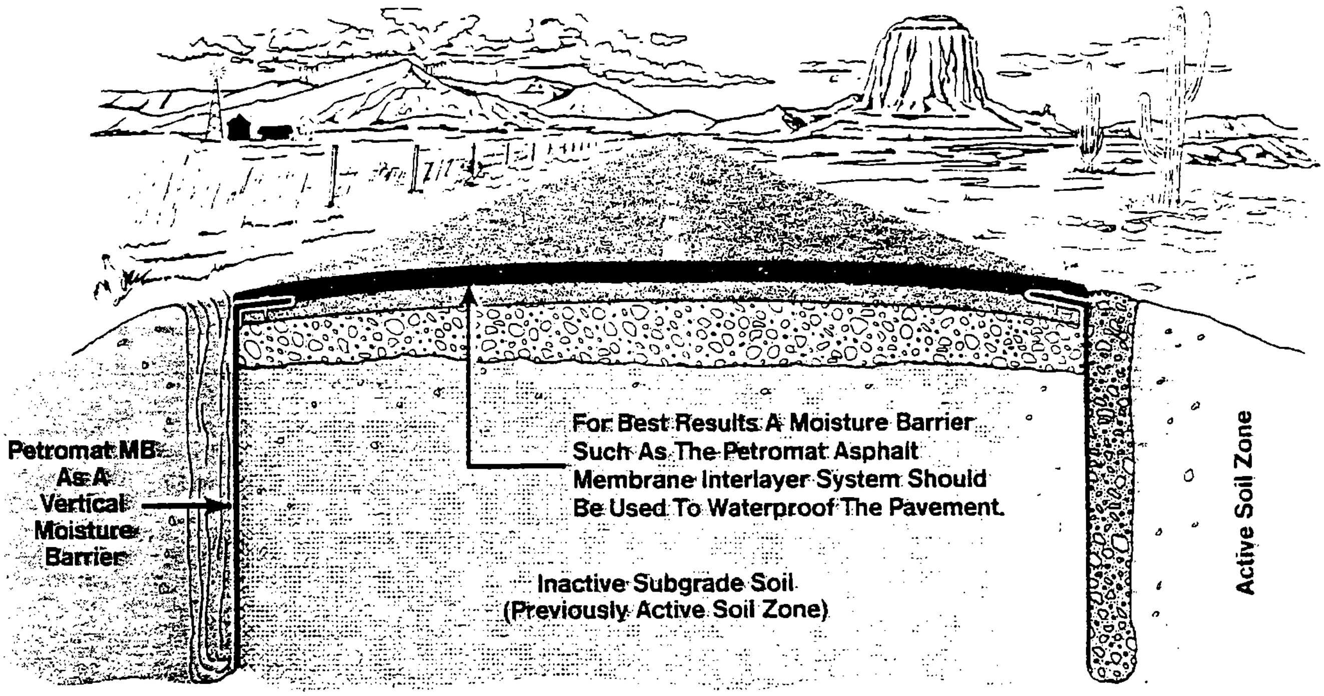
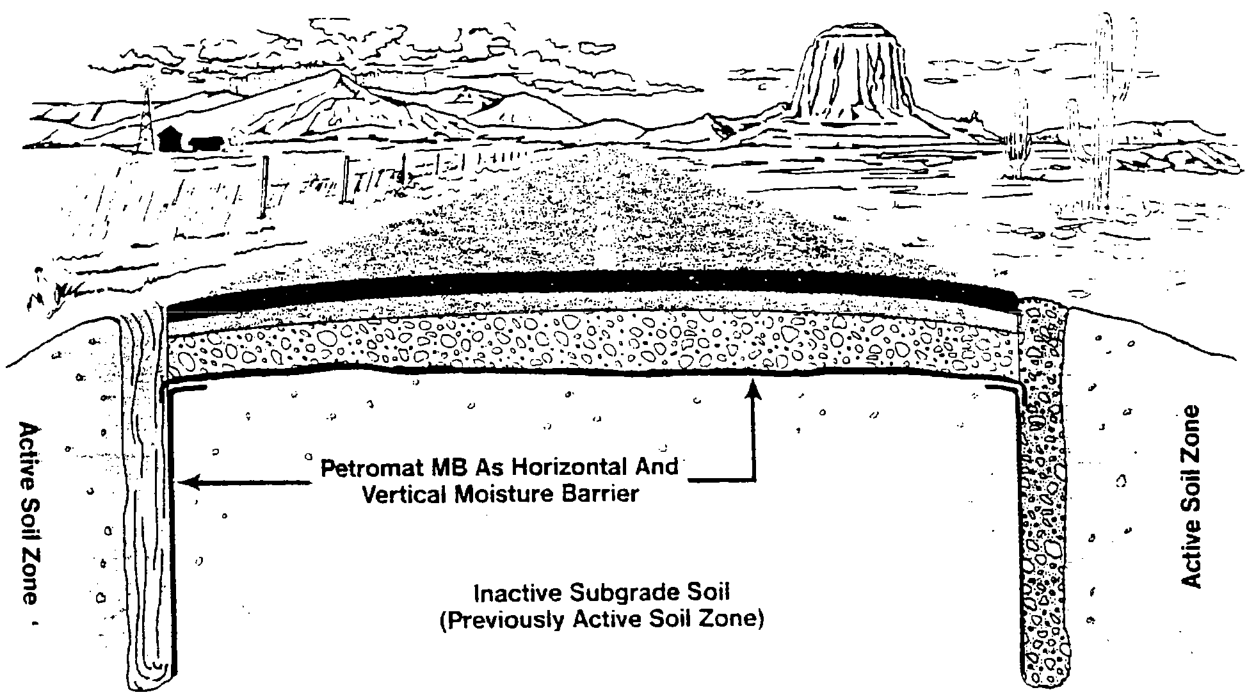
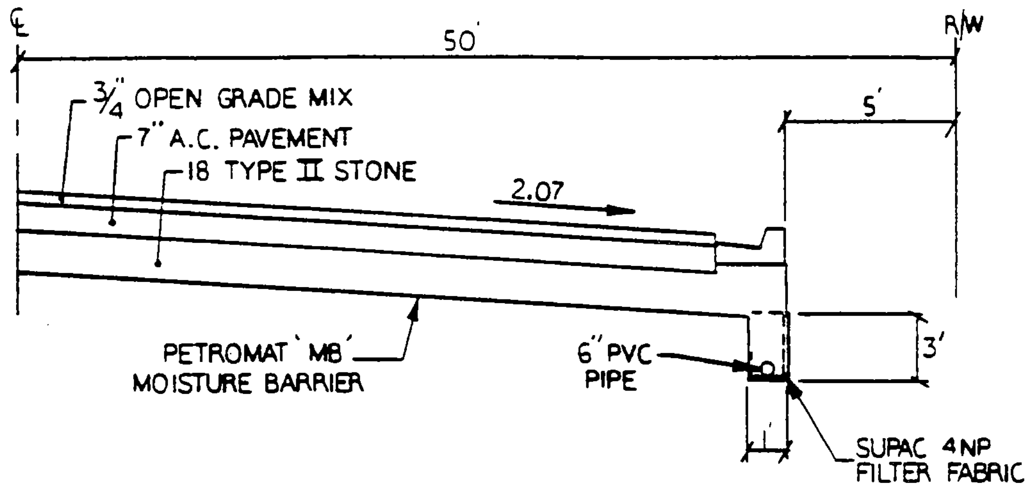


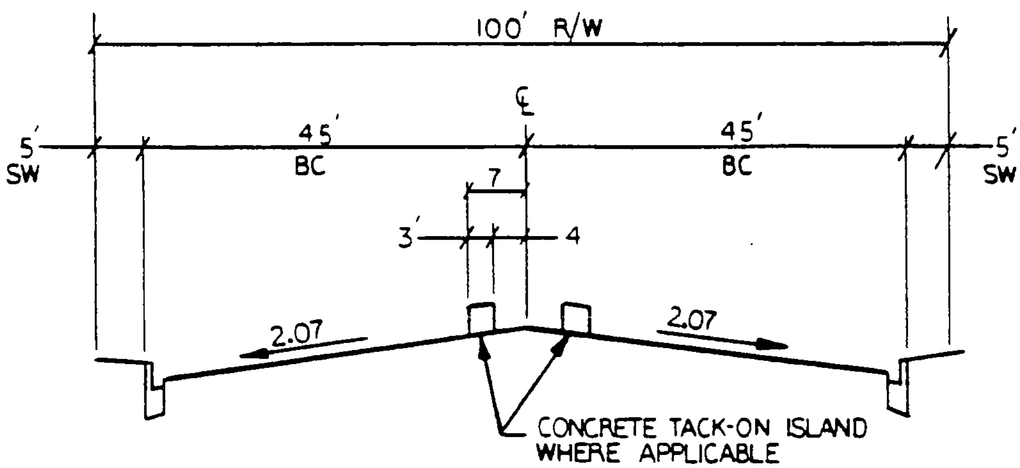
FIGURE 4 2) As a vertical and horizontal moisture barrier





MEDIAN TYP. SECTION

FIGURE 5



MEDIAN TYP. SECTION

FIGURE 6

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INVESTIGATION FOR LANDFILL EXPANSION IN A BEDROCK AREA, SOUTHCENTRAL INDIANA

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INTRODUCTION

The Jackson County Landfill expansion site is located in the western half of Section 16 and the eastern half of Section 14, T5N, R3E, southern Owen Township, western Jackson County, Indiana. It is situated on the 7 1/2 minute Medora quadrangle topographic map. The original expansion was to be 27 acres, but when found to be too small, the area was enlarged to 41 acres. The expansion site adjoins the currently operational Jackson County Landfill of 54.5 acres. Rumpke Inc. owns a total of 300 acres at the site. The current landfill has an irregular shape which runs lengthwise from northwest to southeast. The expansion area lies to the northeast of the current operation.

Most of the landfill waste originates in Monroe and Lawrence Counties. Much of the remaining waste from the area is sent to Rumpke's Uniontown landfill, also in Jackson County. Rumpke wishes to expand its Medora landfill to accommodate the additional waste which will develop when the Uniontown Landfill closes within a year. Closing of the Uniontown landfill will greatly increase the volume at the Medora site.

Medora Landfill receives about 250 to 300 tons of waste per day although these numbers vary considerably. Of the waste received, approximately 25 percent is municipal waste. Less than 25 percent is special waste including asbestos, contaminated soil, sewage sludge, steel slag, and soil waste from the removal of oil or gas storage tanks. The remainder is construction/demolition materials supplied by contractors, builders, and roofing operations. These numbers also vary greatly from day to day. There is no provision for leachate removal from the current landfill. Downward migration of leachate is assumed to be negligible because of the low permeability of the bedrock, and minimal outward leachate migration.

A unique feature of the expansion site is that existing bedrock will be excavated by blasting to increase waste volume. In the expansion area, about seventy feet of rock will be blasted and removed to provide additional space for waste disposal. The removed rock will be used for construction of semi-permanent roads on the site and Rumpke Inc. will apply for a special permit to use rock as landfill cover material. Additional cover will also be available from property to the south of the landfill where soils are up to 20 feet thick. A study of the hydrogeology and design considerations of the landfill expansion comprise the M.S. thesis for the first author (Pittenger, 1993).

GEOLOGIC SETTING

Regional Geology

The Jackson County Landfill occurs in the Mississippian siltstones of the Borden Group in western Jackson County, Indiana. The site is south of the Wisconsinan glacial boundary and just within the Illinoian glacial boundary located to the west. Figure 1 shows the location of the site relative to other geologic features.

The Mississippian siltstones of the area consist of the Edwardsville Member of the Muldraugh Formation of the Borden Group. The Edwardsville Member was deposited as deltaic sediments and part of the Osagean Borden Delta which extends into Kentucky and Illinois. Its source appears to have been from the northeast (Kammer et al., 1983) however these sediments have eroded away. Thicknesses of the Borden Group varies from 485 to 800 feet and the Edwardsville Member varies in thickness from 40 to 200 feet thining to the southwest.

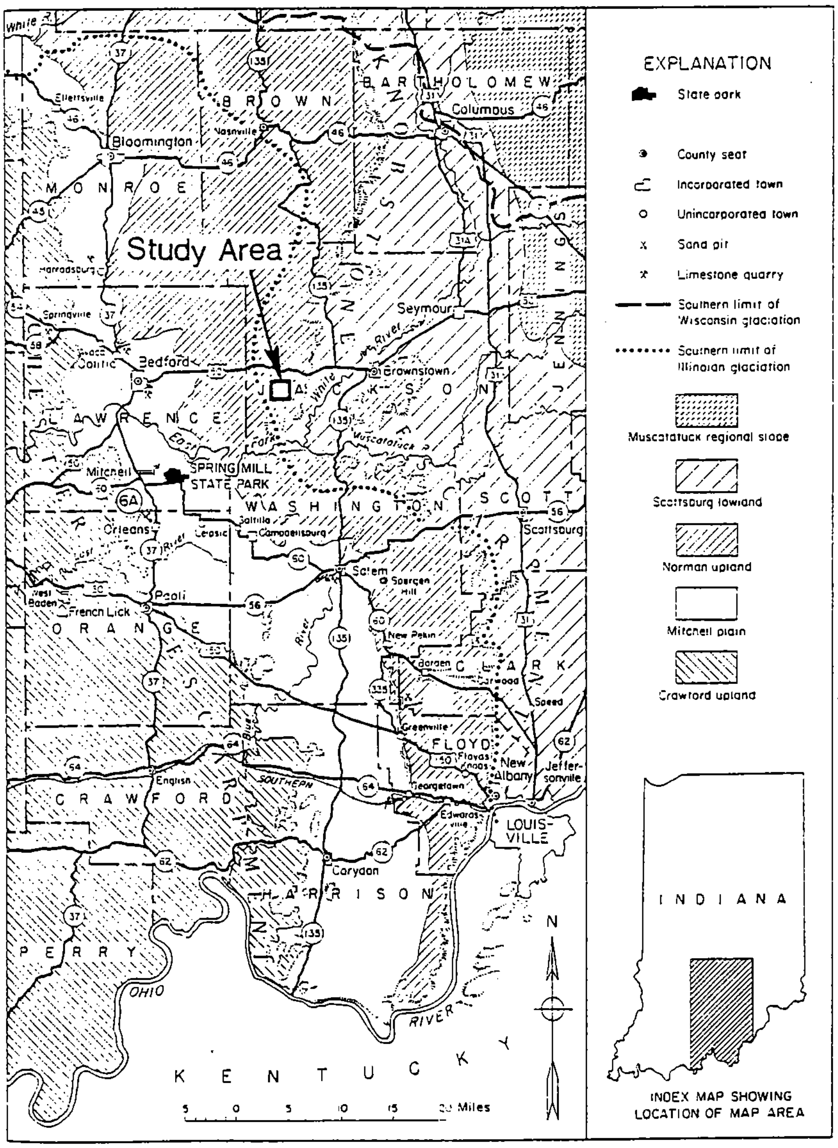

Figure 1. Location of the site relative to physiographic features in south-central Indiana (after Perry, Smith, and Wayne, 1954).

Stockdale (1931) classified the Edwardsville as a formation and it is still referred to in that manner in some literature. However Smith (1965) and Shaver et al.(1970) reclassified it as a member of the Muldraugh Formation with the Ramp Creek Member overlying it and the Floyds Knob Member below (Kepferle, 1977). Stockdale (1931) estimates the thickness of the Borden Group in the Medora area at 600 feet. The Edwardsville in the Medora area is from 70-120 feet thick and consists of alternating layers of shale, siltstones, and sandstone with abundant worm marks (Stockdale, 1931).

The Edwardsville was deposited as delta platform sediments in a deltaic area with two lobes that extend from around Terra Haute, Indiana to Kentucky in an environment of strong current actions. These currents account for the lack of features such as ripple marks, current bedding, and cut-and-fill structures as reported by Ausich, Kammer, and Lane (1979) and Kammer, Ausich, and Lane (1983), who have studied fossil communities and depositional environments extensively in this area. The deposition of the delta platform was subject to a high degree of variability resulting in sandstone channels, interdistributory siltstones and mudstones, and skeletal carbonate banks. This variable deposition likely accounts for the discontinuous sandy zones which are penetrated by many of the water supply wells in the depositional area, and the difficulty drillers encounter when trying to locate these water bearing zones.

Site Geology

The general topography of the area consists of a highly dissected sloping terrain (Bleuer, 1970) with shallow depth to bedrock. It is a forested area with an intermittent stream flowing over bedrock outcrops on the northeast side of the site. Surface elevations range from 650 to 850 feet, MSL. Within much of the expansion area, the soil has been excavated to reveal the local bedrock. The uppermost layers of the bedrock consist of hard, light-gray shale which breaks into three to five inch thick layers with an irregular surface texture. Underlying the shale is the medium-gray, fossiliferous siltstone comprising the majority of rock encountered in the rock corings and is described in more detail below. Siliceous geodes ranging upward to softball size occur in gullies and stream beds in the area. They are commonly brought to the surface during bulldozing operations on the current landfill.

Soil Characteristics

Soil at the Jackson County landfill site consists of up to about 20 feet thick of brown clayey silt with low plasticity. Borings MXP-1, 2, 3, and 9 encountered more than ten feet of soil before reaching the medium gray siltstone bedrock underlying the site. The soil was damp to slightly damp in most locations, but no significant amount of water was encountered. Samples indicate a clay loam which varies in color from dark yellowish-orange to dark red, and is sometimes mottled. The soil is underlain in most areas by a pale brown or pale yellow weathered siltstone from one to five feet thick. Below this layer is the medium gray siltstone that extends downward for the length of the borings. Soils were tested in 1989 for a previous expansion. Zorn Engineering (1989) reports that hydraulic conductivities from five samples of soil ranged from 2.3×10^{-7} to 2.8×10^{-8} cm/sec. Cation exchange capacity (CEC) was listed at 22.5 mEq/100gm although this value appears to be rather high for these loamy soils. Liquid limits ranged from 32 to 41 and plasticity indexes ranged from 6 to 18 for the samples.

Test Boring Summary

A total of ten locations in the expansion area were selected for exploratory drilling. These locations are illustrated in Figure 2 with the prefix MXP. Some locations had more that one exploratory hole. Piezometers were installed at all ten locations with borings MXP-1, MXP-5, and MXP-10 having both deep and shallow piezometers. At these locations, piezometers were off-set only a few feet from each other. Drilling logs were developed only for the deep holes, assuming that the shallow holes would provide the same lithology. Two inch rock corings were also obtained at these three locations yielding a total of three holes each for these three locations. MXP-1 and MXP-10 had enough soil cover that split spoon samples were taken. For the other eight locations the soil had been removed by a bulldozer prior to drilling. Holes for piezometer installation were drilled by Faulkner Drilling Company using the dry air rotary method and a six inch diameter roller bit. Consequently, depth measurements and rock layer descriptions were less accurate from these holes where no drill cores were obtained. Instead, descriptions for those holes were based on cuttings.

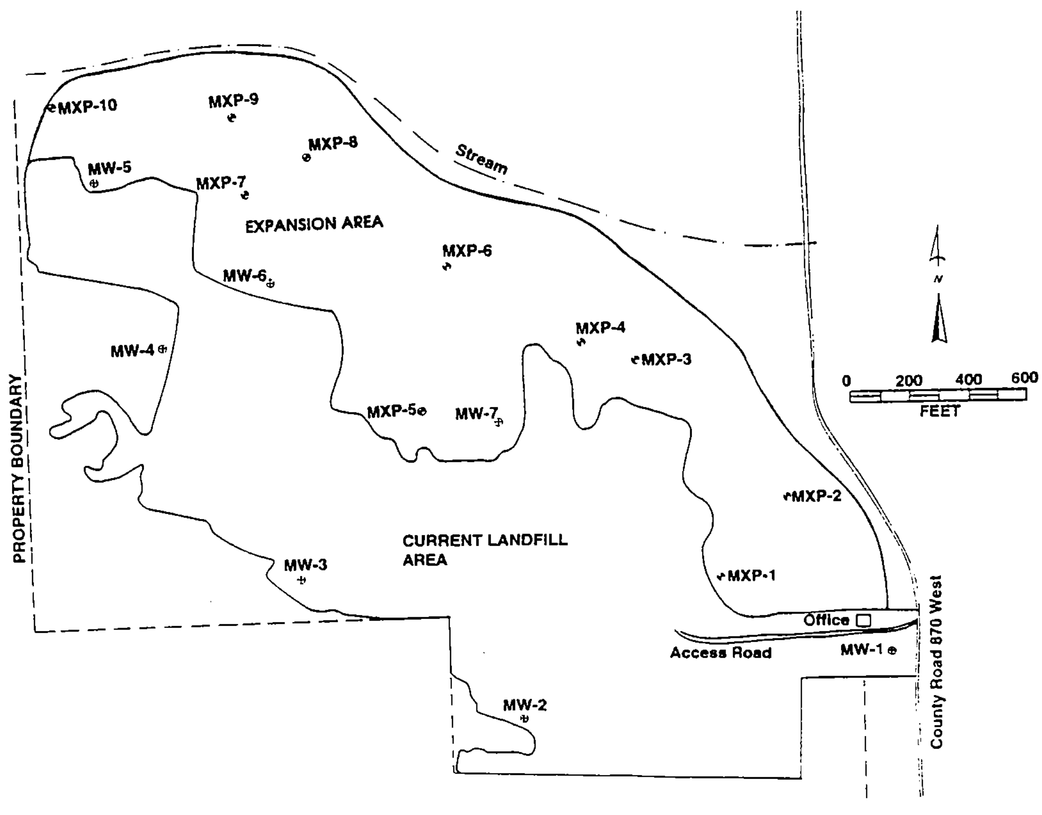

Figure 2. Layout of site with locations of exploratory borings (after Geosciences, Inc., 1989).

Nearly all bedrock below the site is a medium-gray siltstone with some fossils and is slightly calcareous. Some geodes occur in the deeper cores along with some pyrite flakes in some of the cores, however such features could not be distinguished in boreholes drilled with the air rotary method because the rock was pulverized. The siltstone cores show a wavy, light-gray pattern in cross section owing to the fossil fragments. Iron stains up to two inches thick were occasionally found within the upper ten to twenty feet of the siltstone cores, as were a few fractures. In general, however, core recovery was excellent with many of the cores remaining in complete ten foot lengths

An indication of the small amount of ground water in the area is shown by the air rotary borehole logs. Throughout the drilling, cuttings came up alternately damp, slightly damp or dry. No significant water producing layers were perceptible, and frequently the cuttings would go from damp to dry several times within a few feet of drilling whereas other times they would be damp or dry for fifteen to twenty foot intervals. There were no cases of noticeable loss of drilling fluid, nor was there a noticeable, sudden increase or decrease in drilling rates that would indicate a softer layer or a void. There were occasional vibrations of the air rotary rig when resistant layers were encountered.

HYDROGEOLOGIC INVESTIGATION

Hydrology and Hydrogeology

The shale and siltstone bedrock in the area has an extremely low permeability and is ideal for the location of a sanitary landfill, and the of contamination of subsurface water is highly improbable (Bleuer, 1970). Subsurface fracturing is thought to be minimal as indicated by the excellent recovery of the rock cores and the lack of moisture on the cuttings and rods brought up during air-rotary drilling. It is unlikely that new fractures resulting from the blasting would be extensive enough to intersect significant sources of water, as water does not circulate well through these rocks (Harrell, 1935).

Logs of water wells within two miles of the site indicate the absence of a productive aquifer in the area. Many of the wells were screened in shale aquifers, and were either dry or did not produce adequate amounts of water (Geosciences Inc., 1989). Wells that did produce usable amounts of water were due to the chance hitting of permeable lenses (Zorn Engineering, 1989). Municipal water is now supplied to all residents of the area via water lines.

Surface water drainage is extensive because of the shallow depth to bedrock and its low permeability, as well as the rugged topography. An intermittent stream occurs just to the northeast of the site. The stream has an elevation of about 720 feet MSL at the southeast and 700 feet at the northwest. It is fed by surface drainage including drainage from of the landfill. In a site inspection by the first author, drainage pathways to the stream were easily located. At that time, the stream was flowing directly along the top of the bedrock with almost no soil in the stream itself.

Geophysical Well Log Summary

Geophysical borehole logs were obtained on nine of the borings at the site by RMS Geophysical Incorporated. The calliper log was recorded with only moderate resolution but does not show significant change in borehole diameter that could be associated with a change in lithostratigraphic units. MXP-1 was the only two inch hole logged and its diameter is more variable as would be expected owing to the slower drilling rate for rock core samples. All other holes logged were six inch diameter holes drilled by air rotary.

Natural gamma logs were recorded for all nine of the holes logged and are summarized in Figure 3. Again, no noticeable change in lithology was discernable, and the fluctuations appear to be predominantly caused by variations in the clay mineral content of the siltstone. Many of the limestone layers described from the rock core are too thin to show an appreciable decrease in gamma radiation. MXP-1 and MXP-1D being 10.6 feet apart, were the only holes logged that are in close proximity to each other and only general correlation can be depicted between them.

Density (gamma-gamma) logs were also performed on all nine holes logged and no significant correlations can be made for the subsurface strata. These logs are summarized in Figure 4. Because of the variations in water zones in the rock described during drilling, it is difficult to determine if the increased count rate for the peaks is due to porosity variation in the rock or to changes in water content. For this reason and because of a lack of mineralogical data, a quantitative estimate of porosity would be highly questionable based on geophysical well logs alone. Both the gamma logs and the density logs indicate that although there is no major lithologic change in the bedrock, there is slight variation in rock chemistry as well as moisture bearing aspects. The water in the rock is probably vadose water and does not constitute a potentiometric surface. The variations in rock chemistry are most likely due to changes in the depositional environment, i.e. higher depositional velocities yielding a lower clay content.

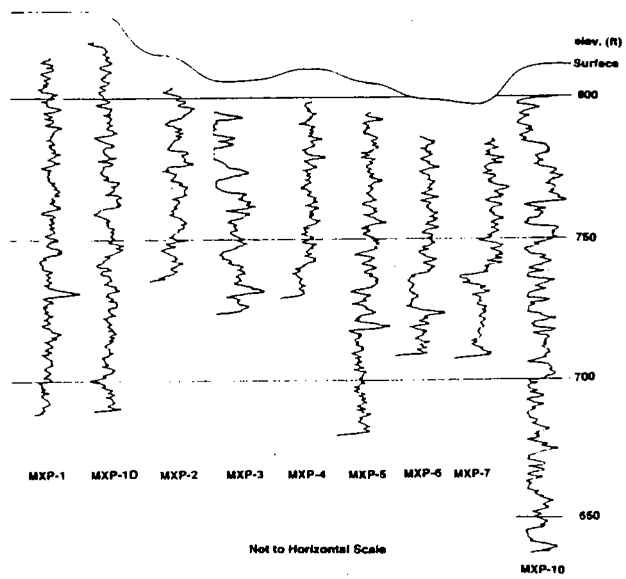

Figure 3. Summary of natural gamma logs for exploratory boreholes.

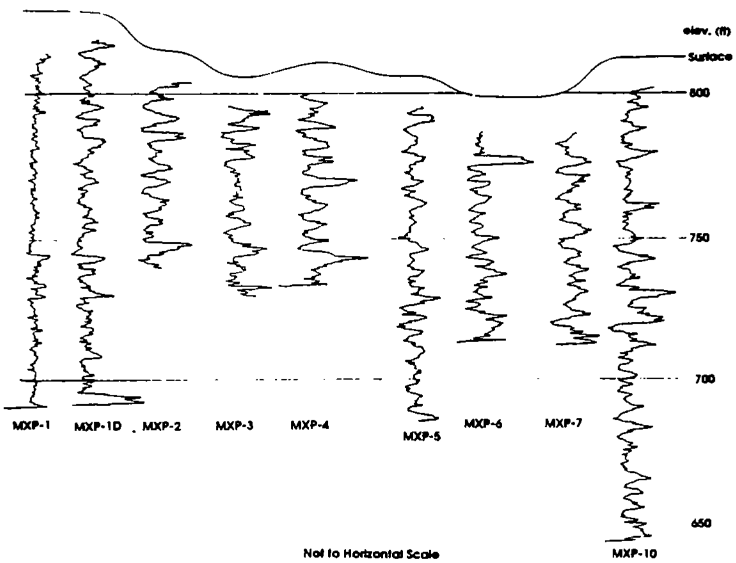

Figure 4. Summary of density (gamma-gamma) logs for exploratory boreholes.

The logs indicate that variations in water content, rock chemistry and mineralogy, and porosity may be more extensive than indicated by the appearance of the rock cores alone. However the geophysical data provides no evidence of a significant change in lithology which would indicate the presence of a water bearing zone.

Piezometric Surfaces

Monthly water level measurements were obtained for each of the piezometers after installation. Water levels were also taken for the monitoring wells. These water levels were used to construct the piezometric surface maps in accordance with the regulations 329-IAC-2-11-6-b-2 (Solid Waste Management Board, 1988). These regulations require illustration of equipotential lines, and an indication of the direction of flow. Water levels were taken for four months at monthly intervals, and a typical piezometric surface is illustrated in Figure 5.

For proper interpretation of a piezometeric surface and flow lines, it is necessary that the aquifer is homogeneous and isotropic (Driscoll, 1986). However, there is reason to believe that these criteria have not been met in the siltstones of the area. Fractures, although thought to be minimal, result in an increase in permeability. The undulating layering of the rock at the surface and the variable moisture encountered in the drill cuttings indicate a possibile anisotropy. From the characterization of the subsurface obtained in the drilling, accuracy of a piezometric surface in this area is suspect. It would appear from the water levels that there may be both a shallow and a deep piezometric surface with a downward hydraulic gradient ranging from about 0.5 to 1.2. For the data taken from the monitoring wells, which go to a much shallower depth than the piezometers, it appears that there is no hydraulic connection between the monitoring wells and the piezometers with the possible exception of MW-1. In fact, MW-6 and MW-7 are frequently dry.

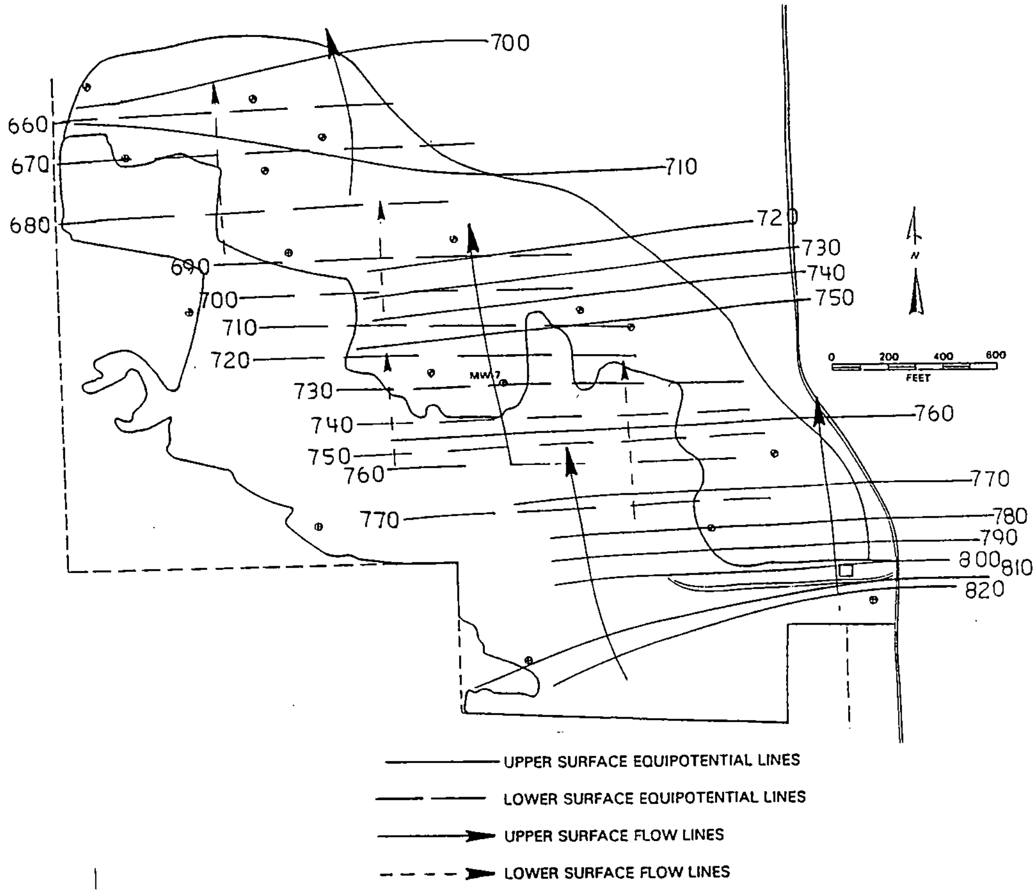

Figure 5. A typical piezometeric surface constructed from the ground water data for November 3, 1992.

Water levels were not provided for some of the piezometers for 10-7-92, and data from 8-25-92 were said to be questionable by Rumpke's environmental manager, Ron Strube. Therefore the most accurate assessment can be made using the other dates. The horizontal hydraulic gradient is quite steep with typical values of 6 percent along the upper surface and 8 percent for the lower surface with maximums as high as 10 percent. All diagrams show a general flow to the north as would be expected because of the stream location and the lower topography at the north side of the expansion. Some curvature of the flow lines occurs. Equipotential lines appear to follow the contours of the ground surface, decreasing in elevation northward as the topography lowers toward the stream. However, there is not enough information in the form of detailed contour maps of the area to determine accurately the relationship between topography and water level.

Water levels ranged from dry to greater than 80 feet above the top of the gravel pack. The occasional high water levels suggest that significant water pressure exists in the subsurface to warrant concern for contamination by the landfill, particularly in the southeast portion. This could possibly be increased by vertical fracturing in the rock caused by drilling stresses. A general pattern of decreasing pressure head towards the northwest can be observed. Depths to water from the ground surface were more consistant throughout the survey and were typically 50 to 60 feet below the surface. This lends support to the possibility that the piezometric surface is more related to the topography than to subsurface conditions. Water levels have been summarized in Table 1.

	8/25/92		9/24/92		10/7/92		11/3/92	
	DEPTH	Height of	DEPTH	Height of	DEPTH	Height of	DEPTH	Height of
	TO	Water	TO	water	TO	water	TO	water
WELL	WATER	Column	WATER	column	WATER	column	WATER	column
MXP-1D	45.23	96.97	116.20	26.00	116.18	17.02	54.51	87.69
MXP-1S	47.92	44.28	53.77	38.43	53.74	31.26	52.48	39.72
MXP-2	54.01	26.39	65.72	14.68	65.71	6.49	54.67	25.73
MXP-3	52.36	29.64	54.65	27.35	54.65	18.75	52.31	29.69
MXP-4	64.12	18.08	66.11	16.09	66.09	9.21	64.10	18.1
MXP-5D	49.94	76.06	118.08	7.92	118.10	-0.20	79.29	46.71
MXP-5S	51.14	24.86	55.52	20.48	55.52	11.98	51.59	24.41
MXP-6	20.01	69.59	87.22	2.38	87.21	-4.71	82.69	6.91
MXP-7	60.80	29.20	87.79	2.21	NA		87.80	2.2
MXP-8	48.73	1.37	50.56	-0.46	NA		50.52	-0.42
MXP-9	46.82	7.68	49.01	5.49	NA		48.40	6.1
MXP-10D	94.31	80.99	152.91	22.39	NA		164.04	11.26
MXP-10S	118.52	6.88	124.73	0.67	NA		122.10	3.3
NA-not available								

Table 1. Summary of water level data for the four dates of measurements.

LANDFILL DESIGN

Design of Landfilled Area

For the Jackson County Landfill, rock will be excavated to increase the volume for new waste disposal. The south wall of excavation will be vertical. Any slope to the wall may lead to leakage under the liner system or slope instability of the waste. As the waste is placed, the final contours of the expansion will yield a continuous surface which will blend in with the final contours of the existing fill. The base of the landfill slopes to the northwest to allow for drainage of leachate through the leachate collection system.

The stream to the north of the waste boundary will be diverted to the north and a berm will follow the north side of the landfill to direct surface runoff. This will protect the area from increased runoff from the fill that could cause erosion or be otherwise damaging. At its thickest point, the waste will be more than 100 feet thick, in the vicinity of what is currently MXP-5.

The expansion has a calculated area of 41 acres and a calculated volume of 65 million cubic feet, or 2.4 million cubic yards of airspace. This volume was calculated by the first author by overlaying the final contours on the base contours and finding the area associated with each contour elevation. This area was multiplied by the contour interval of ten feet to give a volume for each contour interval. The resulting 2.4 million cubic yards is consistent with estimates made by engineers at Rumpke, Inc.

Typically, one fifth of landfill volume is occupied by daily and final cover (O'Leary and Walsh, Sep., 1991) leaving 1.9 million yards of space available for waste. At the current rate of 300 tons per day and density of 500 to 1000 pounds per cubic yard, and at an incoming rate of 600 to 1200 cubic yards per day, five days per week, the expansion could have an estimated life of 5 to 10 years. However the actual life is likely to be shorter owing to the iminent closing of the Uniontown Landfill and the subsequent routing of that waste stream to the Jackson County Landfill.

Shortage of cover material is a major concern for site expansion. One solution being considered is the use of soil from the adjacent property to the south. This property is also owned by Rumpke, Inc., and testing is planned to evaluate the soil as cover material. Based on the above calculations, 0.5 million cubic yards of cover material will be required. Soil in the adjacent property is likely to extend to a depth of ten feet or more based on borehole logs where soil was not removed prior to drilling. Assuming a soil depth of ten feet, approximately 150 acres will be needed to supply the cover material without accounting for compaction.

CONCLUSIONS

The need for increased landfill space in Jackson County is becoming more urgent with the closing of the Uniontown Landfill, and the limited available lifetime of the current Jackson County Landfill. A lateral expansion of the current Jackson County Landfill is one option to provide increased landfill space to provide the solid waste disposal needs for the area.

Strong evidence indicates that no aquifers exist at the landfill and the region would not be susceptible to contamination because of the low bedrock permeability. Evidence includes: the low production of water wells surrounding the site, the minimal amount of fracturing in the bedrock through which contaminants could migrate, the low amounts of water in nearly half the piezometers and monitoring wells, and the absence of significant subsurface water encountered during drilling.

Evaluation of landfill suitability for the area has been provided by other researchers. Regarding suitability of siltstone for waste disposal, Bleuer (1970) states "Excellent disposal sites can be found in these materials provided surface runoff is controlled", and "No likelihood of contamination of subsurface water exists". Harrell (1935) states that "the Borden Group which underlies most of the (Jackson) county is relatively impervious", and that "the rocks are of such fine texture that ground water does not pass through them readily". Concerns for the area include steepness of the topography (USDA, 1990), lack of material for daily cover, and control of surface water (Bleuer, 1970, and USDA, 1990). These problems can be overcome through proper engineering and design of the new landfill.

The location meets other landfill site criteria as well. The soil is a clay loam suitable for cover, with a permeability ranging from 10^{-7} to 10^{-8} cm/sec. The location is devoid of wetlands or areas of high water table, and is well isolated from all but a few residential homes. Municipal water has been supplied to the homes in the area, and low production water wells are no longer necessary for domestic water supplies. Piezometric surfaces have been examined, and although they show that some subsurface water exists, it is quite unlikely that this would constitute an economically useful supply.

With proper engineering and design, the expansion of the Jackson County Landfill can be accomplished with minimal threat to both the ground and surface water in the area. During its lifetime the landfill expansion can provide a sound repository of solid waste for the people of Jackson County in the years to come.

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A STUDY OF SELECTED LANDSLIDES ALONG CINCINNATI ROADWAYS

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ABSTRACT

Five sites in the Cincinnati area, which had experienced slope failures, were selected for detailed study. The sites are designated as Huffman Court, Groesbeck Road, Interstate 74, West Fork Road, and Clough Pike. Slope movement was monitored at the Huffman Court and Groesbeck Road sites for one year. Soil samples from all sites were tested to determine grain-size distributions, Atterberg limits, and strength parameters. These properties were used to characterize the soils at each site and to analyze the stability of slopes using the REAME program.

Results of the study indicate that slope failures in the Cincinnati area generally tend to be slow-moving rotational slumps, particularly where glacial clays or man-made fills are involved. Where road cuts have been made through colluvium overlying weathered bedrock, a combination of rotational and translational movement is more common. Invariably, human activity seems to have accentuated slope movement through oversteepening of the slopes and overloading their tops. Erosion at the toes of slopes by tributary streams, and extrusion of soft clays in toe areas under the influence of overburden pressure, also contribute to slope movement. Stability analysis indicates that pore pressure build up prior to failure is involved in most cases. A typical remedial measure to combat slope movement in the Cincinnati area is the construction of concrete caisson retaining walls.

INTRODUCTION

The Cincinnati area, Hamilton County, along the Ohio River, is an area of high landslide incidence (Radbruch-Hall et al., 1982). It has the highest per capita cost of landslide damage of any metropolitan area in the United States (Fleming et al., 1981). The cost of landslide damages for the 1973-78 period exceeded $5.1 million per year (Fleming and Taylor, 1981). This amounted to a per capita cost of $5.80 per year compared to the per capita costs of $1.30 and $2.50 per year for the San Francisco Bay region and the Allegheny County (Pennsylvania) area, respectively, both of which are well known for their landslide problems. Between 1988 and 1992, the city of Cincinnati allocated $7.5 million for landslide remediation work and the projected cost for stabilization projects between 1993 and 1997 is $8.5 million (Pohana, 1992).

The surface geology in Hamilton County is a consequence of Kansan, Illinoian, and Wisconsinan glaciation. Before Kansan glaciation, the drainage system in the Cincinnati area flowed northward and the land elevation was 400-500 feet (120-150 m) higher than it is today. The north flowing drainage was dammed by the Kansan glaciation resulting in large lakes and widespread deposition of lakebed clays (Hough, 1978). The Illinoian glaciation also resulted in extensive deposition of lakebed silts and clays (Hough, 1978). The Wisconsinan glaciation did not actually reach the Cincinnati area but a great deal of outwash material from this period covered the previous glacial deposits (Hough, 1978). Entrenchment of streams began soon after the Wisconsinan glaciation and continues today. Many of the streams are confined to narrow valleys which flood easily during periods of heavy rainfall, resulting in an accelerated rate of erosion and downcutting. The steep slopes that have been carved into the bedrock are often covered with the colluvium produced from the weathering of the underlying Ordovician shales.

The combination of steep slopes, abundance of fine-grained glacial deposits, easily weathered bedrock, humid climate, and alteration of slopes through human activity has resulted in a large number of damaging landslides in the Cincinnati area. The objective of this study was to perform a detailed investigation of a few representative landslides and see how they have impacted the roads and residential developments in the Cincinnati area. An additional objective was to assess the role of human activity in promoting these landslides.

METHODOLOGY

Field Investigations

Five sites which had experienced slope failures were selected for detailed investigation. The sites selected included the Huffman Court, Groesbeck Road, Interstate 74, West Fork Road, and Clough Pike sites. Figure 1 shows the approximate locations of these sites. Three of the sites (Groesbeck Road, Interstate 74, and West Fork Road) consist of roadways, one (Huffman Court) a housing development, and one (Clough Pike) a valley slope with a house at the top. The failures at West Fork and Groesbeck Roads involve slopes beneath the roads whereas the failure at Interstate 74 site involves a cutslope above the road.

The Huffman Court and Groesbeck Road sites were mapped in detail, using the plane table method, to accurately delineate such physiographic features as scarp faces, fissures, and depressions with ponded water. Physiographic maps of the other sites could not be prepared because of the steep slopes, dense vegetation, and post failure alterations of the slide areas. In addition to mapping, the slope movement at Huffman Court and Groesbeck Road sites was monitored from June, 1990, to May, 1991. Both sites were staked at 10-foot (3 m) intervals from a fixed point assumed to be stable. At Huffman Court, the back of a house above the most recently developed scarp face was assumed to be the stable point whereas at Groesbeck Road a drilled concrete pier was taken as the stable feature. The distance the stakes moved with respect to the stable reference point was measured at approximately 6-week intervals throughout the year. The other study sites appeared to have either stabilized naturally or were remediated during the study period and, therefore, were not monitored for movement.

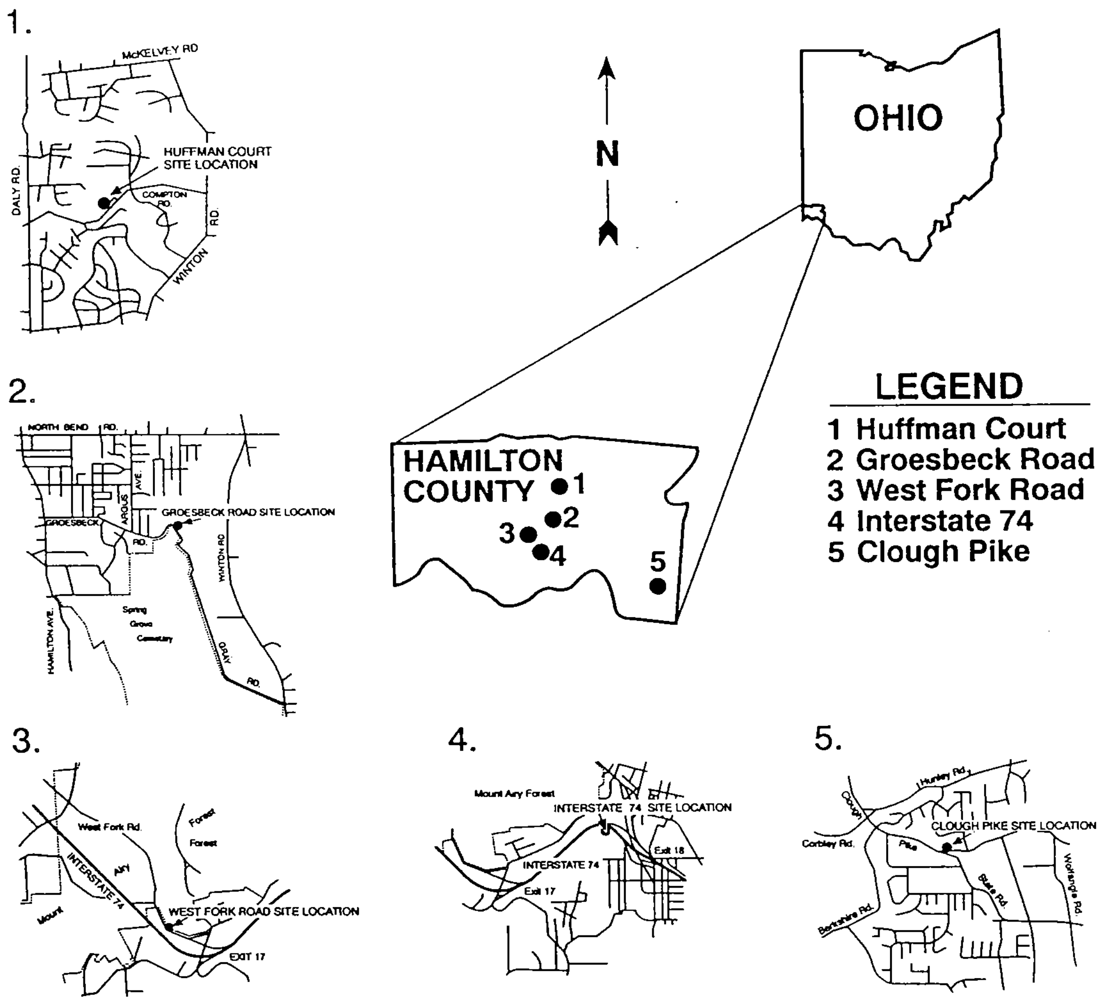


Figure 1: Location of the study sites.

Chunk samples of cohesive soils and bulk samples of granular soils were collected from different sites for lab testing. For sampling, the soils stratigraphy at each site was first established and representative samples were then obtained from each stratum. In addition to field observations and lab tests, drilling information from previous studies, when available, was used to establish subsurface conditions at some sites.

Laboratory Testing

Grain size distribution and Atterberg limits were determined for all samples in order to classify them according to the Unified Soil Classification System. The direct shear test and the blow count (N) values were used to obtain shear strength parameters for stability analysis. A consolidation test was performed on the base clay from the Clough Pike site to determine the amount of settlement of an in-place, concrete-block gravity retaining wall at that site. All tests were performed according to the standard procedures specified by the American Society for Testing and Materials (ASTM, 1991).

Stability Analysis

The REAME (Rotational Equilibrium Analysis of Multi-layered Embankments) program, developed by Huang (1991), was used to analyze the stability of slopes for both dry and wet conditions at the Huffman Court, West Fork Road, and Clough Pike sites. The program assumes a cylindrical surface of failure and uses the Simplified Bishop Method of Slices to determine the factor of safety. Data required for using this program include slope geometry, position of water table, soil stratigraphy, and values of density, cohesion, and friction for individual soil layers. Field observations, lab tests, and blow count values from previous drilling were used to procure these data.

DESCRIPTION AND ANALYSIS

Although five sites were investigated for this study, only three are discussed below; namely the Huffman Court site, the West Fork Road site, and the Clough Pike site. These three sites are representative of the slope stability problems in the Cincinnati area as well as the types of remedial measures used.

Huffman Court Site

The Huffman Court subdivision is located just south of the Cincinnati city limits in Mount Healthy Township (Figure 1). The subdivision was built in the winter of 1971-72 and has been plagued by slope stability problems since then. Two houses along the north side of the Huffman Court subdivision are reported to have been completely destroyed and numerous others have suffered minor to moderate damage. The greatest amount of movement is occurring in the backyards of the homes on the north side of the Huffman Court subdivision, especially the backyards of lots 1012, 1016, and 1020. The movement is manifested as a series of scarps and has undermined the patio of one of the houses (Figure 2). During the period of this study, the largest amount of movement occurred near the property line between lots 1016 and 1020 where the toe of the slope moved more than 15 feet (4.6 m). A plot of

Figure 2: Multiple scarps in the backyards of the houses along the north side of the Huffman Court subdivision. Notice the undermining of the patio due to soil movement.

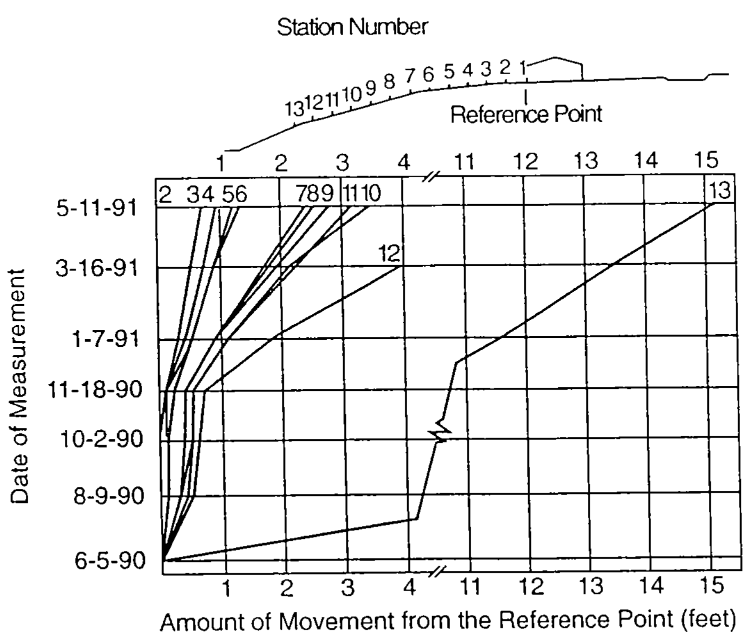


Figure 3: Plot of slope movement versus time for the Huffman Court site.

the movement versus time at this location is shown in Figure 3. The plot shows that the rate of movement is maximum near the toe area (up to 3 feet or 1 m per month) and decreases upslope. Because of this movement, portions of the property at these lots are being lost into a stream which flows at the base of the slope.

The soil profile at the Huffman Court site (Figure 4) consists of approximately 7 feet (2.1 m) of Illinoian till of variable composition, from clay to fine gravel, overlying a 16-foot (4.9 m) thick deposit of brown plastic clay (CH). The brown clay overlies a 2 to 3-foot (0.6-0.9 m) thick deposit of medium stiff, gray blue clay (CL). The bedrock at this site is the Miamitown Formation which consists of interbedded shale and limestone.

The houses have been built on a cut and fill slope which probably was marginally stable even before any development occurred. In some portions of the slope, up to 16 feet (4.5 m) of fill was placed to facilitate the building of the homes. The fill material consists primarily of a low plasticity clay (CL).

Figure 5 shows the results of stability analysis for the Huffman Court site. The soil properties used for stability analysis are given in Table 1. The factor of safety for the critical circle (Figure 5) is 2.4 which indicates a stable slope. This, obviously, is not the case as there is ample evidence of repeated movement affecting the actual slope. The contradictory results of stability analysis and field observations suggest that the failure at the Huffman Court site has not occurred along a single circular surface as assumed by the REAME program. Instead, the failure appears to be a combination of extension of the weak basal clay near the toe area and the block gliding of the overlying till and fill materials, as proposed by Gokce (1992). Extrusion of basal blue clay, in the form of bulges and flows, was clearly seen along the toe area during the course of this study. The stream flowing along the base of the slope at this site facilitates the extrusion mechanism through undercutting and softening of the basal clay. The gentle nature of the slope (< 15 degrees), the presence of numerous scarps, the shallow water table (3-5 feet, 1-1.5m), and the accelerated rate of movement in the toe area are some of the other factors that seem to favor extrusion and block gliding as the main mechanism of failure at the Huffman Court site.

Table 1. Soil properties used in the stability analysis of the Huffman Court site, assuming undrained conditions.

	Density* (pcf)	Cohesion* (pcf)	Friction Angle* (degrees)
Fill	130	2000	0
Till	125	4000	0
Brown Clay	115	900	0
Blue Clay	120	1100	0

* estimated from N values

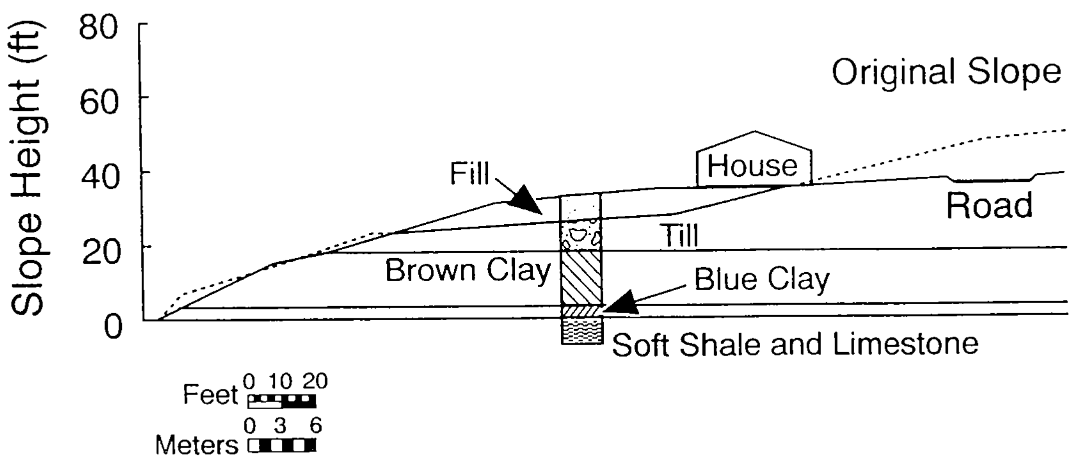


Figure 4: Slope geometry and soil profile at the Huffman Court site.

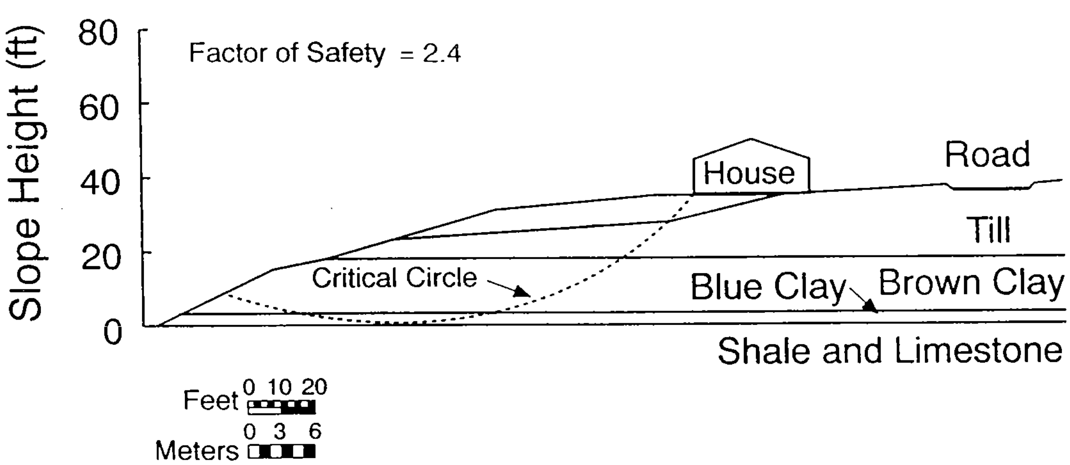


Figure 5: Results of stability analysis for the Huffman Court site.

A number of remedial measures have been suggested by Nutting (1977, 1979) to control future movement at the Huffman Court site. These include enclosing the stream at the toe of the slope in a box culvert, construction of a cantilevered drilled pier or tied-back retaining wall, regarding the slope and reinforcing the stream channel with riprap, and provision of proper drainage. A soldier beam type retaining structure near the toe area, with a gravel filter separated by a filter fabric, may be the most cost effective and feasible remedial measure at this site.

West Fork Road Site

The West Fork Road site is located on West Fork Road, approximately 0.2 miles west of its intersection with Montana Avenue (Figure 1). The failure (Figure 6) affected the west-bound lane on the inside of a curve in the road at a place where West Fork Creek flows along the toe of the slope. Initial failure of the road occurred about 15 years ago (Pohana, 1990). The failure was repaired and a curb was constructed to prevent surface water from flowing onto the slope which extends down to West Fork Creek. Recent movement began during the winter of 1990 following a period of heavy and prolonged rainfall. The movement continued through the summer of 1990 at a rate of 6 inches (18 cm) per month, resulting in a near vertical displacement of the pavement (Figure 6). The movement was rotational in nature and the main scarp was approximately 60 feet (18 m) long. Approximately 14,000 cubic yards (10,700 cubic meters) of material was involved in the failure. Temporary lateral support to the upper portion of the slide was provided in April, 1990, by driving I-beams (Figure 6) to a depth of 7 feet (2.1 m) below the road surface. The I-beams were placed at 3-foot (1 m) spacing and were connected by guardrail panels. Additional support, consisting of 3-inch (9 cm) steel pipe sections, was later added. The hollow left by the failure was backfilled and covered with asphalt. At least two more repairs were made as the movement continued (Pohana, 1990).

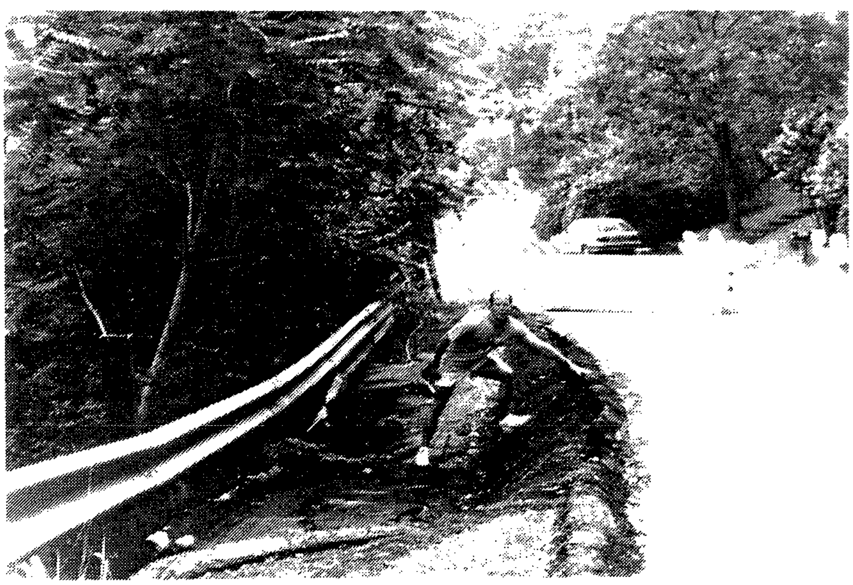
Figure 7 shows the soil stratigraphy at the West Fork Road site. The road was constructed on a fill of variable thickness (0-12 feet; 0-3.7 m) which overlies an approximately 10-foot (3 m) thick layer of colluvium. Below the colluvium is the bedrock of Kope Foundation. The fill consists of black cinders mixed with a silty clay, and trace sand and gravel. The colluvium consists of weathered shale with limestone fragments. Table 2 lists the engineering properties of soil material used for stability analysis.

Table 2. Soil properties used in the stability analysis of the West Fork Road site.

	Density* (pcf)	Cohesion* (psf)	Friction Angle* (degrees)
Fill	100	0	30
Colluvium	110	3000	0

* estimated from N values

(a)



(b)

Figure 6: View of the West Fork Road failure as it appeared on June 3 (a) and August 18 (b), 1990.

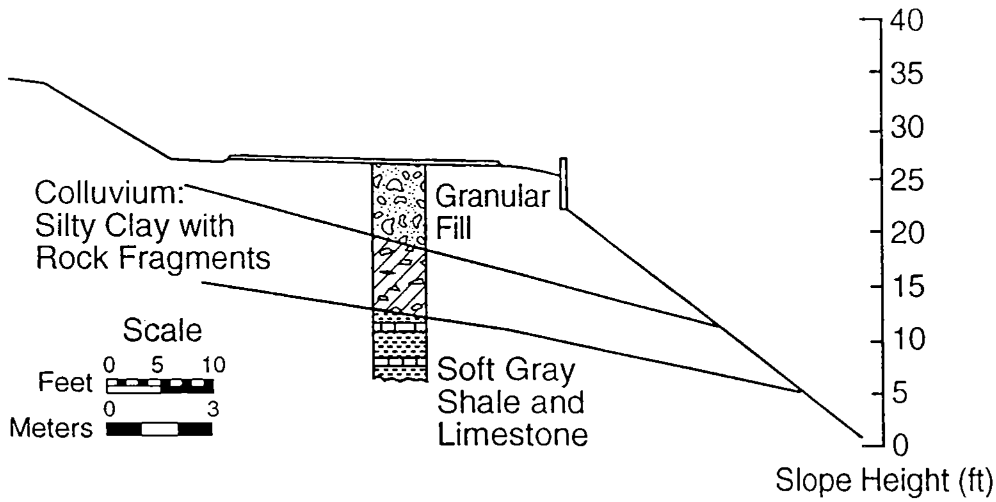


Figure 7: Slope geometry and soil profile at the West Fork Road site.

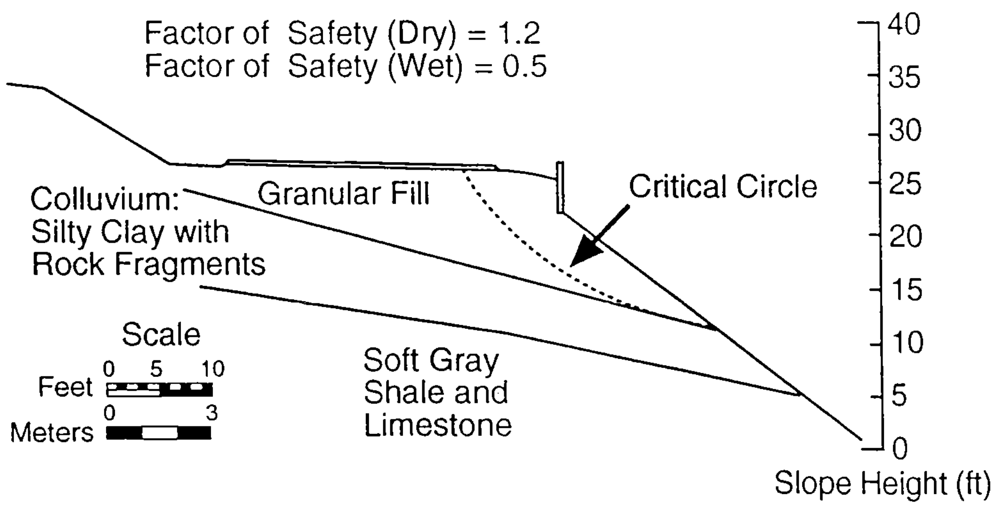


Figure 8: Results of stability analysis for the West Fork Road site.

Figure 8 shows the results of stability analysis. The critical circle is confined to the fill material and is tangent to the stronger colluvium below. The minimum factor of safety for the dry condition is 1.2 and that for the wet condition (completely saturated) 0.5. Thus, water must have played a major role in causing the failure at this site. Numerous water seeps, indicating a high water content and poor drainage, were observed on the slope. The unusually high rainfall in the spring of 1990, prior to failure, very nearly saturated the slope. Another factor that appears to have contributed to the failure is the undercutting of the bedrock toe by West Fork Creek. Weight and vibrations of traffic on West Fork Road probably aggravated the already unstable conditions at the time of failure.

The West Fork site was remediated during the fall of 1990, using a drilled-caisson retaining wall. The 2.5-foot (0.76 m) diameter concrete caissons were placed at 5-foot (1.5 m) centers. The caissons were drilled from the road level to a depth of 5 feet (1.5 m) below the stream level (Pohana, 1990). Six-inch (15 cm) thick concrete panels were used as lagging between the caissons above the bedrock surface. Drainage was provided at the base of the wall, using an 8-inch (24 cm) diameter perforated drain pipe surrounded by fine gravel. A filter fabric was used to separate gravel from the fine-grained fill material. Gabbions were placed at the base of the slope to prevent toe erosion by stream water. The final stage of remediation involved backfilling the wall and repaving the road. Figures 9 and 10 show the remediation work in progress and the finished retaining wall, respectively. Drilled-caisson type retaining walls are a frequently used remediation technique in the Cincinnati area.

Clough Pike Site

A large rotational landslide occurred on June 20, 1982, approximately 30 feet (9 m) behind the residence of Mr. Bart Hughes at 2173 Spinning Wheel Lane (Webb, 1982) (Figure 1). The upper scarp was about 200 feet (60 m) long and the slide, on the whole, involved 18,000 cubic yards (13,800 cubic meters) of material. The failure was rather rapid with the entire movement occurring in a period of a few minutes (Webb, 1982). A large quantity of compacted fill was placed at the base of the slope in the following year to reroute Clough Creek away from the toe of the slope (Webb, 1983). A few years later, a gravity retaining wall, consisting of large-size concrete blocks, was built to protect the fill from erosion by Clough Creek. Figure 11 shows the slope as it looked during the summer of 1990. The dense vegetation serves as a stabilizing agent and no recent movement, other than the usual creep or localized collapses of the retaining wall, has been observed.

The majority of the failed slope consists of outwash deposits from the Illinoian and Wisconsinan ice advances. The outwash deposits range in size from fine sand and gravel to coarse gravel, cobbles, and boulders. At the base of the slope lies an approximately 10-foot (3 m) thick layer of low-plasticity varved clay (CL). Figure 12 shows a generalized soil profile at the Clough Pike site. Table 3 contains the density values and strength parameters of various materials, as used in the stability analysis. The results of sieve analysis indicated that most layers of granular material contained a little clay which explains the steep nature of the slope at this site. Overall, the slope appears to be fairly well-drained.

Figure 9: Remediation work in progress at the West Fork Road site.

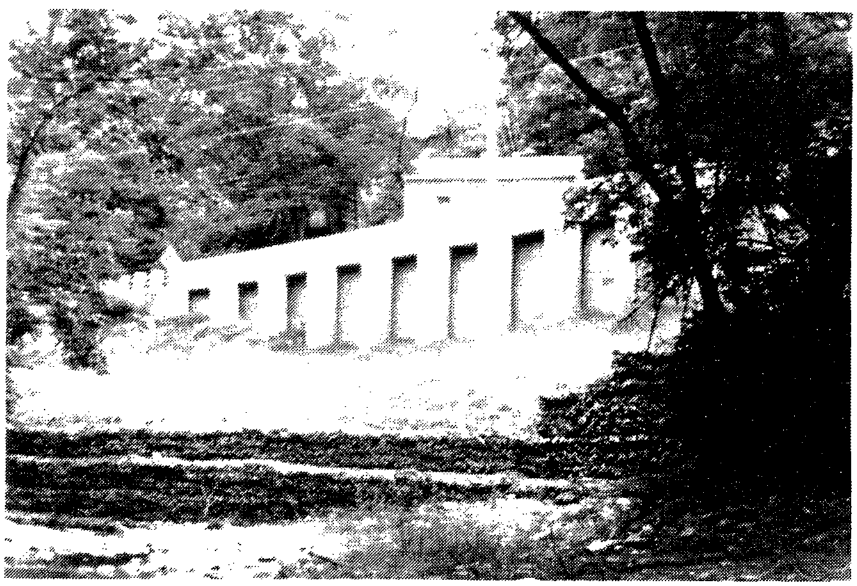


Figure 10: View of the fully remediated slope at the West fork Road site.

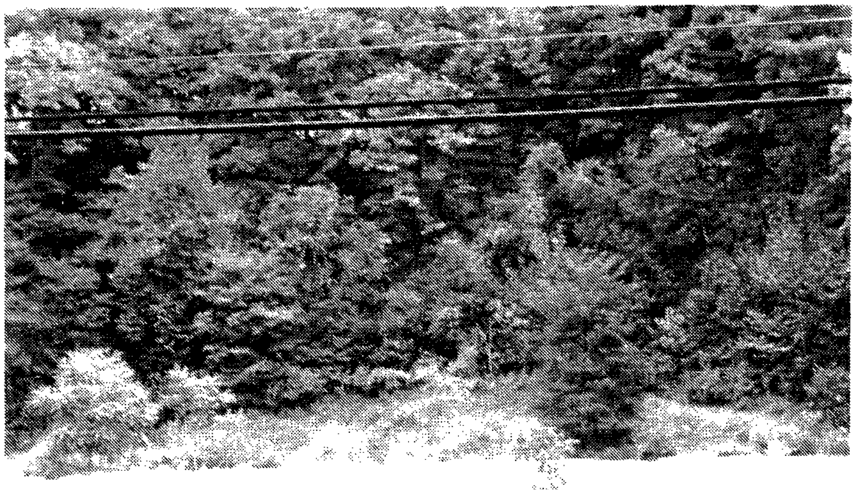


Figure 11: View of the tree-covered slope at the Clough Pike site from the intersection of Clough Pike and State Road.

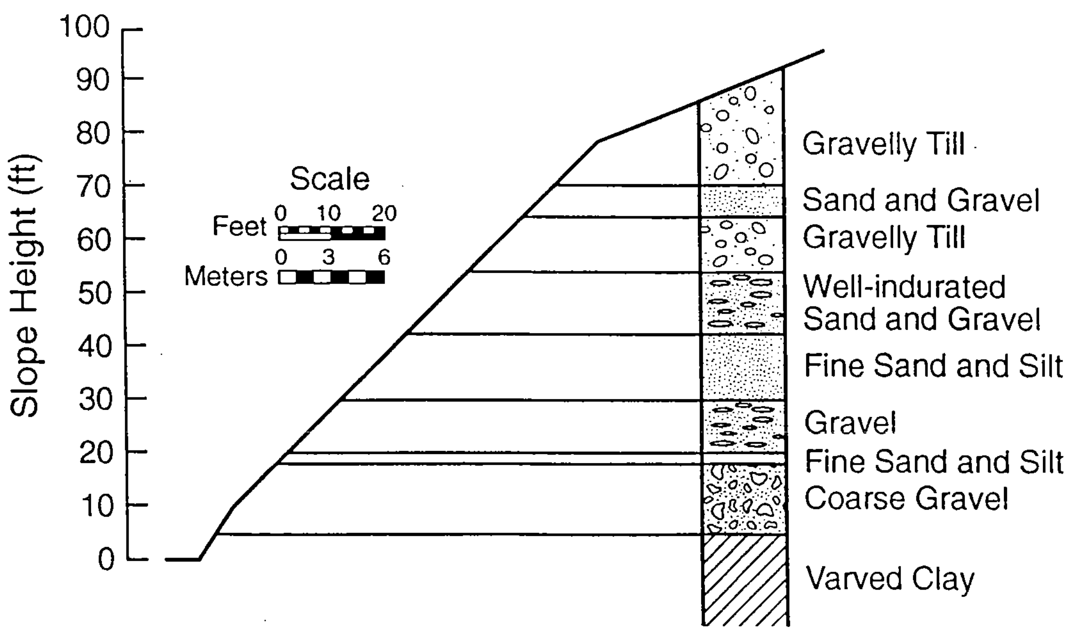


Figure 12: Slope geometry and soil profile at the Clough Pike site.

The results of stability analysis are shown in Figure 13. The critical circle is tangent to the underlying clay which serves as a firm base due to its high strength (Table 3). The minimum factor of safety for the dry condition is 1.1 and that for the wet condition, with the location of the water table as shown in Figure 13, is 0.85. The factor of safety for the completely saturated condition was not determined because it is highly unlikely that a slope like the one at Clough Pike would ever get saturated. A water table position slightly lower than the one shown in Figure 13 would have caused the failure. The failure at the Clough Pike site is probably due to a combination of pore pressure buildup at the time of failure and the undercutting of the slope toe by Clough Creek.

Table 3: Soil properties used in the stability analysis of the Clough Pike site (determined by direct shear test).

	Density (pcf)	Cohesion (psf)	Friction Angle (degrees)
Gravel	130	100	50
Fine Sand	110	36	37
Varved Clay	110	5112	31

The slope has been remediated through the placement of compacted fill at the base of the slope and erection of a gravity wall (Figure 14) to prevent toe erosion. The wall is 392 feet (119 m) long and consists of approximately 4-foot (1.2 m) square concrete blocks stacked one above another. The height of the wall varies from 10 feet (3 m) to 14.5 feet (4.4 m), with the lowest row of blocks partly buried in the clay.

A portion of the wall, about 40 feet (12 m) long, in the 14.5 foot (4.4 m) high section, failed during the spring of 1990. In order to determine the probable cause of this failure, a stability analysis of the wall was performed with respect to sliding, bearing capacity, settlement, and overturning. The analysis indicated that failure was not associated with sliding, low bearing capacity, or excessive settlement. The factor of safety with respect to overturning, however, was calculated to be 1.1 for the dry state and 0.3 for the saturated state. Thus, a small buildup of pore pressure behind the retaining wall could have caused the failure. Also, localized undercutting by Clough Creek may have contributed to the failure. The wall has since been repaired and an additional row of blocks has been placed along the base (Figure 14). It is likely that the wall may fail again.

ENVIRONMENTAL AND ECONOMIC IMPACTS

The Cincinnati area is highly susceptible to landsliding because of its steep topography, weak bedrock, abundance of fine-grained glacial deposits, and humid climate. Most of the damaging landslides occur where humans have done something to alter the delicate natural balance of the slopes. All of the failures investigated, except the Clough Pike site, involved human activity as an important factor in promoting failure. There are,

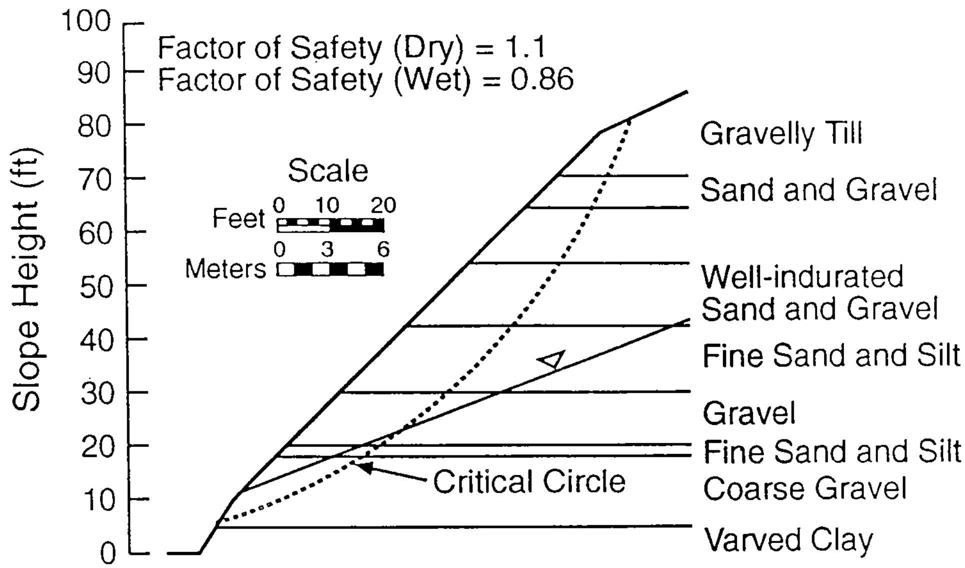


Figure 13: Results of stability analysis for the Clough Pike site.

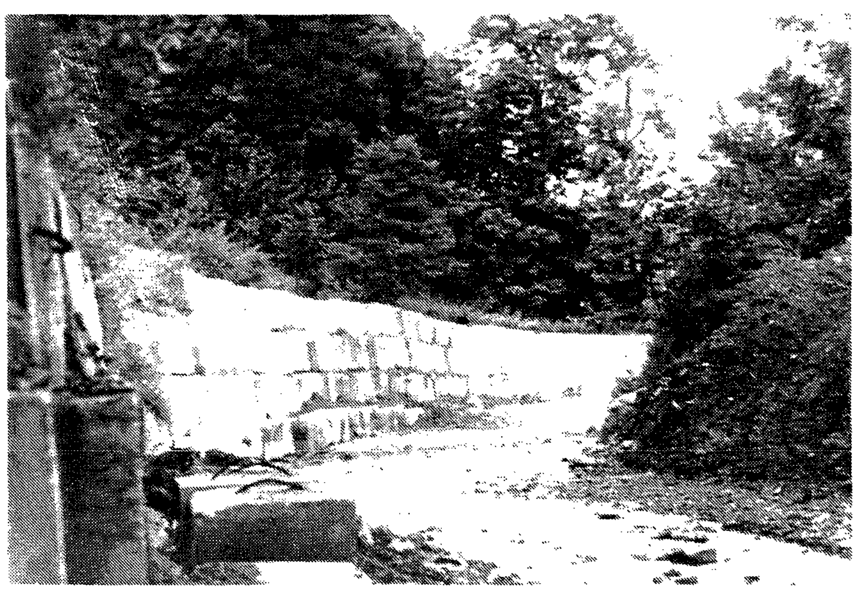


Figure 14: Concrete-block gravity retaining wall at the base of the slope at the Clough Pike site.

however, a number of large ancient slides in the area which are not a result of human-induced changes. Most of these slides involve the movement of large masses of colluvium over the bedrock surface. Although these slides are not originally the result of human activity, human alteration of slopes may upset the natural state of equilibrium which often reactivates these slides.

The exact monetary cost of the five landslides studied is not known, although it must be high. At the Huffman Court site two houses have already been completely destroyed, and numerous others have been moderately damaged. The slides have reduced the property values of the houses which remain far below their original prices. The east ends of the road and the sidewalk through the development have been repaired a number of times. Utility lines break on a regular basis which requires crews to come out and make emergency repairs. It is impossible to assess the painful mental toll the landsliding has had on the people who have watched their property values drop and their homes slowly break apart. Nutting (1979) has estimated that it would cost between $700,000 and $1,000,000 to fully remediate the failures occurring at the Huffman Court site. The Groesbeck Road slope failure was small in size but the remedial measures involved exceeded $185,000 (Nyberg, 1992). Major repairs were necessary because the stability of the road was threatened. West Fork Road was repaired numerous times prior to its permanent remediation using the drilled caissons. This failure, like the Groesbeck Road failure, affected the stability of the road and, therefore, required extensive repairs. The remediation of this failure ultimately cost the Cincinnati taxpayers approximately $220,000 (Nyberg, 1992). The Clough Pike failure caused an immediate reduction in the value of the house at the top of the slope. The large gravity retaining wall at the base of the slope, initially costing approximately $30,000 and requiring periodic maintenance, is an expense born by the taxpayers of Hamilton County (Webb, 1990).

Since the Cincinnati area is highly susceptible to slope movements, it is recommended that before the construction of a new structure, a detailed geotechnical evaluation of the site should be conducted. If possible, sites with glacial lake clays and steep slopes should be avoided. Cincinnati has no laws on the books at the present time which regulate the building on potentially unstable ground unless it involves the cutting or filling of the land in order to change the grade.

SUMMARY

The findings of this research can be summarized as follows:

1) The topography of the cincinnati area is characterized by deeply entrenched streams resulting in numerous steep slopes. The bedrock of the area consists of easily erodible claystones and shales interbedded with thin layers of limestone. As a result, the slopes are covered with thick layers of colluvial deposits and residual soils. Furthermore, fine-grained glacial deposits are quite abundant in the area. The combination of steep slopes, easily erodible bedrock, humid climate, and human activity has resulted in a high frequency of landslide incidents in the Cincinnati area.

2) Among the five failures investigated, two are rotational slumps (West Fork Road, and Clough Pike), one a combination of a rotational slump and an earthflow (Interstate 74), one a debris slide across the bedrock surface (Groesbeck Road), and one (Huffman Court) a combination of extrusion and block gliding.

3) The most common type of remedial measure used in the Cincinnati area to combat landsliding is the drilled-caisson type retaining wall, often with lagging between the caissons. The second most common type of remedial measure, and the one most often used by individual homeowners and businesses, is the gravity retaining wall.

4) Human activity appears to have contributed to all slope failures studied except for the slope at the Clough Pike site which failed due to erosion of the toe and buildup of pore pressure.

5) The Cincinnati area has the highest per capita cost of landslide damages in the United States.

6) There are currently no landslide susceptibility maps available which could be used as a basis for developing regulatory statues for new construction in areas of high landslide susceptibility.

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EROSION AT TIDAL INLET BRIDGES: ON THE CURRENT STATE OF ANALYSIS

By Mark A. Ross[1] and Mark S. Vincent[2]

Abstract
Scour in the vicinity of tidal inlet bridge structures is a combination of local, contraction and general scour. These processes can operate on varying temporal and spatial scales, often resulting in the loss of adequate structural bearing capacity and failure. Recent bridge investigations have revealed that many tidal inlet bridges have experienced severe undermining of support piers, piles and spread footers. A review of available scour screening and analysis methods is presented. Methodologies for tidal hydrodynamics are contrasted with riverine analysis. For inlets where aggressive long term general and contraction scour processes are dominant, analysis using a two dimensional linked hydrodynamic and sediment dynamic Scour model is advocated.

1.0 INTRODUCTION

The Federal Highway Administration (FHWA) estimates that at least 20% of the nation's 483,000 bridges are susceptible to scour. In coastal states like Florida, many of these bridges are located in tidally controlled inlets, estuaries and bays, and experience significantly higher rates of scour susceptibility. The complex hydrodynamics of these systems, involving reversing tidal flows, wave contributions, littoral drift and wind setup, render many of the standard riverine approaches to bridge scour prediction inappropriate and inadequate. This paper provides a cursory overview of the principle scour processes and current approaches to analysis.

2.0 SCOUR CATEGORIES

Erosion of sediments in the vicinity of tidal inlet bridges is a result of the superposition of several processes. These scour mechanisms which can operate on a variety of spatial and temporal scales, include: degradation and aggradation (general scour); contraction scour; and local scour (Raudkivi, 1990; Simons and Senturk, 1977). Following, is brief discussion of each of these processes as they relate to tidal inlet facilities.

2.1 Aggradation/Degradation

Aggradation/degradation, or frequently referred to as general scour, is the key operative process which governs the large scale morphology of an inlet, independent of any bridge structures. For a tidal inlet, aggradation/degradation will be primarily a function of inlet, shoreline, bay and sediment characteristics. Energy sources include short and long period waves (tides) and storm

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seiching. During ebbing or flooding of the tides, if the resulting velocities exceed the conditions for incipient motion of sediment, erosion may occur. Additional conditions which compound aggradation/degradation in tidal inlets include: reversing flows; additional shear stress imparted from short period waves; longshore drift budget; and openings and closures of adjacent inlets.

2.2 Contraction Scour

Often, natural or more frequently anthropogenic structures are present within an inlet, which constrict the effective cross sectional area of the flow. From elementary hydraulics, a reduced cross sectional flow area will necessitate an increase in flow velocity in order to convey the tidal volume. The accelerated velocity will aggravate erosion by increasing both the frequency and magnitude of the critical shear stress exceedance. Sediment will erode until sufficient bed deepening occurs to achieve equilibrium. Large bascules, crutch bents and pile groups are the prime agents driving this form of scour. To a lesser degree, the conglomeration of associated structures in an inlet, such as docks, marinas, and bank fill, will also contribute to contraction scour. A notable example of significant contraction scour is the Johns Pass Bridge located in Pinellas County, Florida. This bridge which was constructed along a new alignment in 1974, is supported by four 10 meter x 5 meter bascules as well as numerous smaller piers and crutch bents. Since its construction, the bed beneath the bridge has experienced over 6.3 meters of erosion, and undermining of several pier supports. Currently, erosion rates of up to 0.5 meter per year have been recorded. As the piers have been undermined, the Florida Department of Transportation has installed larger diameter crutch bents for support. Unfortunately, these larger supports have further aggravated contraction scour, thus causing additional adverse erosion.

2.3 Local Scour

Scour also ensues as a result of the complex three dimensional flows imposed by pilings or other structures. Local scour can initiate when the approach velocity or shear stress is less than one half of the threshold value for incipient motion of sediment transport. The principle physical components of local scour include downward flow at the upstream pier face, cast off vortices and wake, and a horseshoe shaped vortex. Many researchers have attempted to numerically describe the relationships between the important parameters of pier width, depth of flow, approach velocity, sediment size and sorting, pier shape, angle of attack and the resulting local depth to scour. However, as noted by Raudkivi (1990), these various approaches have dissimilar appearances and even more disparate results. The key point here is that the value of calculated local scour must be superimposed to the general and contraction scour to ascertain structural integrity.

3.0 Tidal Bridge Analysis

For ongoing FHWA required scour investigations, a four phased design approach is advocated. This is qualitatively depicted in the accompanying figure. Basically, Phase I is a qualitative assessment to determine whether there is a scour problem. Phase II is a quantitative assessment to determine the scour depths. Phase III involves evaluating structural and geotechnical alternatives, and Phase IV is to formulate and implement recommended remedial action(s).

3.1 Rational Analysis

For Phase II and III efforts, the bridge design and maintenance engineer is faced with the difficult task of predicting scour trends for various structural or design event scenarios. Ramifications of this analysis extend from unpredicted bridge failure to advocating expensive bridge remediation or replacement. Presently, a variety of analyses are available, each with various degrees of complexity and or limitations. These approaches, which include rational hand calculations, 1-Dimensional modeling and 2-Dimensional modeling are briefly discussed below.

First, most tidal bridges categorized as scour susceptible in the Phase I screening require additional data gathering. In a few cases, this is sufficient to characterize the magnitude of the problem. In most cases, however, this would be followed by rational hand calculations using accepted methods to estimate storm velocity magnitudes and durations and scour magnitudes following newly revised HEC-18 and HEC-20 guidelines. In severe cases, a more detailed modeling study would need to be performed.

It should be emphasized that the FHWA HEC-18, HEC-20 recommended empirical equations are order-of-magnitude and extremely conservative estimates only (even more so when extended to tidal applications). However, these crude, first approach, conservative estimates applied with the design event conditions provide, together with known or estimated pile conditions (depth and bearing), a valuable indication of the severity of the problem at the site. Because they are quickly and easily applied, they can be performed as a preliminary approach for phase II investigations at nominal costs.

The problem with the FHWA recommended guidelines for Phase II analysis for tidal bridges is that it is still oriented towards 100 year and 500 year design storm event response. This is very limited and should only be one element of the design/evaluation process. This is because the scour problem at tidal bridges in Florida is most commonly associated with daily or seasonal dynamics, i.e., longer term response (Vincent and Ross, 1992; Vincent, 1992). The scour problems experienced during major return events are only compounding or aggravating conditions that can end up being the final catalyst for the potentially critical condition. This is observed in bridge inspection reports (bi-annual cross-sections) for most Florida bridges in high energy coastal environments.

An approach advocated herein is to ascertain the cause of the scour problem at a structure by first determining the hydrodynamic field at and adjacent to the structure under daily, seasonal-average, and annual (or greater) event conditions. Next the bottom shear conditions and sediment supply are compared with the critical tractive forces required for incipient motion together with all stabilizing conditions of the bed material at the structure. The first requirement can be achieved with the Dynlet1 model but this is not the procedure recommended by FHWA. As advocated by FHWA, the Dynlet1 model is run to ascertain maximum velocities during the design storm and these are used with riverine scour equations to quantify scour magnitudes. Because these equations were largely derived from flume studies for ultimate scour depths, these results are unrepresentative in the tidal application and extremely conservative.

3.2 USF/CMHAS Scour 2-D Model

A severe limitation of the above approaches is their lack of treatment of spatial and temporal variation in tidal scour. This omission is potentially fatal in systems where the dominant scour is driven by long term degradation or contraction. An analysis tool which allows for the dynamic linking of inlet hydrodynamic and sediment dynamic processes is the Scour Model developed at the University of South Florida, Center for Modeling Hydrologic and Aquatic Systems (USF/CMHAS). The Scour model is a two-dimensional hydrodynamic model that has been dynamically linked to an internal sediment dynamic model. Salient features of this model are its ability to simulate tides, waves, winds, inlet geometry, bridge structures and bed armoring. To date the Scour Model has been employed in several tidal inlet engineering investigations along littoral drift shorelines of the West Coast of Florida. Ross (1990) employed the prototype version of the model for a bridge and channel design study for Clearwater Pass. The Johns Pass Bridge maintenance/replacement study was performed using a modified, moveable bed version of the model (Vincent et al., 1992). The Johns Pass investigation provided annual scour/deposition rates for seven structural alternatives. This investigation indicated that contraction scour from the bridge was a primary factor in the historical erosion of up to 6.3 meters of sediment, which had undermined several bridge pier foundations and piles. In addition, the model simulations indicated that past remediation measures such as the installation of additional flow constricting crutch bents, further aggravated scour rates.

4.0 NEEDS AND FUTURE DIRECTIONS

In summary, the recommended course of action for phase II investigations involving tidal bridges is: 1) Determine which bridges require phase II analysis (perhaps all recommended from phase I efforts) and for what reason and priority (i.e., to meet FHWA requirements or to more quantitatively evaluate remedial alternatives prior to an immediate course of action); 2) Determine which of these require an additional field investigation and to what level; 3) Conduct the appropriate field investigations and supplemental data collection (cost savings can be achieved by considering multiple bridges at once); 4) Perform simple (rational) hand calculations to qualitatively evaluate the magnitude of the problem at the structure; 5) If there is sufficient need because of the magnitude of the problem or suggested remedial action, perform a model study on the site; 6) Furnish a report on the results, recommending remedial action alternatives that will go forward to phase III analysis involving geotechnical and structural considerations.

As reviewed here, a variety of measures are available for evaluating tidal inlet bridge scour. The hand calculations and one-dimensional modeling efforts are suitable for use in preliminary screening exercises, however, are deficient for purposes of evaluating long term and spatially variable scour. For these needs a detailed sediment transport model such as the USF/CMHAS Scour model is the preferred alternative. Although the Scour model has been successfully employed on several inlet programs, it is still undergoing an active research and development program. Various aspects of enhancement such as improved algorithms for sediment advection, sediment sorting, and bed armoring are warranted.

A present limitation affecting the advancement of this technology, is the surprising deficit of accurate hydraulic and bathymetric data of the inlets investigated. The present data is insufficient to fully calibrate and validate any model. Additional data collection, simulations and analysis will be needed in this regard. Therefore, only qualitative and comparative results can be obtained at this time.

Although considerable theoretical and modeling improvements are warranted, the USF/CMHAS Scour model shows merits as a tool to be used in management and design studies, as well as a vehicle for understanding basic processes and responses of tidal scour.

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Tidal Scour Evaluation

Collection & Review of Available Data

Including As-Builts, Design Plans,
Bridge Inspection Reports,
Historical Aerials & Charts
Recent Bathymetry,
FEMA/NOAA/COE; Wind, Wave, Storm Surge Data
Preliminary Site Visit

↓

Field Investigation

Including Bathymetry,
Sediment Size & Distribution,
Tidal Range & Phase Lag,
Current Measurements

↓

Analytical Screening

Critical Velocity, Shear Stress,
Incipient Conditions
Equilibrium Depth
Tidal Prism/Cross Section Ratio

↓

Application of DYNLET1 or USF Scour Model

Evaluate Scour Cause & Severity
Evaluate Potential for Continued Scour
Recommendation and Preliminary Evaluation
of Remedial Alternatives

↓

Geotechnical & Structural Assessment

Evaluate Alternatives
Pile Capacity Foundation Analysis

↓

Formulate Reccommendations

Remedial Action
Countermeasures Design

44th ANNUAL FIELD TRIP GUIDE
UNIQUE CONSTRUCTION PROJECTS AND PROBLEMS IN THE TAMPA BAY AREA OF FLORIDA

by R. E. Goddard

INTRODUCTION

This guide is prepared for the 44th Annual Highway Geology Symposium's one-day field trip in the Tampa Bay Area, Florida, May 20, 1993. A few select geotechnical problems are presented with at least one practical solution. Three main areas of concern during highway construction in Florida are: weather, foundation stability, and disposition of used construction materials.

Indirectly, weather was a contributing factor to the collapse of the southbound span of the old Sunshine Skyway, the largest bridge in Florida. This bridge was originally designed to link the rapidly growing areas of St. Petersburg and Bradenton-Sarasota. During a heavy rainstorm, May 9, 1980, a ship struck the main pier of the western (southbound) span on an approach through the channel. A section of the bridge roadway was broken away and collapsed into the bay (Figure 1).

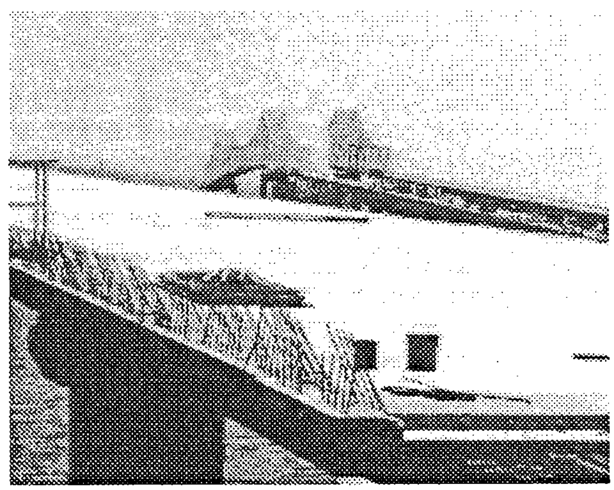


Figure 1. Old Sunshine Skyway spans (background). New Skyway construction (foreground).

A bus and several cars plummeted 150 feet to the water. The eastern span was undamaged and continued to be used for traffic in both directions. An immediate study was initiated to determine the best solution for keeping this vital roadway link open. Some of the possibilities considered were rebuilding the damaged span, building a tunnel under the bay, or building a completely new structure. A tunnel was an attractive option in that no additional surface protection would be needed. The one overriding factor became safety in heavy winds like those in a hurricane which might drive a ship into any structure and repeat what had occurred once before. The tunnel was ruled out due to environmental and cost concerns. The decision was made to build a new structure with four increased width lanes (two south, two north) and the addition of a breakdown lane. Measures were also designed in to protect the structure from errant shipping. Another weather concern was for lightning. The Tampa Bay area has been called the lightning capitol of the world. A lone structure of this height in the bay would also have to withstand numerous lightning strikes.

Florida's net population growth in the latter 1980's was 18 people an hour. It is now the fourth largest state by population. Along with this growth has come increased concern for the state's natural resources (potable water, forests, minerals, wildlife, and wetlands). Disposal of waste has become a very highly regulated process. In that central Florida has been one of the world's largest suppliers of phosphate, the byproduct from the beneficiation process is gypsum, which are stored in mounds called "Gyp stacks" or "Gypsum Stacks." The gypsum is slightly radioactive and no plausible use has been found. The Florida DOT has investigated its use as a base material and filler, but did not recommend its use. Construction materials in Florida may only be disposed of in designated landfills. Many of these construction materials such as concrete are just as inert as the surrounding materials in the ground. Instead of filling these landfills with useable materials, many of these materials can and must be recycled and reused, just as aluminum, newspapers, plastic, and glass are being recycled in the population center of the United States. When 30 miles of a six lane concrete roadway (part of Interstate 75) around Sarasota started to breakup after only two years of use, a use for the broken up concrete had to be found.

In Florida subsurface stabilization is a constant concern. With 90 percent of Florida underlain by limestone, the occurrence of sinkholes in most areas is a real possibility.

SUNSHINE SKYWAY - Structure "A"

At the north end of the old Sunshine Skyway, a structure was needed to allow passage of pleasure craft to eliminate unecessary traffic within the main channel. A draw bridge was installed. Currently, that draw bridge is being replaced with a hump bridge structure. This will still allow passage of pleasure craft and not hinder the greatly increased traffic load from the original installation of the drawbridge.

The granular embankment material in the approaches is protected by a layering of riprap material and bedding stone underlain by filter fabric (Figure 2). The riprap material comes from just south of Englewood, Florida and Ft. Myers, Florida. The bedding stone material comes from Brooksville, Florida. Existing rubble riprap lining the shoreline is from Punta Gorda, Florida. The riprap is fossiliferous limestone which has been recrystallized and cemented together.

The tour of structure "A" starts on the southern approach and walking north to under the bridge.

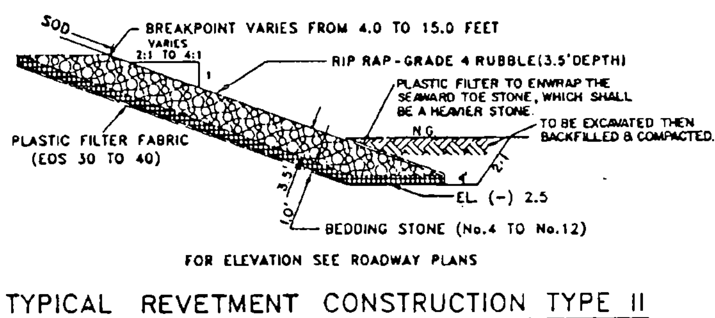


Figure 2. Cross-Section of Slope Protection.

SUNSHINE SKYWAY - Main Structure

The main span section is a segmental cable-stayed bridge. Each segment is added and tied to the previous segment by tensioning cables. Each individual segment includes four travel lanes, breakdown lanes, median, and necessary structural support (Figure 3). These segments are post-tensioned and adjusted (Figure 4). Certain segments are then attached to the main piers by cable for stability.

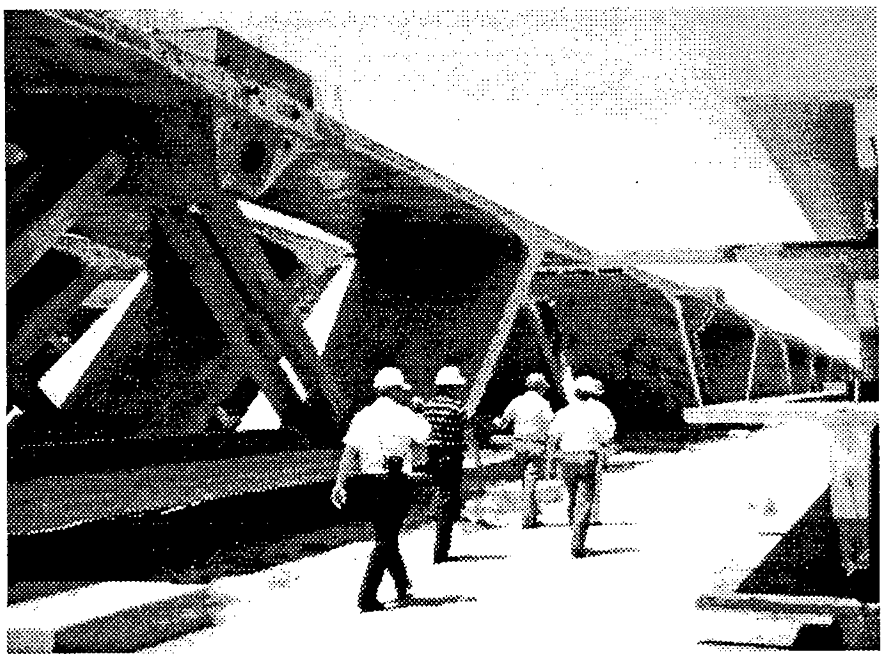


Figure 3. Main span segments at precast concrete yard.

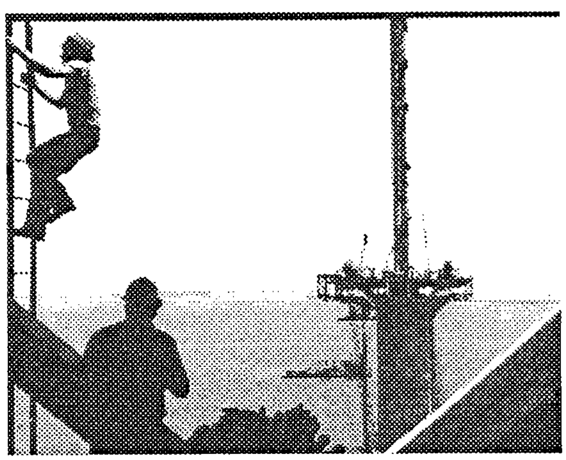


Figure 4. Construction on the southern main span pier from an added segment on the north main pier.

Interstate 75 Repair and Rebuilding -

In the mid-1980's construction began on extending Interstate 75 from from just above Tampa to bypass Tampa on the east side and continue down to one of the United States most rapidly growing areas: Florida's southwest coast. Various types of construction were employed throughout this approximately 120 mile extension. A big political and engineering concern was the use of asphalt or concrete in the main roadway section. Concrete and asphalt were both used for different sections. Concrete was used for the majority of the Tampa section. Then just below Tampa at the US301 exit asphalt was used until about the southern intersection of Interstate 275. From there until about the first Venice exit, concrete was again used. From there south asphalt was used. All of these sections were not without there own unique problems. The two asphalt sections around Ruskin, Florida and Venice, Florida had to be resurfaced shortly after they was opened to traffic as the friction course layer broke up and came loose. This was due to the design of the roadway and construction timing in surrounding sections. The friction course (top layer) was not compacted or rolled to the final desired density as the anticipated traffic would more than accomplish density. The traffic, however, was not introduced until much later than expected. Top layer aggregate was broken out from the brittle hardened liquid fraction instead of being packed and incorporated. This created two types of driving hazards. First, the aggregate was projected into traffic, cracking many windshields and denting cars. Second, the rutting of pavement by the removal of the surface friction course made changing lanes difficult.

In the Tampa area shortly after opening the Interstate 75 concrete roadway to traffic, a sinkhole caused a depression of approximately ten feet wide and a few inches deep in the travel lane northbound. An engineering resolution to the problem stabilized the depression and no more subsidence has occurred.

The concrete section around Sarasota of approximately 31 miles was open to traffic just about a year when cracking of the throughout the whole section became a problem. Cracks in the 10 inches of concrete were up to four inches wide and 60 feet long at the surface in the middle of three lanes. The stability of the concrete slabs was becoming a safety problem. The decision was made that the roadway would have to be redone.

Reprocessing Site -

The concrete from the defective Interstate 75 would occupy an enormous amount of special construction material landfill. On the worse section, material was not allowed to be reused in that same section, however, in subsequent sections the concrete is being reused as an aggregate component in asphalt. This is done in the same manner as you would crush and size rock for the same purpose. The reprocessing site is then treated as a mine and has to be approved as a source of aggregate. This was done previously on a section of Interstate 10 in the panhandle with success. The material is broken up at the roadway and shipped to the reprocessing site. The rock is stockpiled and then fed into the processing system. Before going into a crusher, the rock is scanned with a series of magnets to eliminate any stray iron or steel material. The primary crusher reduces the material in preparation for the secondary crusher. After the secondary crusher reduces the sizes of the particles, the material is sized and combined to give the desired gradation to agree with the design mix. After washing, the material is stockpiled and used the adjacent asphalt plant. This process is beneficial to both the State of Florida and the contractor.

SINKHOLES

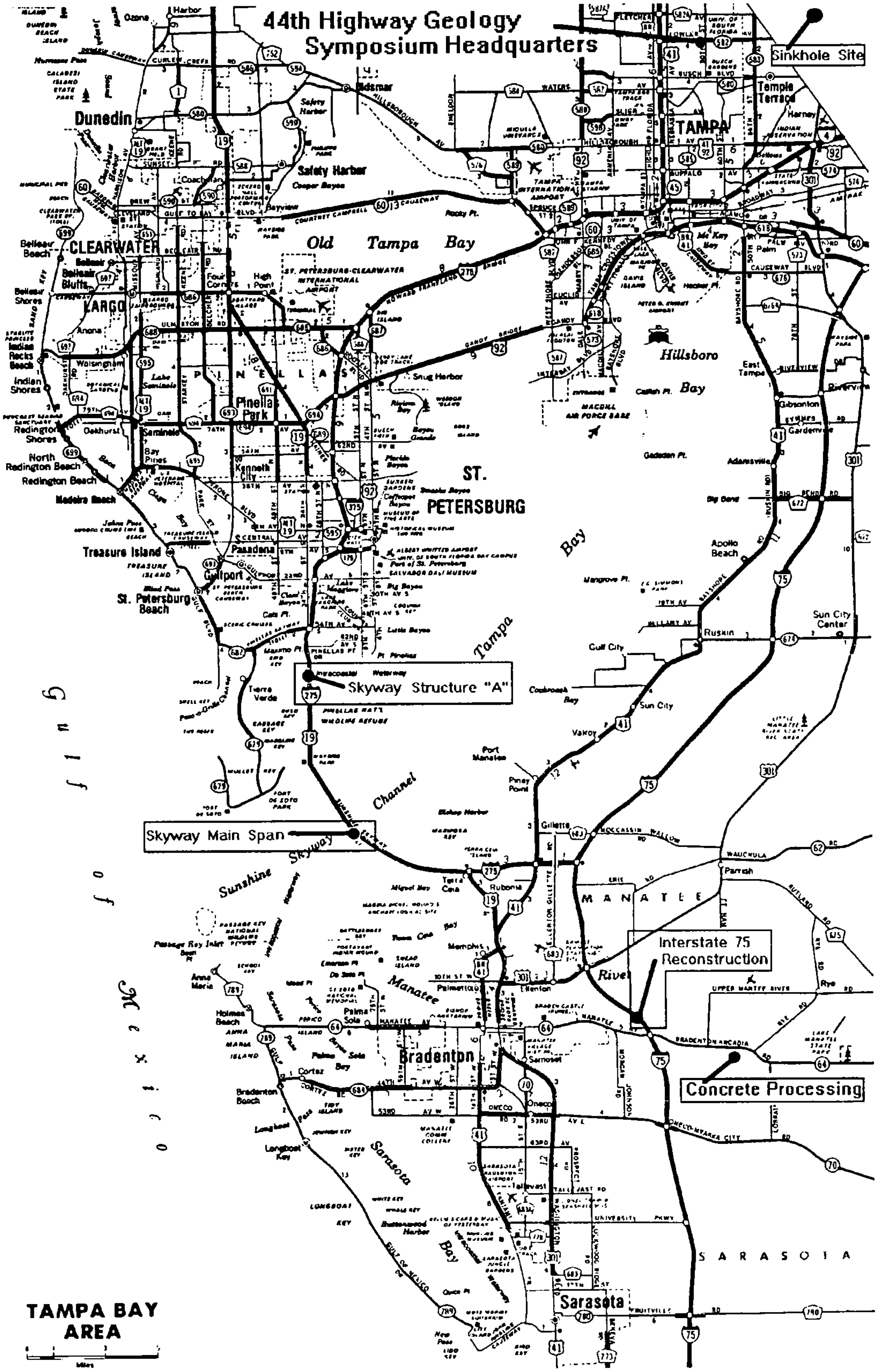
A major concern for the construction and maintainence of roads in Florida are sinkholes. The vast majority of Florida is underlain by limestone. This limestone contains the potable water supply on which Florida depends on nearly all the drinking water. It is solution riddled and occasionally a catastrophic collapse occurs. These sinkholes represent a construction problem in that part of the corridor study should include sinkhole potential and covered up or filled in sinkholes should be identified. In roadway maintenance, a subsidence of a few inches on a highway can create severe problems. Not only is this a driving hazard, but the remediation is very hazardous. In many cases the remediation is very expensive and is ineffective over time.

44th ANNUAL FIELD TRIP GUIDE

INTINERARY

MAY 20, 1993

Time	Function	Location	Mileage
0800	Depart	Holiday Inn, Fowler Ave., Tampa, Florida	0
0900	Stop 1	Sunshine Skyway Structure "A" Sunshine Skyway Main Span	37.5 42.1
1045	Stop 2	Sunshine Skyway Main Span Sunshine Skyway Structure "A"	
1230	Lunch	Sunshine Skyway South Rest Area	45.6
1400	Depart	South Rest Area	
1430	Stop 3	Concrete Recycling Plant	64.2
1515	Depart	Concrete Recycling Plant	
1530	Stop 4	Rest Area Stop on Interstate 75	85.7
1615	Depart	Rest Area	
1645	Stop 5	Sinkhole at Fletcher Ave.	115.1
1715	Depart	Sinkhole at Fletcher Ave.	
1730	End	Arrive Holiday Inn, Fowler Ave.	120.9


44th Highway Geology Symposium Headquarters
Sinkhole Site
Skyway Structure "A"
Skyway Main Span
Interstate 75 Reconstruction
Concrete Processing
Dunedin
CLEARWATER
TAMPA
Temple Terrace
Old Tampa Bay
Hillsboro Bay
ST. PETERSBURG
Tampa Bay
Gulf of Mexico
Bradenton
Sarasota
MANATEE
SARASOTA
TAMPA BAY AREA

MECHANICAL PROPERTIES OF LUNAR SOILS AND IMPLICATIONS FOR A LUNAR BASE

by

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ABSTRACT

Although the surficial lunar soil is loose and fluffy, the density increases very rapidly with depth. Just to 5 to 10 cm below the surface, the soil density is much greater than can be accounted for by the weight of the overlying soil; denser, in fact, than could be achieved with heavy compaction equipment on Earth. This phenomenon is due to meteorite impacts, which stir and loosen the surface, and at the same time, shake and densify the underlying soil.

As a result, the lunar soil is quite strong and is capable of supporting virtually any conceivable structure.

Trafficability on the lunar surface is well-understood: The energy consumed by the rolling resistance of the soil is small compared to other energy losses, most notably inertia. The cruising speed of a lunar roving vehicle is limited by surface roughness and terrain to about 6 to 7 km/hr. Slope-climbing of a wheeled-vehicle is limited to 23^{o} without grousers; and to about 30^{o} with grousers.

Excavation on the lunar surface, whether for mining or habitat installation, will require mechanical equipment. Vertical faces in the lunar soil can be excavated to a depth of 2 to 3 m. Once excavated, the lunar soil cannot be re-compacted to its original *in situ* density and, consequently, it will occupy about 15% more volume.

GEOTECHNICAL CONCLUSIONS

1. Meteorite impact is the major geologic process operating on the Moon, whereas the familiar terrestrial processes are absent: glaciation, running water, wind, and chemical decomposition.

2. The soil consists of bits of basaltic rock, plagioclase minerals, and as much as 50% glass. The meteorites impact the lunar surface at velocities of 2 to 20 km/s; the blast effect breaks and melts the rock, thereby creating soil.

3. The soil extends at least 2 m deep, and perhaps 20 m or more, virtually everywhere on the Moon's surface. There are occasional big boulders, but "bedrock" is not exposed at the surface.

4. The soil is classified as a silty sand, with roughly half of the material finer than a No. 200 sieve (74 microns: silt-size). There has never been an atmosphere or water on the Moon and, hence, no chemical weathering. As a result, there are clay-*sized* particles, but no clay *minerals*.

5. The particle size distribution is relatively constant from place to place: As soon as big pieces are broken into little pieces, the little pieces are welded back together by molten glass to form big pieces.

6. The density of the individual soil particles, or specific gravity, averages 3.1 (compared to 2.7 for most terrestrial soils). The high value is due to the presence of iron and titanium. However, some of the bits of glass are in the form of hollow beads, and their density is the same as water: 1.0

7. Whereas the *in situ* soil on the lunar surface is loose and fluffy, it becomes very dense just 5 to 10 cm deep. The density of the deeper soil is much greater than can be accounted for by the weight of the overlying soil; denser, in fact, than could be achieved with heavy compaction equipment on Earth. This is due to the meteorite impacts, which stir and loosen the surface, and at the same time, shake and densify the underlying soil.

8. As a result of the high density, the shear strength of the soil (expressed as the friction angle) is extremely high for low to medium imposed stresses. But some of the glass particles are fragile and tend to crush at higher stresses, although the strength is still quite large.

IMPLICATIONS FOR A LUNAR BASE

1. The bearing capacity of the lunar surface is more than sufficient to support virtually any conceivable structure.

2. Settlements of structures and instrument packages placed on the surface are small enough to be accounted for in the design.

3. The primary exception would be a large, settlement-sensitive telescope. In this case, the foundations would have to be 1 m deep, below the diurnal temperature variation.

4. There is sufficient depth of machine-excavateable soil in order to bury habitat structures for thermal, radiation, and meteorite insulation.

5. Drilling to 10 to 20 m depth should be possible. Below that, drilling fluids will probably be required. Solar wind hydrogen obtained from lunar mining operations may be practical.

6. Trafficability on the lunar surface is well-understood and advanced roving vehicles can be designed with existing knowledge. Ballistic hoppers will be required for long distances and for terrain steeper than 23° to 30°.

LUNAR GEOTECHNICAL INSTRUMENTATION

1. SASW: Spectral Analysis of Surface Waves. Advanced geophysical method utilizing a seismic source for detailed investigation of *in situ* structure and density.

2. Particle size measurements and fractionations: sieves; particle counter

3. Specific surface area; electric charge distribution

4. Minimum and maximum density; compaction (full-scale)

5. Hand-operated cone penetrometer

6. Vacuum direct shear testing device

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FIGURES FROM THE *LUNAR SOURCEBOOK*

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PENNSYLVANIA TURNPIKE EXPANSION: A RETROSPECT

by

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INTRODUCTION

In September, 1985, the Pennsylvania General Assembly passed Act 61, the Turnpike Organization, Extension and Toll Road Conversion Act. Act 61 empowers the Pennsylvania Turnpike Commission to undertake the construction of new projects and to operate them as part of the Pennsylvania Turnpike System. These new projects included two new expressways in Western Pennsylvania. The Beaver Valley Expressway, Toll 60, will complete a 16.5 missing link between New Castle and Beaver Falls, while the 13.2 mile Amos K. Hutchinson Bypass, Toll 66, will bypass the City of Greensburg. They were declared "fast track" projects with a five-year schedule from the beginning of final design to the completion of construction. The projects were divided into twelve design/construction sections involving seven different consulting firms and ultimately ten different contractors and numerous subcontractors. The Beaver Valley Expressway was completed in November, 1992 at a cost of $243 million. The Amos K. Hutchinson

Bypass is scheduled to be completed in November, 1993 at a cost of $276 million. Both projects were financed with $1 billion in bonds that were issued between 1986 and 1989 - no state or federal money was involved.

GEOLOGY

Both these projects are situated in the Allegheny Plateau Physiographic Province. The rock units underlying both alignments are of Pennsylvanian Age; in particular, the Allegheny and Conemaugh Groups. The groups are predominantly interbedded shales, siltstones, sandstones, coals, and limestones. The only problem formation, as far as stability is concerned, was the Pittsburgh Redbeds. Fortunately, where this formation was encountered, it was of relatively insignificant thickness.

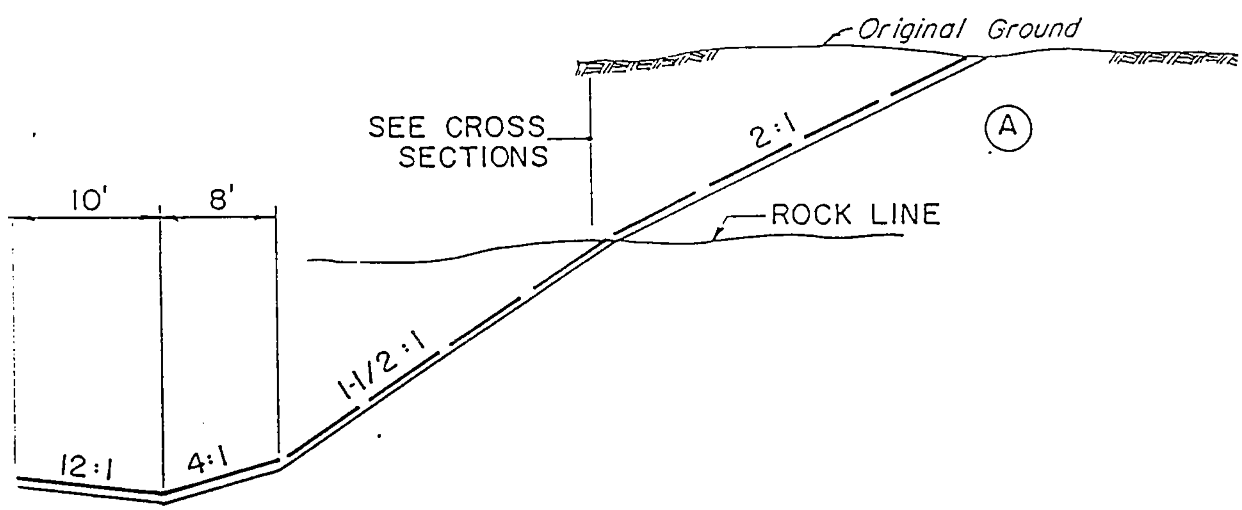
The northern half of the Beaver Valley Expressway was affected by glacial advances during the Pleistocene Age. Ice advanced into northwestern Pennsylvania during at least two stages of the Pleistocene - the Illinoian and Wisconsin stages. Five advances are known to have occurred during the Wisconsin stage and two advances during the Illinoian stage. At least two of these seven advances appear to have reached the project area. Most significant of the glacial activity was the disruption of the drainage direction of the rivers. Prior to the glaciation, the drainage direction was to the north into the present day Lake Erie region. During the glacial movement, river channels were filled with glacial material forcing them to define new paths. The result is currently seen today with drainage taking place to the west, as part of the Ohio Valley Drainage System.

The glacial materials and perched water conditions did propose a problem during construction. This led to an unusual amount of undercutting and seepage interceptors to stabilize cut slopes. The amount of quality rock available in these areas was also a problem.

DETAILS AND PROVISIONS

Since the Turnpike had not been involved in new construction for almost 20 years, new geotechnical typical details were developed for cut slopes, embankment slopes, toe benching, transition benching and coal seam treatment. New special provisions were written for acceptable rock, benching, settlement monitoring and special rolling associated with transition benching.

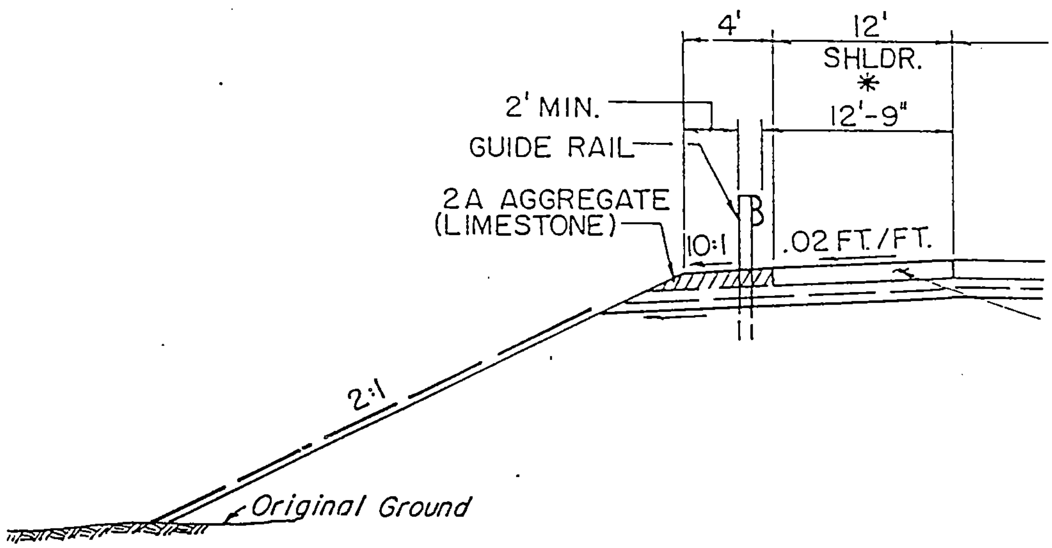
The typical detail for cut slopes is shown in Figure 1. The



CUT SLOPE DETAIL
(Not to Scale)
FIGURE 1

general rule was 1½:1 cut slopes when in rock and 2:1 slopes when in soil. There were two instances where this rule was broken, both occurred in ramp areas. On the Beaver Valley Expressway, we have two ramps that are ½:1 presplit slopes with benches at 2 coal horizons. This was done to reduce the amount of excavation. On the Amos K. Hutchinson Bypass, one ramp slope was steepened to a 3/4:1 presplit slope in order to save a township road. As you can see in Figure 1, there is also an 8 foot 4:1 slope away from the toe of the slope. This was done to reduce the amount of erosion directly at the toe of the slope and better define the swale.

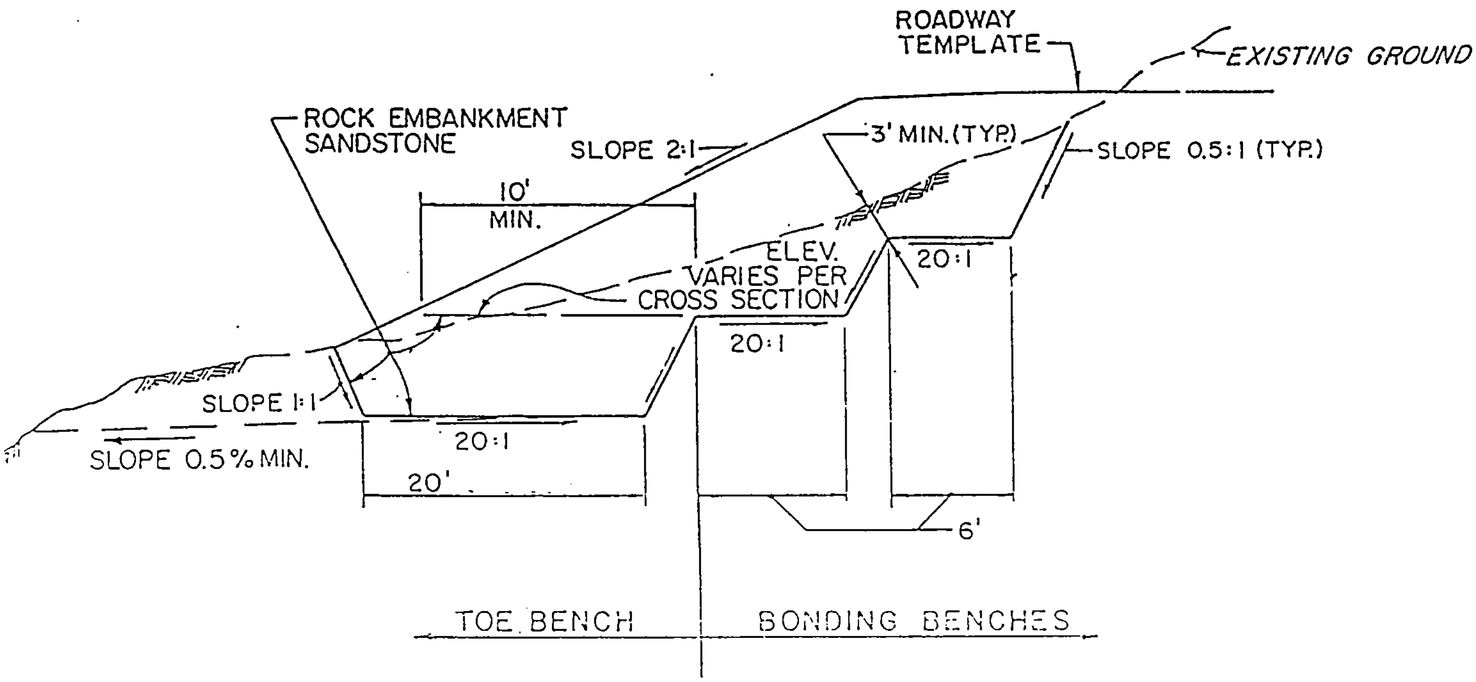
The original Turnpike was built with 1½:1 embankment slopes. The new embankment detail calls for a 2:1 slope, as can be seen in Figure 2. This was done for stability purposes. There were also



FILL SLOPE DETAIL
(Not to Scale)
FIGURE 2

exceptions to this where we steepened embankment slopes in order to miss electrical towers, driveways, wetlands, etc.

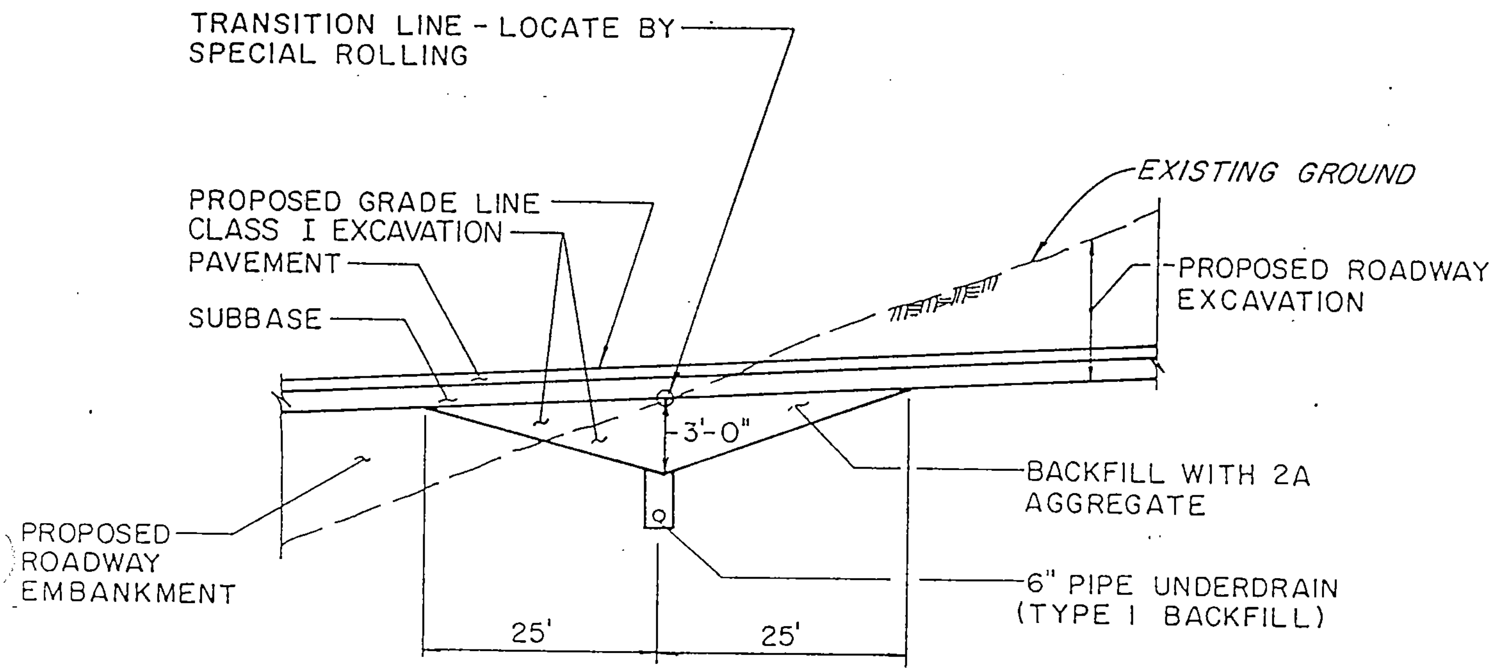
The typical detail for toe benching is shown in Figure 3. The



TYPICAL TOE BENCH DETAIL
(Not to Scale)
FIGURE 3

depth of the bench was estimated from the geotechnical information available. The bottom should be on competent material, preferably rock. The bottom of the bench is 20 feet wide, sloping back at 20:1. The benches were to be backfilled with rock, preferably sandstone, to at least 5 feet above the original ground line. We expect the rock to provide both stability and drainage. At low points in the topography, where possible, drainage galleries were placed. These galleries were 10 feet wide, filled with rock and cut at a 0.5% grade to promote positive drainage.

The Transition Bench Detail is shown in Figure 4. The transition bench was to be installed in order to eliminate the differential settlement that occurs between cuts and fills and also



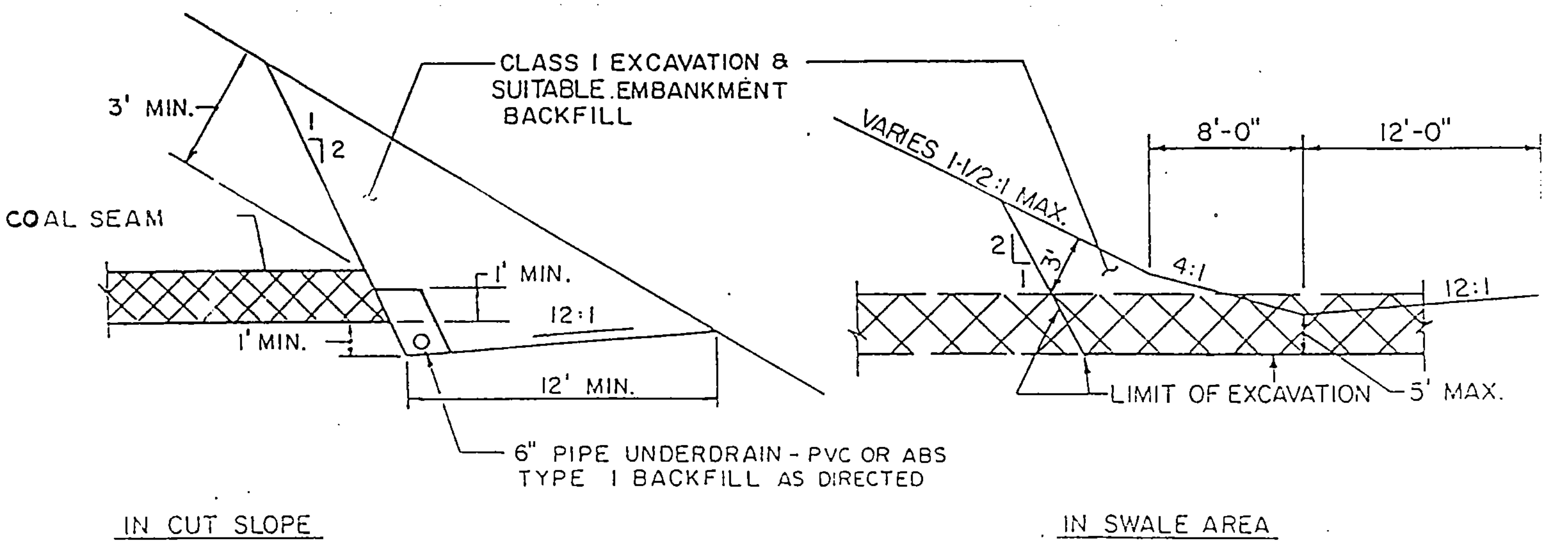
TRANSITION BENCH DETAIL

(Not to Scale)

FIGURE 4

sidehill construction conditions. The area was to be brought to subgrade, then the transition was to be located by special rolling. Once the location was determined, the area was excavated, underdrain placed and backfilled with aggregate. Many of these areas were difficult to locate due to the area being disturbed during material hauling.

A detail was developed for the treatment of exposed coal seams. This detail is shown in Figure 5. It calls for cutting a



TREATMENT OF EXPOSED COAL SEAMS
(Not to Scale)
FIGURE 5

bench at coal seams that are of 1 foot thickness or greater. The purpose of this is to drain the seam if water exists but more importantly to cover the seam to eliminate differential weathering problems.

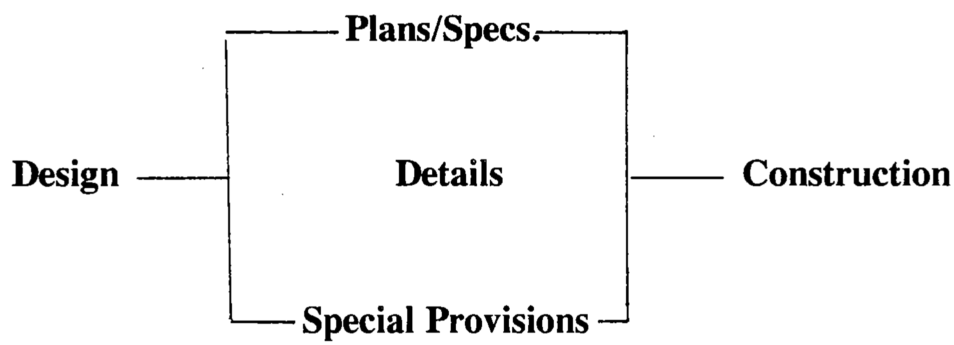
A few geotechnical special provisions were written; the most controversial being the provision describing acceptable rock. The provision was titled "Sandstone Placement". The first thing this provision did was to define the preference of materials. Sandstone was preferred, followed by rock from the excavation excluding claystone. It then prioritized how the sandstone/rock was to be placed. The first priority being benches and buttresses, followed by a 3 foot rock blanket under the entire embankment, followed by existing drainage channels. If you interpreted this provision literally, you could make a contractor take sandstone from one end of their section to the other. Realistically, it is not possible to satisfy the priorities due to construction sequencing. All the areas prioritized, such as toe benching, are not available at the same time. Fortunately, the geotechnical people in the field worked well with the contractors to avoid many problems. Only in very critical areas was sandstone specifically used when reasonably good quality rock was the only thing readily available.

A provision for settlement monitoring was written to address embankment areas in the vicinity of structures. Embankments were to be monitored for periods of 90 to 180 days to try to allow for as much settlement as possible prior to the construction of the structures. This was placed in a provision to let the contractor know "up front" so they could build this time into their schedule.

Other provisions were written for such things as benching, coal treatment, transition benching and special rolling associated with transition benching. These provisions were developed predominantly to clarify what was shown on the details.

DESIGN/CONSTRUCTION OBSERVATIONS

As we moved from design into construction, we began to see how well our provisions and details would work. As we expected, some worked well while others didn't - enter the project managers. From our experience on the two projects, project managers provide a valuable service if retained early enough in the design phase of a new project. In addition to providing a "constructability" review, the project manager helps provide an understanding of construction details and/or special provisions.



The so called "understanding" of details and special provisions should not be taken too lightly. Somehow, we need to insure that we don't have a "missing link" between the designer and the construction manager and his inspection staff. Too often, the inspection staff will misinterpret the meaning or intent of a construction detail.

An example of this would be the standard toe benching detail for embankment construction as shown in Figure 3.

The detail actually becomes part of the plans and is shown on every cross-section sheet. When shown on the cross-sections, the bottom of the toe bench is shown at the elevation of anticipated rock or acceptable material. The elevation is determined or set by the geotechnical consultant, based on his interpretation of test borings that were advanced for the design phase of the project.

When construction begins, the contractor and/or construction inspector has the tendency to excavate to the elevation shown on the cross-section. This then leads to the unnecessary excavation of sound rock or an excavation that stops short of terminating in a firm, suitable material that would provide the stability for the proposed embankment slope.

The point to be made here is that a detail of this nature is provided as a guide and does require the inspection and approval by someone with an understanding of the intent of the benching detail.

Some construction details may require more thought by the design consultant prior to blindly including it as part of the construction plans. An example of a detail that must cover a number of field conditions is the "Transition Bench Detail" as shown in Figure 4.

This construction technique is intended to provide a "transition" from cut to fill, thus eliminating the undesirable bump in the roadway pavement.

The easiest transition bench to deal with is one as shown in the sketch with the roadway centerline shown in profile. This means the roadway centerline is perpendicular to the face of the hillside and the underdrain will be placed across the roadway and outletted over the embankment slope.

However, in most instances, the transition becomes askew to the roadway centerline. In some instances, if the existing ground contours parallel the roadway centerline, the transition bench may run for hundreds of feet and may become difficult to outlet properly. Another problem would be one of interrupted benches. This could happen if we go into and out of a series of knolls that parallel our roadway centerline. The above problem can be visualized by looking at our "transition bench detail" as a roadway in cross-section or a sidehill cut-fill situation.

If the design consultant understands the application of the transition bench, he can eliminate or restrict this construction technique to short transitions that can be easily drained. As an example, in the case of a very long transition (i.e., one that parallels the roadway), the designer can call for an undercut that is an inexpensive class of excavation. The undercut will eliminate the need for a transition. We have seen this occur unintentionally in a number of transition areas. As the contractor begins excavation at the top of a hill, he cuts a haul road to the embankment at the base of the hill. His haul road virtually eliminates the transition zone.

A special provision is included with transition bench construction to provide an item for "special rolling". The special rolling is intended to take place after the roadway is brought to the subgrade elevation. It is through the special rolling operation that we can actually locate the "transition line". By use of the special rolling provision, we can verify whether or not we have successfully eliminated the need for the transition bench.

The designer and construction manager often look at the same item in a different perspective. We have all experienced the problems associated with bridge construction. Most of our structures are designed as a pile-supported structure or a spread footing on rock.

When the designer and his geotechnical engineer determine that the structure can be designed as a spread footing on rock, they attempt to become too detailed. The result is a stepped footing, attempting to set the bottom of footing on sound rock capable of supporting the design loads. The steps can number anywhere from one to a half dozen, depending on the results of the test borings.

To begin with, this stepped footing results in additional labor and forming costs. Therefore, attempts to save concrete by stepping the footer are not realistic. Also, with the number of borings advanced for a given structure, it is virtually impossible to accurately define the steps through the sound rock.

With a stepped footing, changes to the footer geometry are not practical since all of the reinforcing has been fabricated to conform to the steps.

A much better approach to the footing on rock would be to set one elevation for the bottom of footing. After the excavation is complete, an inspection of the footing is made. If it is determined that a portion of the footing contains weathered rock or rock of a quality not capable of supporting the design loads, that rock is "overexcavated" to sound rock. The overexcavated portion of the footer is simply backfilled with a lean concrete to allow for construction of a footing at one elevation.

A pile-supported structure should not present any major problems. However, we are aware of the bump or dip that always seems to appear at the approach to a structure.

In many cases, the pile-supported structure sets on an embankment that has been constructed for the new roadway. The bridge abutment becomes fixed in its location at the top of an embankment.

It appears that we do not put enough design into the embankment adjacent to the structure.

First of all, we have a specification in Pennsylvania for embankment construction. If the embankment is to be over five feet in depth, top soil is not required to be removed and stumps can be left within six inches of the ground surface.

Second, because of anticipated settlement problems, we may call for a waiting period prior to completion of the structure. But, do we have time to allow for the settlement to be virtually complete?

Perhaps we should take a more aggressive approach to the settlement problems adjacent to the structure. If we have settlement problems for an embankment not involving a structure, they can be more forgiving. However, they are too apparent next to the structure.

Maybe the thing to do is to revise the standard specification for embankment construction and create the "special provision" to address problems of the bridge approach. We may want to call for undercutting any compressible zones near the structure. This can be done for any problem soils located near the original ground surface.

The Turnpike also chose to take a different approach to the disposal of excess excavation on these projects. These two projects generated approximately 8 million yards of excess excavation. The common practice in Pennsylvania has been to make the contractor responsible for the disposal of excess material. The contractor had to locate the site, enter into an agreement with the property owner and obtain the appropriate permits. The contractor usually needs an area quickly and the permitting process can take time. Working in cooperation with the environmental agencies, the Turnpike decided to take an active role in planning for the disposal of the excess excavation and address the environmental issues in detail before construction began. The sites were located by the Turnpike with the help of the designers, project managers and an environmental consultant. The parcels were purchased by the Turnpike and the necessary permits were obtained. In some cases, these areas were developed right into the plans; in others, the contractors were given certain design criteria which they had to meet and they developed the final plan. It is hard to quantify the exact cost savings directly related to this approach, but we feel there are some. It obviously eased the burden on the contractor, but it should be noted in most cases the contractors found alternative sites that they used for some of the material. The other advantage is that we ended up with two usable parcels; one that will house a maintenance facility and one that is a future commercial site.

CONCLUSIONS

The Turnpike Commission learned many valuable lessons during the design and construction of the Beaver Valley Expressway and Amos K. Hutchinson Bypass. The following is an attempt to summarize our ideas that worked and the ones that need modified:

WORKED

- Project Managers "on board" during design
- Cut Slope Detail
- Embankment Detail
- Toe Bench Detail/Provision
- Coal Seam Treatment Detail/Provision
- Designated Waste Areas

NEEDS MODIFICATION

- Rock Placement Provision - needs to be more realistic
- Transition Bench Detail/Provision
- Special Rolling Provision - needs updated
- Settlement Monitoring - more aggressive approach to accelerate settlement around structures
- Stepped Footings - eliminate in favor of uniform footing (one elevation with a provision for lean concrete)

There were many other specific problems encountered during construction. Each one of these could warrant a paper of its own. The Turnpike Commission is looking forward to using the knowledge obtained from these projects on future ones. The Commission plans on starting the Mon/Fayette Expressway by the end of 1994. This is a $1.75 billion, 65-mile project that will connect Pittsburgh with Morgantown, West Virginia. On the heels of this project is the

Southern Beltway, a $800 million, 35-mile project that will connect the new Pittsburgh International Airport with the Mon/Fayette Expressway. Western Pennsylvania is not the only place the Turnpike is involved in new construction. A new interchange is to be built above Philadelphia, linking I-95 and the Turnpike. This project is estimated to cost between $350 - $450 million and construction is slated for 1997, if not before. We certainly intend on continuing to improve on these future construction projects using the lessons we've learned on the Beaver Valley Expressway and the Amos K. Hutchinson Bypass.

LABORATORY STUDY ON PROPERTIES OF RUBBER-SOILS

by

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LABORATORY STUDY ON PROPERTIES OF RUBBER-SOILS

ABSTRACT

by

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Scrap tires by the millions are discarded annually in the United States and other developed countries of the world, the bulk of which is currently landfilled or stockpiled. The composition of rubber tires makes them bulky, resilient, compaction resistant, and non-biodegradable. Because of their composition, used tires are a unique solid waste and disposal of large quantities of tires has, accordingly, many economic and environmental implications. Scrap tire piles which are growing each year pose two significant threats to the public: (1) fire hazard - once set ablaze, it is almost impossible to extinguish; and (2) health hazard - the water held by the tires attracts disease carrying mosquitoes and rodents. Efforts to sharply reduce the environmentally and economically costly practice of landfilling/stockpiling have stimulated the pursuit of non-landfill disposal or reuse of waste tires. A synthesis study conducted by the authors on the "Use of Waste Materials in Highway Construction" indicated that shredded tires can be used in highway construction as a lightweight fill or as a soil reinforcement material. This application of waste tires was considered very promising, since besides reducing landfill costs and related environmental problems, it can provide significant engineering and economic benefits to the highway industry.

A questionnaire survey conducted by the authors showed that some of the state highway agencies (Minnesota, Oregon, and Vermont) have experimented with the use of shredded tires as a lightweight fill material. Their experience indicated that the use of tires in embankment is feasible and quite beneficial. The University of Wisconsin-Madison, in cooperation with the Wisconsin Department of Transportation, has conducted a limited field and laboratory study to determine the feasibility of incorporating shredded tires in highway embankment. They constructed a test embankment consisting of ten different sections using locally available soil and shredded tires in a number of different ways. Their two years of sampling, monitoring and evaluation indicated that shredded automobile tires had not measurably or detrimentally affected groundwater quality. The findings from their research support the use of properly confined tire chips as a lightweight fill in highway applications. However, information on this application of waste tires is severely lacking. Only a few limited laboratory studies have been reported in the literature.

Encouraged by the results of various field studies, the Indiana Department of Transportation, in cooperation with the Federal Highway Administration, has sponsored a laboratory study to assess the feasibility of using shredded tires in

highway constructions. The study is currently being conducted by the authors at Purdue University. The study, based on comprehensive laboratory testing and evaluations, will assess the technical, economic, and environmental feasibility of using shredded tires in highway applications. The study primarily focuses on determining compressibility (through static and dynamic testing) and shear strength (by triaxial testing) of compacted rubber soil samples. Some of the variables being considered include: methods of compaction, compactive effort, tire chip size, type of soil, soil/chip ratio and methods of incorporating tire chips in soils. Environmental implications of using shredded tires in highway embankment will be determined by synthesizing information from various leachate studies. An economic evaluation will also be done to assess the benefits of this application.

The paper will include: characteristics of test materials; a brief description of testing equipment and procedures; and a summary of results of compaction, compressibility, and shear tests on compacted rubber-soil specimens. Analysis of compaction and stress-strain-strength behavior of test soils containing rubber chips will also be presented. Finally, a summary of conclusions and recommendations will be given. The recommendations will specify what tests must be run on these materials for design and construction purposes.

Environmental Property Assessments for Highway Projects: Key Elements for Successful Program Implementation

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Abstract

In 1989, the Illinois State Geological Survey (ISGS) began conducting research on methodologies and developing criteria for a comprehensive environmental property assessment program on behalf of the Illinois Department of Transportation (IDOT). Using these techniques, more than 425 environmental property assessments (EPRAs) have been completed, ranging in length from intersection improvements to 70 mile-long corridors with alternate route placements. Although EPRAs have become relatively common in conjunction with commercial real estate transactions, procedures for handling such investigations for highway projects are less well defined. Customary techniques may be applicable, but the scale or level of investigation required, time and cost considerations, and related risk assessment differ considerably from the type of assessment or audit commonly completed for a single-parcel purchaser.

As with other real estate transfers, EPRAs for highway projects are used to establish that an appropriate inquiry was made for the purposes of the "innocent land-owner" defense provided under the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA). Another purpose of an EPRA is to provide knowledge of environmental conditions that might create construction delays because of the need to obtain special permits, meet disposal requirements, and obtain and use special equipment to prevent liability through uncontrolled worker exposure to health hazards. The major, and most important, difference between the usual EPRA and that necessary for highway projects resides in the fact that the ordinary potential buyer of real estate may elect not to purchase if environmental site conditions are shown to be unfavorable, whereas the state may have to proceed with the purchase because of public need for the proposed project. Of course, such information can also be used during eminent domain proceedings.

Thus, in addition to an assessment of environmental conditions associated with a particular project, a risk assessment that evaluates those conditions in light of the project parameters must be completed. The ISGS has developed procedures for determination of a relative risk factor or "Risk Assessment Rating (RAR)" for highway projects. These RARs, expressed as *"No, Low, Moderate,* or *High",* define the level of hazard that might be encountered during construction activities and are based on specific site findings evaluated in terms of the proposed scope of work delineated for that project. The RARs reflect the **potential** for impact to the project from **actual or highly probable** environmental conditions. The findings of an EPRA can be used to mitigate potential delays and problems if a well-designed EPRA program provides essential information at critical milestones in the highway planning process.

In order to arrive at a proper RAR, the program of investigation incorporates historical research and physical examination methods. The most important attribute of the assessment process is the inclusion of a geologist with field experience and an understanding of earth-surface processes. Additionally, these assessments require an extension of effort beyond the now traditional "Phase I" limits in that some subsurface investigation must be done where historical or present use indicates a high probability of encountering contaminants. Follow-up investigations (Phase II) may be necessary and are completed by a statewide consultant on an annual contract. Information obtained as a result of the assessment process also requires care in its dissemination so that proprietary rights are maintained without putting the public at risk and creating regulatory complications.

Introduction

Environmental Property Assessments (EPRAs) have become an important component of real estate transactions, especially with regard to commercial property. These assessments, also known as Environmental Site Assessments or Property Transfer Assessments, are defined as the process to determine the environmental condition of a site, a process usually undertaken for the purpose of establishing the innocent landowner defense to the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA). The State of Illinois is the largest single property owner in the state, and the Illinois Department of Transportation (IDOT) acquires most of this property through a regular program of new highway construction and improvement to existing roadways. The Illinois State Geological Survey (ISGS) has developed an EPRA program for IDOT that provides information on environmental conditions associated with highway projects. This paper, prepared expressly for the 44th Highway Geology Symposium, briefly discusses elements necessary for successful implementation of a program that provides information on environmental conditions that may impact highway projects. Due to necessary limitations of space, detail on these elements is limited here, but will be available in a manual that will be available from the ISGS in late 1993.

Background

State and federal laws (*e.g.*, the Illinois Environmental Protection Act, Illinois Responsible Property Transfer Act, and CERCLA) make it necessary for landowners, including the state, to be aware of the environmental condition of the property under their ownership or control. IDOT routinely acquires property for new projects and improvements to existing right-of-way. It is important for IDOT to be able to assess environmental risks and associated liabilities that may accompany new acquisitions or that may arise during improvements to already-owned properties. To enable determination of environmental risk, IDOT requested the ISGS to develop procedures to identify sites that contain natural and man-made hazards prior to acquisition of property or prior to improvements to already-acquired property.

Various natural and man-made hazards may be present on properties that are the sites for IDOT operations and/or construction. More importantly, some of these hazards may be avoided if discovered in a timely fashion. The discovery of these hazards depends on thorough investigation of targeted properties and includes research into and review of the historical use of the property and adjacent properties, an examination of present uses and conditions, and subsurface investigations.

In 1989, the ISGS began a program to conduct environmental property assessments for IDOT throughout the state. This program included the evaluation of equipment and the development of procedures and methods to determine potential environmental impacts on IDOT highway construction projects. Sites investigated have ranged in length from a single parcel of a few tens of feet to a 70-mile corridor with several proposed alternate routes. More than 600 projects and addenda had been received by ISGS through March 1993 and more than 425 have been completed (Figure 1). The projects include new road construction as well as improvements to existing roads, and almost all projects, even intersection improvements, require the acquisition of some new right-of-way.

As important as the environmental condition of the property to be acquired is the nature of the project itself. Simple widening projects may not be impacted significantly by unfavorable environmental conditions such as soil or groundwater contamination from a leaking underground storage tank. But if new traffic signal equipment or other utilities will be installed, then excavation to depths of eight or more feet for foundation needs may intersect the contamination and require special or hazardous waste permits and special material handling techniques. Such considerations are relatively easy to deal with provided that IDOT has pre-knowledge and is prepared for the situation. Unexpected conditions generally can create delays, expense, and liability, especially if contamination or other conditions are exacerbated by construction and can jeopardize the health and safety of workers and the public.

Programmatic Elements

The first essential element in a program is a determination of the need for an assessment. After reviewing their needs, and with knowledge of the process used by the California Department of Transportation, IDOT designed a flow chart of screening criteria to be used by their Districts to determine if a project needs to have a hazardous waste survey and assessment (Figure 2). This flow chart was eventually converted to a Checklist/Survey Request Form that District staff could use to assess the need for additional investigation and to provide documentation for a request for investigation or to continue without a Hazardous Waste Survey and Assessment.

In order for an EPRA to qualify under CERCLA, it must be established that an appropriate investigation into the environmental condition of the real estate in question was conducted. When ISGS began this program, little published information was available pertaining to EPRAs. Subsequently, various journal articles and books have been written on the subject, and several short courses on the procedures required for completing EPRAs have been developed and presented by professional organizations. To assist in providing definitive guidelines, the American Society for Testing and Materials has recently developed a "Standard Practice for Environmental Site Assessments: Phase I Environmental Site Assessment Process (Standard Practice E.50.02.2)."

There is, however, an important difference between the "standard" EPRA as is conducted for a commercial real estate transaction and the EPRAs conducted for IDOT. Although one need for IDOT is to establish that an appropriate inquiry was made for the purposes of the CERCLA, another is to provide knowledge of environmental conditions that otherwise would impact the highway project by creating delays in construction through the need for special permits and equipment or as a result of liability associated with worker exposure to hazards. Whereas the ordinary potential buyer of commercial real estate may elect not to purchase if environmental conditions are shown to be unfavorable, IDOT may be required to proceed because of public need. Primary concerns for IDOT are construction delays, spreading of contamination by IDOT during construction, and worker health and safety. Also, in most cases, IDOT is not purchasing entire real estate parcels but only a portion which may have contamination, but may not include the contaminant source itself. Whereas environmental impact statements are developed to assess project impacts on the environment, these atypical needs require a risk assessment process designed more specifically to evaluate the impact of environmental conditions on the highway project.

Risk Assessment Rating Methodology

A model for evaluating the impact of environmental conditions on an individual highway project evolved from the concept of risk assessment. Generally, risk assessments are numerical evaluations; they are statistical analyses of likely events based upon probabilities of occurrence. For IDOT projects, however, such numerical appraisals are impractical since 1) the elements of an assessment cannot easily be reduced to quantitative values, and 2) no firm basis exists from which to analyze or evaluate events arising from some environmental condition unique to the highway project. Even the application of the Hazard Ranking System (Federal Register, Volume 55, No. 241, December 14, 1990) results in "meaningless" numeric scores because the HRS is focused on point source contamination.

For the purposes of the IDOT program, we developed a qualitative risk assessment method that enabled us to assign a relative risk factor to the probable and likely consequences of encountering man-made and/or natural hazards at a project site. For these purposes, a hazard is simply defined as a set of inherent properties believed to be detrimental to the environment and, thus to the project. The Risk Assessment Rating (RAR) was established and carries an implication for the level of hazard that might be encountered during IDOT construction activities.

Four levels of risk were originally established for use in IDOT EPRAs. The levels and definitions are:

NO. After a review of all available information, there is nothing to indicate hazardous materials or natural hazards would be a problem. It is possible that hazardous materials could have been handled on and/or transported to the parcel(s); however, all information indicates that the quantity of residual material, if any, is low and problems should not be expected.

LOW. Land use may include a hazardous waste generator, storer, or transporter, or dealings with hazardous materials. The project may be located on a low frequency flood plain or have geologic materials conducive to movement during seismic activity of low magnitude. However, based on all available information, there is no reason to believe there would be any involvement with hazardous materials of significant quantity or that natural detriments could not be easily overcome. (This is the lowest possible rating a gasoline filling station operating within current regulations could receive.)

MODERATE. After a review of all available information, indications are present that identify known soil and/or water contamination or other environmental hazard. However, the problem does not need remediation (reason must be stated), is being remediated, and/or continued monitoring is required. The complete details of remediation/monitoring requirements are important to determine what IDOT must do if the project were to continue and the parcel(s) acquired. A recommendation should be made on any parcel in this category as to acceptability for use, actions required if the parcel is acquired, and/or possible alternatives if any can be determined. (This is the lowest possible rating if anticipated construction intersects an UST.)

HIGH. After a review of all available information, it is determined that there is a high potential for hazardous material problems, man-made hazards, and/or significant natural hazards on the project (parcel). In general, a **HIGH** risk also implies that a Phase II assessment will usually be required to determine the extent of the problem. The specific presence and/or levels of hazardous materials, other concerns, and the need for remedial action will be incorporated in the report. A recommendation must be included for what further assessment is required. When a "High" RAR is assigned, a Preliminary Assessment Hazard Ranking System (PA/HRS) value will be calculated and reported.

These RARs define the level of hazard that might be encountered during construction activities and are based on specific site findings evaluated in terms of the proposed scope of work delineated for the project. It is important to understand that the RARs reflect the **potential** for impact to the project from **actual or highly probable** environmental conditions. A well-defined and substantiated RAR allows IDOT to make an informed decision as to the need for additional investigations at the site, appropriate mitigation activities, or potential alignment adjustments.

Project Research Components

In order to provide a pragmatic RAR, a program of investigations was established to collect the information necessary to determine the environmental conditions within individual project limits. This involves two general areas of investigation: 1) historical research and 2) physical examination. The historical research is conducted by staff who specialize in this subject, with assistance from a field geologist who is responsible for the physical examination of the project area, the writing of the overall report, and the determination of the RAR for the project.

Historical Research

When a project is received, it is first assigned to a member of the Historical Research/Data Management staff of the Environmental Assessment Section of the ISGS. This person initiates an investigation of the historical background and the available records on the geological/hydrogeological character of the project site. A variety of resources and reference materials are utilized in the historical background evaluation and these are constantly updated and modified as new information becomes available. In addition to material collected by the historical staff, the field geologist also supplies information essential to the completion of the historical investigation. As part of the report format, a checklist is provided that includes the various sources of information used in the report (Figure 3). This list is used by the preparer of the report and also notifies the reader as to the materials reviewed. The checklist includes the resources commonly accepted as necessary for a thorough historical investigation.

Physical Examination and Field Procedures

The field investigation for an EPRA is divided into two phases: a reconnaissance of the site termed the initial site visit (ISV) and the subsurface investigation (SI). The purpose of the ISV is to verify the geological character of the area, to confirm present land use, to identify properties that contain potentially hazardous materials, to determine natural hazards, and to delineate areas for additional investigation, while SI visits are designed to collect more detailed information including determination of the depth to the water table, soil description, soil gas sampling and analysis, polychlorinated biphenyl (PCB) screening, geophysical screening for buried objects, and other investigations as may be required.

The ISV or site reconnaissance is conducted relatively early in the project history. However, to properly conduct an ISV, it is advantageous to have some of the basic historical information already in hand. This information serves to direct the field geologist's attention beyond present land use in the search for potential environmental problems. Depending on the length and complexity of the project, an ISV includes a "walk-over" of the site or a slow drive along the project area with stops as necessary to investigate specific parcels of interest. The ISV consists of several elements necessary to a successful start of the RAR process including proper preparation prior to going into the field.

The field investigation must involve a geologist with experience in making field observations and taking meticulous notes regarding those observations. It has been our experience that although trained non-geologists can provide quality input from reconnaissance trips that focus on historical data, only an individual with an understanding of earth-surface processes can bring all of the historical, geological, and geochemical data into the perspective required to develop a risk assessment for the project for which there is a high level of confidence.

At the site, the field geologist must take detailed notes describing the following: 1) topography and drainage patterns not clearly shown on topographic maps; 2) the physical appearance of water in streams or ponds; 3) the presence (or absence) of animals and plants and their relative health; 4) the number, location, and character of buildings along the project route and conditions including ground stains or discolorations, tanks, debris, drums, dumped materials and any other features of potential concern; 5) areas where subsurface testing may be necessary (photographs commonly are used); 6) the locations of natural gas or petroleum pipelines as well as other utilities and unusually large arrays of transformers or capacitors on or above the ground; and 7) interviews with local residents and business owners. These are a few of the items necessary to the ISV. Figure 4 is the checklist used by the field geologist to direct activities at a site.

In the preceding list, two items are particularly important. The first is the great care that should be taken to identify buildings and parcels formerly used as gasoline stations. The positions of stations may have changed through time; they may have been moved back from the highway right-of-way (ROW). Yet, as the ROW encroached outward from the centerline, in many cases, underground tanks were not removed. In one case, the discovery of a tank within the ROW also revealed soil saturated with gasoline even though the tank had been secured thirty years ago using closure methods approved at the time. Evidence for possible past use of a building as a gasoline filling station includes the presence of dispenser islands, outlines of islands (new asphalt or concrete in the shape of an island), new asphalt or concrete along former piping areas, filler caps, vent pipes along the building, garage doors, service bays, pillar/canopy-type architecture, pipes protruding from the ground, tank-sized gravel areas on the lot (these may have subsided as the gravel has settled), and "wing"-type overhead lights, among other features.

The other important step in the initial site visit, and one that continues throughout the assessment process, involves interviews with local residents. Particularly valuable sources of information may include local government officials, owners, managers, and employees of businesses along the project route, and long-time residents near the site. The nature of the interview questions depends on the type of project but, as an example, for an active gasoline station these might include: 1) how long the property has been a gasoline station; 2) whether other gasoline or fuel stations were present earlier; 3) where the tanks were or are buried; 4) how many tanks are on the property; 5) whether the tanks are the original tanks; and 6) whether there is a waste oil underground storage tank.

For a former or suspected former gas station, the interview questions might include: 1) whether or not the property was ever a gas station; 2) for how long and how long ago; 3) what was present here between the time of the gas station and the present facility; 4) whether more than one gas station was present; 5) whether and when the tanks removed or filled; 6) where the tanks are or were; and 7) whether the former owner/operator still resides in the area.

Even if a gas station or underground storage tank is not a former component of the landscape, interviews with long-time residents can provide important data. For example, in one case, the resident recalled "trucks backing down into the ground." This turned out to be an accurate recollection since the location was the former site of a brewery and below-surface cooler facilities existed that had since been filled with debris and other materials. Geophysical techniques were eventually used to define the structure.

The field geologist also uses the ISV to verify the historical information and to develop a preliminary listing of sites where subsurface testing will likely be required and conducted.

<u>Subsurface Investigation (SI)</u>

In the private sector, there has been some debate regarding the inclusion of SI testing or sampling in a "Phase I Environmental Property Assessment." Generally a SI is regarded as an invasive technique that is costly and unnecessary at this point of the investigation. Without belaboring the debate, we can state that our experience strongly suggests that at least some subsurface investigation should be performed, especially at sites where there is a high probability that contamination is present. As with any program or study, the key limitation is usually fiscal. Random testing is not practical over the length of a highway project regardless of the technique used. Even an intersection project requires some constraint on the number of samples taken and tested. Equally important is the method used. Sending soil and/or water samples into a lab for analysis is costly and time-consuming.

We selected the soil gas method, a viable assessment tool that has gained in popularity since we first applied it. Technological advancements and refinements over the past two or three years have increased efficiency and accuracy and enabled users to develop a high level of confidence in the results of soil gas analysis, especially where volatile hydrocarbons are present. Prior to actual field sampling, however, a work plan needs to be developed, discussed, and approved by program supervisors. The plan must have as a focus the collection of information specific to the project description so that the data can be properly interpreted to establish a RAR appropriate to the project. Care must be taken to account for both false positive and false negative results since either carries an implication for additional, and perhaps costly, work.

The plan must include a listing of all sites at which SI will be performed. This includes the number and locations of boreholes at each parcel, and the planned depth of boreholes. The geology and estimated groundwater gradient and depth are discussed to the extent that these affect sampling sites and the potential for identifying environmental problems as indicated from historical and/or present land use. The number of holes emplaced per parcel depends on parcel size, facilities located on the parcel and any dynamic results of actual sampling.

Also important to the plan is a listing of all sites where it might be expected that SI testing would be performed, but that will not be tested, and the reasons that testing will not be conducted. Such reasons might include the distance of the facility from the proposed project, information from regulatory agencies on site status, the estimated or known groundwater gradient, and/or the nature of the geologic materials between the site and the project limits.

Following approval of the work plan, the geologist prepares for the SI in several ways. First, the locations of subsurface utilities must be determined. This can usually be accomplished with a joint meeting of utility staff and in Illinois, as in many other states, these needs are recognized and organized. Where particular utilities are not included in the coalition, checks for the locations of underground structures and pipes must be arranged separately through city or county public works departments. Also, many natural gas and petroleum pipelines are not included in these utility coalitions; phone numbers for these companies are generally located on pipeline marker signs.

Other preparations include the following: 1) notification of property occupants, perhaps using right-of-access documents; 2) recording general site conditions including date, time, temperature, weather, barometric pressure, and personnel involved; 3) surveying site conditions for immediate or surficial hazards; 4) determining general soil characteristics using an auger hole to describe the general soil characteristics such as color, grain size, odor, and texture; 5) locating of the saturated groundwater zone or water table; 6) using geophysical tools such as electromagnetic (EM) devices and magnetometers; 7) emplacement of boreholes or probes for the collection of soil gas and groundwater samples and 8) collecting samples and performing field analyses for volatile organic compounds (VOCs), polychlorinated biphenyls, and other contaminants of interest.

Throughout the entire process, concerns for safety are paramount and staff must be trained to recognize both potential and real hazards and react properly. This includes not only chemical safety issues, but proper tool manipulation and traffic awareness. The other aspect of this preliminary screening program related to field work is that almost all analyses are completed in the field. Samples generally are not collected for shipment or transport to a laboratory. The sophistication of analytical instruments for field use allows reasonably accurate determinations and assays to be made in the field, thus preempting the need to transport and dispose of potentially hazardous materials.

Report Style and Configuration

A major concern was the development of a report format that transcended individual writing styles since many staff members would be involved in report writing. We also wanted a format that could be built upon as data were gathered so that an "interim" report could be produced at almost any stage of the assessment process in order to provide IDOT with information useful as planning and construction activities progressed or changed. This form evolved considerably over the first year or two into its present configuration. Figure 5 is the Table of Contents from a typical report. Late in program development, milestones were established so that, depending on various types of projects, certain elements of the reports were completed within specified times following receipt of the project initiation request.

Program Statistics

Since the program was started four years ago in March 1989, more than 510 requests for surveys or assessments have been forwarded by IDOT to the ISGS. Additionally, about 90 projects have been significantly expanded or changed through an addendum to those projects. Once the program was well underway, we began to compile relevant statistics in order to develop procedures that would result in programmatic improvement, both in service and quality of product.

The statewide distribution of projects has remained fairly consistent over the years. The state is divided into 9 IDOT Districts of different sizes. Generally, the allocation of highway funds is proportionate to population and road use. Thus, District 1, which encompasses Chicago, Cook County and the counties immediately adjoining it, receive a significant proportion of the highway funds and, therefore, a comparable number of projects. Further, because of the long industrial and commercial history associated with the northeastern Illinois area, concern for the occurrence of man-made environmental hazards is foremost. District 1 projects account for approximately 40% of the property assessment program with District 2, immediately to the west, about 20% and District 6, which includes the state capitol at Springfield, Illinois, about 10%. The remaining 30% of the projects are divided among the other six districts.

Completion time for a project can vary considerably for different reasons. An average completion time is about 270 continuous calendar days counted as of the date the project is received. There is a very large standard deviation around this average, however. The high variability is caused by several factors, including project location, immediate need, length, and level of risk assessment rating (RAR). Both project location and RAR can be statistically linked at the 95% confidence level or above to the length of time required to complete the assessment. In addition to intrinsic factors, the experience and continuity of staff have a major impact on project completion times, especially with complex projects. Staff assignments are critical to ensuring program success.

The distribution of RARs generally appears to follow the overall distribution of projects with expected variations. For example, 46% of HIGH ratings are in District 1 (40% of projects) whereas only 12% are assigned to District 2 (20% of projects) presumably reflecting the urban and rural settings respectively. We expected that, as the program developed, the number of NO and LOW RARs would decline both quantitatively and as a percentage of total project assessments. This expectation was based on the belief that IDOT District staffs would apply the pre-screening procedure more effectively once reports had been delivered and the critique-feedback process associated with this type of program was underway. Analysis of the data as moving averages indicates that NO RARs are decreasing and HIGH RARs are increasing. However, NO findings have fluctuated markedly and an observed downward trend in MODERATE RARs has limited statistical significance. Figures 6a and 6b show the distribution of RARs and the relative completion times in terms of the RAR displayed as project blocks that reflect the first four years of completed assessments.

Use of Assessment Results

Following the submission of a final report, IDOT evaluates the assessment in terms of the latest information and status of the project. In general, only those projects that receive a MODERATE or HIGH become of concern to the Central and related District offices. Since assessment findings are based on limited physical evidence, it is prudent and necessary to conduct additional or "Phase II" work, usually over specific portions of the project route or on individual parcels associated with project construction and acquisition. For this purpose, IDOT contracts a consultant on an annual, statewide basis to followup on the assessments provided by the ISGS. The scope of work for any given project is determined from the information provided in the EPRA or "Phase I" assessment and, if necessary, from interaction with IDOT and ISGS staff familiar with the project.

IDOT uses the results from both phases of the investigation to determine the best practice to follow in continuing with the project. In a few cases, where the environmental conditions over a project area could significantly increase cost such that regional budgets would be in jeopardy, the project was cancelled. Since the cancellation of highway projects may have political ramifications, it is important that assessment results be scientifically and economically defensible.

Other implications arise from the discovery of contamination and other regulatory matters. Since the ISGS is a scientific research agency, its work is considered "research in progress" and not necessarily subject to review by the public and/or regulatory agencies. Additionally, because the work is performed under a contractual agreement between IDOT and the University of Illinois for the ISGS, data collected from the assessment investigations are considered proprietary and their use is under the control of IDOT. Nonetheless, it has been determined that it is in the best interests of the citizens of the State of Illinois for the responsible agency to be notified when contaminants of concern are encountered. In Illinois, the Office of the State Fire Marshal is notified when contaminants are associated with an underground storage tank, and the Illinois Environmental Protection Agency for other hazards.

Case Synopses

A wide range of project conditions is commonly encountered for various types of construction needs, and the assessment findings reflect those projects. Limitations of space preclude detailing case histories, but the projects can be summarized in groups with similar considerations. In all cases we have eliminated or changed the names of businesses, owners, and others where such disclosure would be inappropriate.

Urban Projects One project extended 15.5 miles through 7 villages and cities in the western suburbs of Chicago. The project entailed widening to provide two 12-foot wide through-lanes in each direction separated by a center median. Auxiliary lanes were to be added at each major intersection along with modernization of existing traffic lights. The project required new acquisition of new land (frontage of parcels), trenching for traffic signals, and rechannelization for drainage. The route crossed a complex of glacial outwash sands and gravels, alluvium, peat/muck, colluvium, and clayey and silty clay tills. One hundred sixty-nine well records existed for the nearby area.

Historical research noted that the area was nearly fully developed with light industries and other, typical commercial establishments. Some 56 facilities (parcels) contained underground storage tanks and 17 LUST incidents had been reported. Based on historical research and site visits, 78 properties had subsurface testing. Of these properties 18 had chemicals identified in the soil gas as components of petroleum products including 5 of the LUST sites and at least one site that did not contain an underground storage tank.

The RAR was stated in the following manner:

> Based upon the following and as of June 20, 1991, the date of the last physical examination of the project area, it is determined that this site has **HIGH** risk for the occurrence of hazardous materials and/or natural hazards. Compounds with retention times similar to components characteristic of gasoline were detected in soil gases in boreholes at 18 locations: [locations were listed].

Urban Intersections Urban intersections usually require very small acquisition of new right-of-way for changing the turning radius, and deep excavation for traffic light installation or improvements. Many of the intersections have had gasoline service stations occupying the corner locations since the early 1900s. The usual problems are possible contamination and location of old buried tanks that historically were next to the roadways. Mast arm supports for traffic lights require deep excavation and may increase the likelihood of encountering contamination problems by intersecting the groundwater and by excavating a large volume of soil.

In one investigation of an intersection, an apparent discrepancy between soil gas data from two separate visits required consideration for construction plans since it could reflect a temporary condition resulting from a gasoline(?) or other petroleum emission/release or the influence of atmospheric and soil conditions. In this case, we simply advised caution "since contamination at depth and variable atmospheric conditions may have contributed" to inaccurate or erroneous findings.

Rural Intersections These projects generally entail the widening of an intersection to accommodate truck traffic associated with rural business. In one case, IDOT requested a Survey because two corners of the intersection were occupied by agricultural service businesses. Both were hundreds of feet back from the roadways. One farm dealer was an old family business that had a 550-gallon underground storage tank for gasoline. Subsurface investigations for possible contamination from this source encountered buried cinders and parts of clay brick. Previous investigation of available air photos and topographic maps showed **no** signs of a buried, abandoned railway line that paralleled the state route. The owner of one business recalled seeing trains some 65 years ago and an additional, more detailed search through old topographic maps showed that a Wabash Railway line existed and that it was abandoned in the early 1930s. The current landscape shows no signs of this railway.

Rural - Proposed New Highway Corridors These projects usually consist of evaluating several possible new highway corridors. In western Illinois, one project, totaling 45 miles in length, was proposed that traversed mostly through rural farmland. Some of the proposed route paralleled existing highways and passed through several small towns. Possible hazards were two farm properties that contained underground storage tanks, a junkyard, commercial livestock farms, school UST, electrical substation and 6 crossings of floodplains. Another important concern was the need to identify sites uses as dumping grounds.

Traditionally, rural residents selected an area of their property that was either undesirable (wetlands, for example) or out-of-the-way (and out of sight) for the location of a dumping ground. As pesticides gained popularity, these areas were also used for the disposal of wastes associated with pesticides as well as motor oils and solvents. In many instances, families or businesses informally shared these areas and, as a result, contamination can be considerable and require cleanup if present. Thus, it is important to understand that rural routes may intersect contaminated areas despite the absence of commercial or industrial sites. In this case no contamination was discovered, although an area that had been used by area residents (perhaps commercially) to dispose of automobiles, farm equipment, and household goods and debris was noted and avoided in the most recent alignment.

Railroad Beds In recent years, the abandonment of rights-of-way by railroads has led to acquisition of these corridors by public entities. Typical of IDOT involvement with former railroad ROW was the acquisition of a six-mile strip of abandoned track and ROW that IDOT purchased on the behalf of, and turned over to, a group of local government bodies who will convert the right-of-way into a bicycle/recreation trail. Several industries are or were located along the tracks and ROW. Present tenants included a manufacturer of storm windows and doors and a producer of coatings, varnishes, and resins. Former industries in the immediate vicinity included a manufacturer of electric, gas, and diesel lift trucks and a metal products corporation. At least two companies were known generators and transporters of hazardous waste and/or non-hazardous "special waste". A CERCLA site existed less than 1/4 mile from one portion of the proposed project. Drainage from this site flowed toward and across the abandoned railway.

Abandoned railroad lines are notorious areas of garbage dumping and this site was no exception. Empty and partially empty drums were present along the tracks, as well as various other discarded items. Areas of fill and larger dump sites, possibly containing a variety of sanitary and industrial wastes and building materials, were also present. There was a high probability that small quantities of hazardous materials could be encountered along the path of the abandoned railway.

Our findings and recommendations in this case were that:

1. Hazardous wastes are known to have been generated and transported in the immediate vicinity of this route. A more detailed investigation of the eastern section may be desired although there is no overt evidence to suggest that the right of way contains any significant deposits of hazardous material.

2. Groundwater and surface water quality may have been and may be affected by runoff from nearby industries, particularly during periods of high water table. However, field tests showed no detectable organic compounds in standing surface water and no obvious large areas of contamination were observed.

3. No known major man-made hazards are present along the western half of the route. The western part of the site has been under individual ownership since the late 1800s, and is primarily agricultural in use. Relatively small amounts of materials improperly disposed of and other problematic fill material may represent a potential low-level hazard and should be considered during any clean-up.

In addition we stipulated that the assessment and resultant RAR of NO was based upon the limited use described by IDOT. Alteration of the property for special and adjunct use such as rest stops and/or water fountains and wells would require site-specific assessments. In general, the key to the successful assessment of long stretches of railroad ROW is centered on establishing the character of use along and on the ROW and identifying areas or "hot spots" that reflect that use.

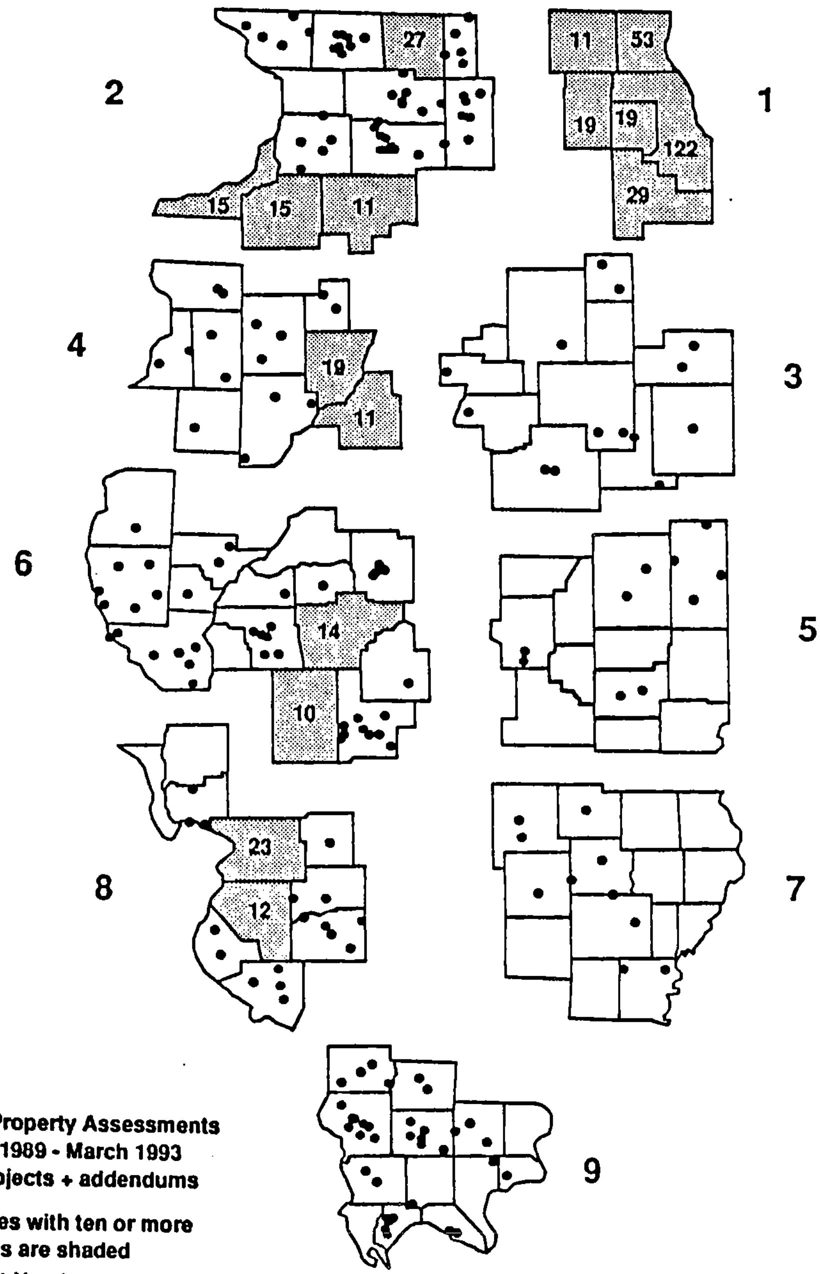
<u>Railroad Yards</u> Projects that intersect or cross railroad yards present an entirely different problem for assessing environmental conditions. The nature of activities normally associated with yards and the industrial complexes that adjoin them may appear to provide a barrier to "accurate" assessment. Intuitively it is expected that these projects will receive a HIGH RAR and, therefore, there appears to be little reason to conduct an extensive investigation to arrive at a predetermined conclusion. Nevertheless, it is necessary to adequately portray known and suspected environmental conditions over the project area to protect the DOT from liabilities that arise from working in the area.

The historical investigation portion of an assessment that includes a railroad yard is reasonably straight-forward and, because of the regulated nature of interstate transport, many records may be available to trace yard use, chemical spills, and other, related activity such as hazardous material generators and transporters. The physical examination, however, is usually most difficult to implement. In part this results from the reluctance of property owners and tenants to allow representatives of the DOT to take physical samples since this may carry an implication for being a principal responsible party (PRP) to costs for cleanup of contamination discovered on the property. If more than one alignment for the project is possible, it becomes critical to define the environmental conditions as precisely as possible, since the costs for cleanup of contaminants along one route may exceed the additional acquisition and construction costs associated with another route. With railroad yards and industrial areas in general, the state DOT will, no doubt, bear the major cost of remediation if it wants to complete the project in a timely manner. Further, Illinois courts have ruled that negative values cannot be offered for contaminated property since, at the time of evaluation, actual remediation costs have not been determined.

SUMMARY

Environmental property assessments for highway projects differ considerably from the types of assessments or audits commonly completed for a single-parcel purchaser. Commonly, the Department of Transportation must proceed with the acquisition of environmentally damaged property and may bear the major cost of remediation. At the least, delays and added construction costs that result from intersecting contaminated soil and water impact already limited and reduced public transportation budgets. A qualitative risk assessment rating defines the level of hazard that might be encountered during construction activities provides proper guidance for proceeding with the project. These ratings must be based on specific site findings evaluated in terms of the proposed scope of work delineated for the project and reflect the potential for impact from actual or highly probable environmental conditions.

Employment of individuals specifically educated and trained to collect and evaluate historical and physical evidence that will be used for risk assessment ratings is critical to a successful program. This fact alone has implications for the design of a property assessment program. Where several consultants are necessary, the procedures to be followed and basic contents of the report must be clearly stated and uniformly applied so that all assessments have the same relative value and can be correlated from project to project. In general, urban projects and projects that cross industrial complexes have greater potential for being "high risk" projects, although the designation of "high risk" in the context of highway property assessment does not necessarily imply difficulty with the project. In fact, knowledge of problematic environmental conditions prior to project commencement allows for proper planning and mitigation of conditions that might otherwise create significant impacts.



IDOT Property Assessments
March 1989 - March 1993
602 projects + addendums

Counties with ten or more projects are shaded
District Numbers are shown.

Figure 1. Distribution of IDOT Property Assessments by IDOT District.

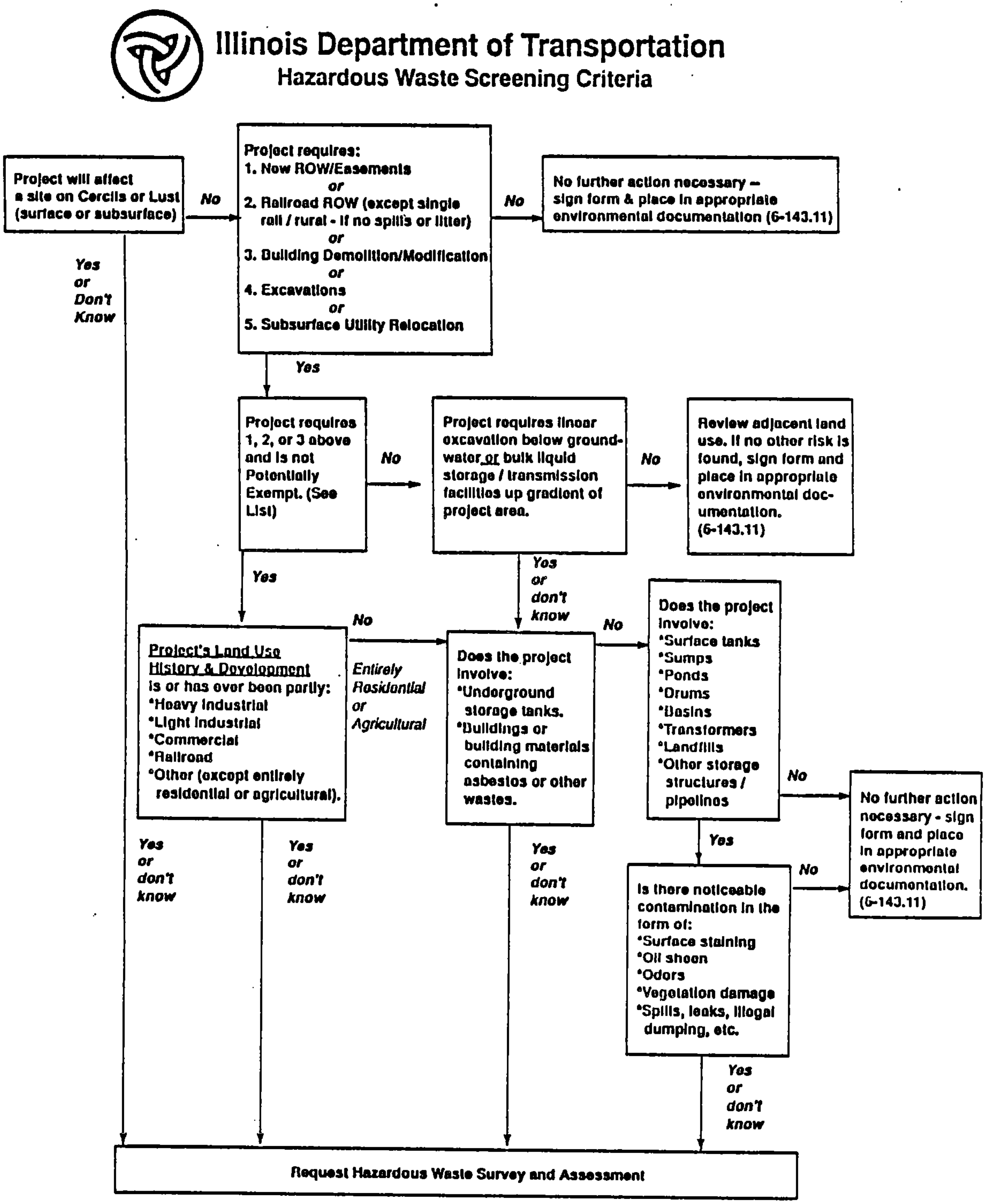


Figure 2. IDOT flow chart showing hazardous waste screening criteria.

APPENDIX A

ISGS PRELIMINARY ENVIRONMENTAL PROPERTY ASSESSMENT CHECKLIST

IDOT: ________ ISGS: ______
County: ____________
Location Coordinates ______
Nearest City/Town: ____________________

IDOT District Contact:
Name: ____________________
Phone: ____________________
ISGS Lead: ____________________

PROJECT CHECKLIST

Pre-field Checklist:

Task	Date	By
Original Material Copied:		
Current topographic map(s): Historical topographic map(s):		
Geologic Maps: Bedrock Piskin/Horberg Stack Unit Soil Survey		
Plat Maps:		
Sanborn Maps:		
Aerial Photographs:		
City Directories:		
Industry/Hazardous Materials Check: (State Museum Database, Industry Sheets)		
Pipeline Map:		
Well Inventory:		

Pre-field Checklist:

Task	Date	By
Illinois Manufacturers Directories:		
CERCLIS List:		
IEPA Well Site Survey Reports:		
IEPA Quarterly Hazardous Waste Update Report:		
Inventory of Waste Handling Facilities (IWHF):		
HWRIC Information:		
UST List:		
LUST List:		
Street Map:		
Land Use:		
Flood Hazard Map/FEMA:		
Seismic Risk Zone:		
Landslide/Slumping Potential:		
Mined-Out Area Maps (strip and underground): Subsidence Potential:		
County Collection:		
Toxic Release Inventory:		
Coal Gasification Sites:		
Injection Well Inventory:		
ICC contacted re: Railroad Spills: IDOT contacted re: Abandoned Railroads:		
Environmental Inspector contacted:		
Write-Up:		
IDOT District Environmental Coordinator Contacted:		

Historical Survey Completed by: ______ Date:
COMMENTS:
MIF = Materials in File
NF = Nothing Found
NA = Not Applicable

Figure 3. ISGS pre-field checklist form.

APPENDIX B

Initial Field Survey Checklist

Site Name ____________ IDOT NO. _________ ISGS NO. _____
Date _________ By: _____

ITEM	YES	NO	UNK	COMMENT
FLORA/FAUNA 1. Area covered by vegetation 2. Vegetation stressed 3. Animal activity or presence				
NATURAL FEATURES AND CONDITIONS 4. Depressions 5. Mounding or soil piles 6. Wetlands, ponds, lakes 7. Rivers, streams, creeks 8. Lagoons, surface impoundments 9. Soil discoloration 10. Water discoloration				
CULTURAL FEATURES AND CONDITIONS 11. Buildings/Structures 12. Inside of building(s) inspected 13. Landfills (type of fill) 14. Industry (type) 15. Asbestos source/presence 16. Drums or storage tanks 17. Pumps/Protruding pipes, etc. 18. Railroad spurs 19. Trails/Dead end roads 20. Sewer Lines 21. Water wells 22. Monitoring wells 23. Septic tanks 24. Boreholes 25. Pits 26. Solid Waste (garbage) 27. Transformers, capacitors, etc.				
AMBIENT ENVIRONMENTAL CONDITIONS 28. Unusual or noxious odors 29. Noise pollution 30. Dust/Smoke				

COMMENTS:

APPENDIX C

Field Test Information

ITEM	YES	NO	COMMENT
UST Located			
Buried Pipes			
Auger/Boreholes			
Soil Condition			
Soil Gas			
Groundwater			
Surface Water			
Canisters, cans, drums, other suspicious material			
PCB Chemical Test			

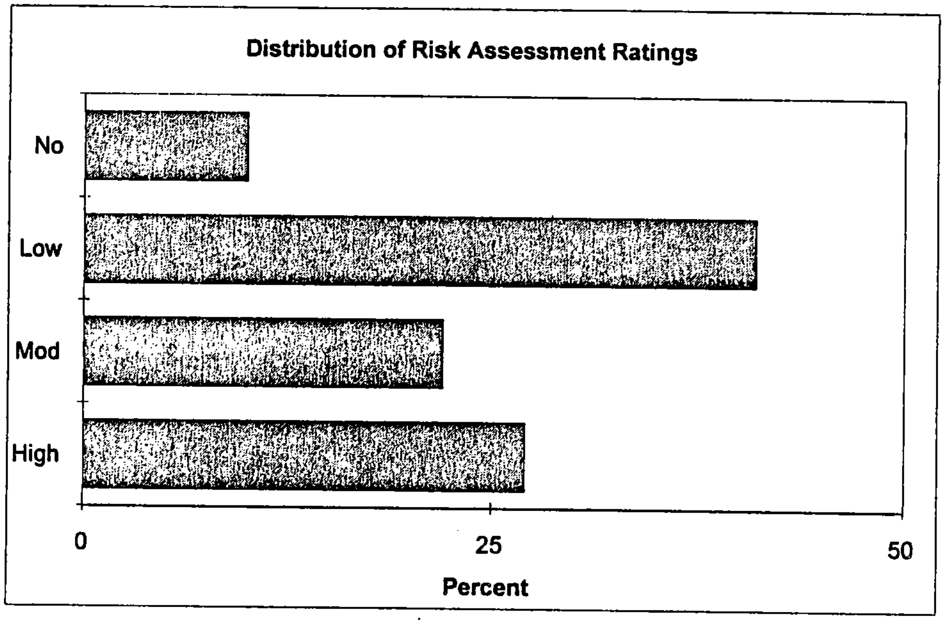
COMMENTS:

Figure 4. ISGS checklists for initial and subsequent field work.

Table of Contents

Figure 5. Typical "Table of Contents" for an IDOT assessment report.

a)



b)

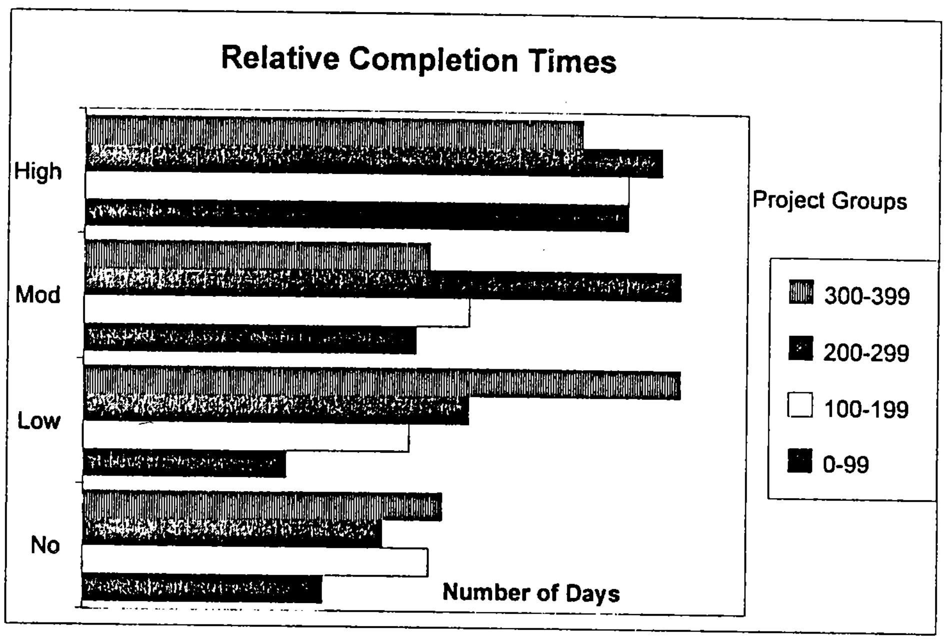


Figure 6. a) Distribution of Risk Assessment Ratings by percent; b) Relative completion times for projects (days).

CONSTRUCTION OF A FOUR-LANE

HIGHWAY EMBANKMENT

OVER A

CONTAMINATED LANDFILL

by

Larry D. Madrid, P.E.
and
Ted J. Smith, P.G.

for

44th Annual Highway Geology Symposium
hosted by USF Department of Civil Engineering
and the Florida Department of Transportation

May 21 - 23, 1993

Construction of a Four-Lane Highway Embankment over a Contaminated Landfill

Abstract

Several difficulties were encountered in the proposed crossing of the Universal Door site by the Northwest Expressway, a limited-access facility in Tampa, Florida. The site was initially altered by removal of near-surface soil for fill materials, and was then backfilled with miscellaneous construction debris. This would have resulted in settlement and ravelling of embankment material from the roadway embankments. In addition, an unspecified quantity of soils allegedly contaminated by industrial solvents was disposed of at the site.

This paper discusses the geotechnical aspects of crossing the parcel with a roadway embankment, and the various construction options considered. Additional discussions are made regarding the contamination assessment, groundwater characterization, and resulting recommendations regarding cleanup. Finally, the current status of the project is discussed in light of legal implications of the contamination, and the effect on construction scheduling.

INTRODUCTION

The Universal Door site provides an interesting case history of geotechnical and hydrogeological investigations required for road building projects. The site is located in Tampa, adjacent to the proposed corridor of the Northwest Hillsborough Expressway (Veteran's Expressway), a limited access toll facility (Figure 1). This site was originally a combination of pasture, wooded uplands, and wetlands, based on historical aerial photography. Sometime after 1972, a previous owner excavated fill from the southern two-thirds of the parcel. Starting in early 1985, this pit was filled with construction debris. Aerial photography indicates that a large portion of the site was filled with construction debris. Investigations for the Expressway began in 1988, however, additional information regarding site activities were available from regulatory files previous to 1985.

REGIONAL GEOLOGY AND HYDROGEOLOGY

The Universal Door site is located in northwestern Hillsborough County, within the Gulf Coastal Lowlands physiographic province. Relief in the area is gentle, with elevations ranging from 15 to 25 feet above mean sea level (MSL). Local drainage is to the west and southwest. Channel G, located about 800 feet north of the site, flows west into Rocky Creek, which drains into Tampa Bay.

Mr. Larry D. Madrid, P.E. is President of the Madrid Engineering Group, Inc., Tampa, Florida, and is currently involved in the construction of the Northwest Hillsborough Expressway.

Mr. Ted J. Smith holds the position of Project Geologist with Bromwell and Carrier, Inc., Lakeland, Florida, and is one of the consultants to the Tampa-Hillsborough Expressway Authority.

Figure 2 is a generalized north-south geologic cross-section through the central portion of Hillsborough County. Surficial sediments consist mainly of well sorted, fine-grained, Pleistocene terrace sands deposited during periods of higher sea levels. Sediments assigned to the Miocene, Hawthorne Group underlie the surficial sands and are divided into the Peace River and Arcadia formations. In central Hillsborough County, the Arcadia Formation is further subdivided into an upper unnamed member, followed by the Tampa Limestone Member.

The Peace River Formation consists of variably phosphatic interbedded sands, clays, and dolomite that are variably phosphatic. The extensive phosphate deposits in eastern Hillsborough County and west-central Polk County are hosted in the upper portions of the Peace River Formation (Bone Valley Member). The unnamed (upper) member of the Arcadia Formation is made up of sandy and clayey, phosphatic dolomites and limestones with interbedded sand and clay units. The Tampa Limestone Member consists of sandy limestone with minor dolomite, sand, and clay. Formations of the Hawthorne Group generally dip to the southwest, and thicken in the same direction.

The Oligocene, Suwannee Limestone underlies the Hawthorne Group except in its outcrop area, in the bed of the Hillsborough River in northeastern Hillsborough County. The formation consists of a soft to hard, fine-grained limestone with local silicified, cherty zones near the top of the formation. The Suwannee Limestone also dips to the southwest. Limestones of the Ocala Group underlie the Suwannee limestone.

The superficial water table aquifer typically occurs at depths of 25 feet or less within the surficial sands. The intermediate aquifer system, where present, is hosted by middle portions the Hawthorne Group and is separated from the surficial aquifer by clay units in upper portions of the Peace River Formation. A lower confining unit is present near the base of the Hawthorne Group. The Floridian Aquifer is hosted by the Suwannee Limestone and the Tampa Limestone Member of the Hawthorne Group, as well as in the underlying Ocala Group.

LOCAL GEOLOGY AND HYDROGEOLOGY

Local site geology (and hydrogeology) was determined from well logs of shallow monitor wells installed by LBG, Inc. during the contamination assessment and from deeper borings performed by Law Engineering, Inc. during the geotechnical investigation of the expressway corridor. Figure 3 is a north-south geologic cross-section across the site along the approximate roadway centerline. As shown, the fill material is wedge-shaped, reaching a maximum depth of 14 feet at the south end of the site and decreasing to the north. Native material underlying the fill consists of 15 to 40 feet of fine sands, silty to clayey sands, silt, and clay followed by a discontinuous clay layer and weathered limestone bedrock. This limestone is interpreted to be the Tampa Limestone Member of the Hawthorne Group. The clay layer is considered to be the paleo-weathered residuum from the limestone and overlying Hawthorne Group sediments.

The water table at the site occurs at a depth of 5 to 10 feet within fill and native soil material. Groundwater flow is to the northwest, as determined from water level measurements in monitor wells. Hydraulic conductivity values for the fill material and the upper fine sands were determined during the soil vapor and groundwater survey discussed in the following sections. Values ranged from 5 to 8 feet/day for the fill material to 0.12 feet/day for the fine sands. Average linear flow velocity for the fill material was calculated to be about 1.3 feet/year.

In the area of the Universal Door site, the Tampa Limestone Member is considered to be the top of the Floridian aquifer. Over much of Hillsborough County, the Floridian aquifer is confined by overlying clay units. In the local area of the site, however, the clay confining layer is discontinuous and is breached by numerous karst features.

SOILS EXPLORATION PLAN

The soils exploration plan for the Veteran's Expressway was modeled after exploration plans approved by the Florida Department of Transportation (FDOT). The exploration plan consisted of shallow hand auger borings, deep auger borings, and a limited number of SPT borings. Based on the proximity to roadways at which overpasses would be required, the entire section across the Universal Door property would be an above-grade embankment section between 4 and 6 feet high.

During explorations, miscellaneous construction debris (concrete, rubble, and wood) interfered with and prevented completion of the hand auger borings. At this point in time, the exploration was modified to include test pits. Test pits were dug across the site to depths of 12-15 feet. Miscellaneous debris was encountered, including reinforced concrete, wood, rocks, and other construction debris. It should be noted that there were many voids associated with the loosely placed construction debris.

GEOTECHNICAL DESIGN OPTIONS

Several design options were presented in Law Engineering's geotechnical report for this section of the Veteran's Expressway. These design options included: (1) excavation and replacement of debris, (2) reinforcement of embankment materials with geofabric and geogrid to allow the embankment material to cross over the rubble, (3) deep dynamic compaction, and (4) a combination of options 2 and 3.

The standard procedure when an unsuitable material is encountered at embankment sections is to excavate the materials and replace them with suitable backfill. In most cases, this is the least costly procedure, and is very effective in improving the roadway foundation.

A preliminary design for a reinforced embankment was also presented by Law Engineering. This plan included the use of a nonwoven geofabric directly above the construction debris. The geofabric would serve as a barrier layer so that embankment material would not ravel into voids which could open within the construction debris. In order to resist potential differential settlement within the embankment, geogrid materials could be placed near both the top and bottom of the embankment section. Two layers of Tensar geogrid were recommended, as shown in Figure 4.

Deep dynamic compaction was also mentioned as an option. This option consists of using a heavy weight dropped from a considerable height to effectively compact loose material within the construction debris area. The deep dynamic compaction is completed on a grid at regularly spaced intervals. A minimum of two passes are required to provide the proper coverage of the area. It was also suggested that deep dynamic compaction be completed to reduce potential settlement from the buried construction debris followed by a reinforced embankment with geofabric & geogrid. This would provide the highest quality control for construction of the embankment.

Based on the geotechnical exploration, the embankment configuration, and the limits of the debris, the method recommended by the geotechnical engineer was to excavate and replace.

ENVIRONMENTAL ASSESSMENT OF PARCEL

As part of the overall project requirements for a limited access highway, a preliminary environmental assessment of parcels within the corridor was conducted. The preliminary assessment included review of site, operational histories, aerial photos, lists of small-quantity hazardous waste incinerators, and other regulatory files to determine the locations of potentially contaminated sites.

Based on the site history of the Universal Door site, as reported in the regulatory files, three temporary wells were placed to further investigate the nature of buried debris, as well as surface staining observed during a walk-over site reconnaissance. The locations of the wells are shown on Figure 5. Groundwater samples from these wells were analyzed by a certified laboratory for primary drinking water standards as well as various petroleum products using EPA methods 601, 602, and 610. The analyses of the groundwater samples indicated low levels of petroleum contamination in each of the three wells. Constituents included total benzene, ethyl benzene, toluene, and xylenes (BTEX) in concentrations of approximately 8 parts per billion (ppb), with benzene at 0.6 ppb. The benzene concentration was below detectable limits in two of the three wells. Based on the results of the preliminary screening, four additional groundwater monitoring wells were constructed, and three soil borings were performed at the subject parcel in November, 1991. The location of these additional wells and borings are also shown on Figure 5. From the boring logs and monitor well log data, a geologic cross section was generated, as depicted in Figure 3. In addition, water levels from these wells provided a better indication that groundwater flow direction was to the northwest. It should be noted that advancing of the borings for the monitor wells was difficult due to the presence of bricks, rubble, organic matter, and trash. Open cavities or cavities filled with loosely compacted materials were encountered on numerous occasions.

During the installation of the monitor wells, a flame ionization detector was used to screen soils for the presence or absence of petroleum hydrocarbon and methane vapor levels within the unsaturated zone of the fill.

An additional round of groundwater sampling was conducted in November. This time, a full suite of contaminants were investigated, including analyses for the presence of volatile organics (EPA method 8260), semivolatile organics (EPA 8270), RCRA metals, chlorinated herbicides (EPA method 615), and organochlorine pesticides and PCB's (EPA method 608).

Metals were detected at MW-4 and MW-5 including arsenic, barium, chromium, and lead. Drinking water standards of 50ppb for lead & chromium were exceeded in both monitor wells. Low concentrations of lead were detected at monitor wells TW-6 and TW-7. Volatile organic compounds (VOCs) detected at monitor wells MW-4 and MW5 were toluene with concentrations of 71 ppb and 27 ppb respectively.

PHASED CONTAMINATION ASSESSMENT

After removal of surface debris piles, the leveled site was laid out in a grid pattern for completion of a magnetic survey. The purpose of the magnetometer survey is to locate magnetic anomalies which might indicate the location of buried metal such as drums. The magnetic gradient and total magnetic field at each station was measured and internally stored by the magnetometer. The field data was then downloaded to a personal computer and converted to a contour map of the gradient, using a statistical software program. The resulting countours of magenetic gradients were evaluated for the presence of anomalies in the magnetic field. Due to the presence of rebar in the concrete rubble, the results were generally inconclusive. No conclusive information regarding buried drums was found.

The next level of the assessment was a gas-chromatographic survey which was conducted with the assistance of a subcontractor. Sixty-four soil gas samples, eight groundwater samples, and two soil samples were collected over a period of five days. The soil gas and groundwater samples were analyzed for BTEX compounds and several chlorinated hydrocarbons using a portable gas chromatograph. The soil samples were analyzed in the laboratory by EPA methods 601 and 602. The soil gas survey was subjected to extensive quality control procedures to ensure the reproducibility of results.

The gas chromatograph survey identified three areas where contaminated soils were likely to be present at elevated levels. Therefore, additional wells were placed in these areas of most likely contamination.. Results of laboratory analyses of soils at the site indicated that the fill materials located at the Universal Door site could be considered clean, when compared to the clean soil criterial listed in Chapter 17-775.400 of the Florida Administrative Code. These cleanup levels were adopted for soils treated in a thermal treatment facility, but can also apply to any soil which would be disposed of following treatment by any methods.

An additional round of groundwater samples were collected in July, 1992. Florida drinking water standards were exceeded by benzene at MW-4, lead at MW-9, and chromium at both MW-8 and MW-9. However, other organic and inorganic compounds were not detected in most of the wells. The results of the contamination assessment were reported to the Florida Department of Environmental Regulation in October, 1992.

FOLLOW-UP ACTIVITIES:

Additional sampling was undertaken in October 1992. Sampling was restricted to the two wells located at the "hot spots" (MW-4 and MW-5), the upgradient well (MW-8), and the downgraded well (MW-9). The samples were analyzed for chromium. lead, benzene, toluene, ethyl benzene, xylenes, and total VOA. The results of this final round of sampling indicated that all of the analyzed constituents were below detection limits in all of the wells. Results for the July 1992 data compared with the October 1992 data are shown in Table 1. On the basis of these analytical data and the information contained in the contamination assessment, a recommendation of "No further action" was made for this site.

REGULATORY COMPLIANCE

Throughout the process of the contamination assessment, the Florida Department of Environmental Regulation was contacted and advised of the progress. With each level of consideration, a different division within the Department expressed concern. On the basis of the analytical results, the site was viewed in three distinct manners. Initially, the site was considered a potentially contaminated petroleum site, based on the results of the screening tests completed by BCI. Later on, when additional compounds were analyzed, the site was viewed by the regulatory agencies as a potential hazardous waste site. Finally, when contaminant levels had dissipated to below detection limits, the site was considered as a landfill. Regulatory compliance is different for each of these three categories, and potential remedial efforts also differ.

As a result of the contamination assessment, the initial recommended method of excavation and replacement was reviewed. On the basis of additional costs required for monitoring during removal, special disposal costs, etc., this option is currently being reviewed and compared with other options for constructing over the top of the existing landfill, and providing an impermeable cover material for the landfill in compliance with Florida landfill regulations. Discussions with the regulatory agencies are ongoing concurrent with the roadway construction around the site.

ACKNOWLEDGEMENTS:

Bromwell & Carrier, Inc., Consultants to the Tampa-Hillsborough County Expressway Authority for Geotechnical and Environmental Assessments, Lakeland, Florida

H.D.R., Inc..,Consultant for Contaminational Assessment and Remedial Action Planning, Tampa, Florida

Leggette, Brashears, and Graham, Inc., Groundwater Consultants, Tampa, Florida

The Tampa-Hillsborough County Expressway Authority

Florida Department of Treansportation, and Post, Buckley,Schuh,& Jernigan (General Consultants)

Howard, Needles, Tammen, and Bergendoff, General Consultants to the Expressway Authority

Madrid Engineering Group, Inc:
Deborah Ground, Word Processor
Ricardo Leon, Engineering Technician
Roger Zickermann, CAD Designer

NORTHWEST HILLSBOROUGH EXPRESSWAY PROJECT

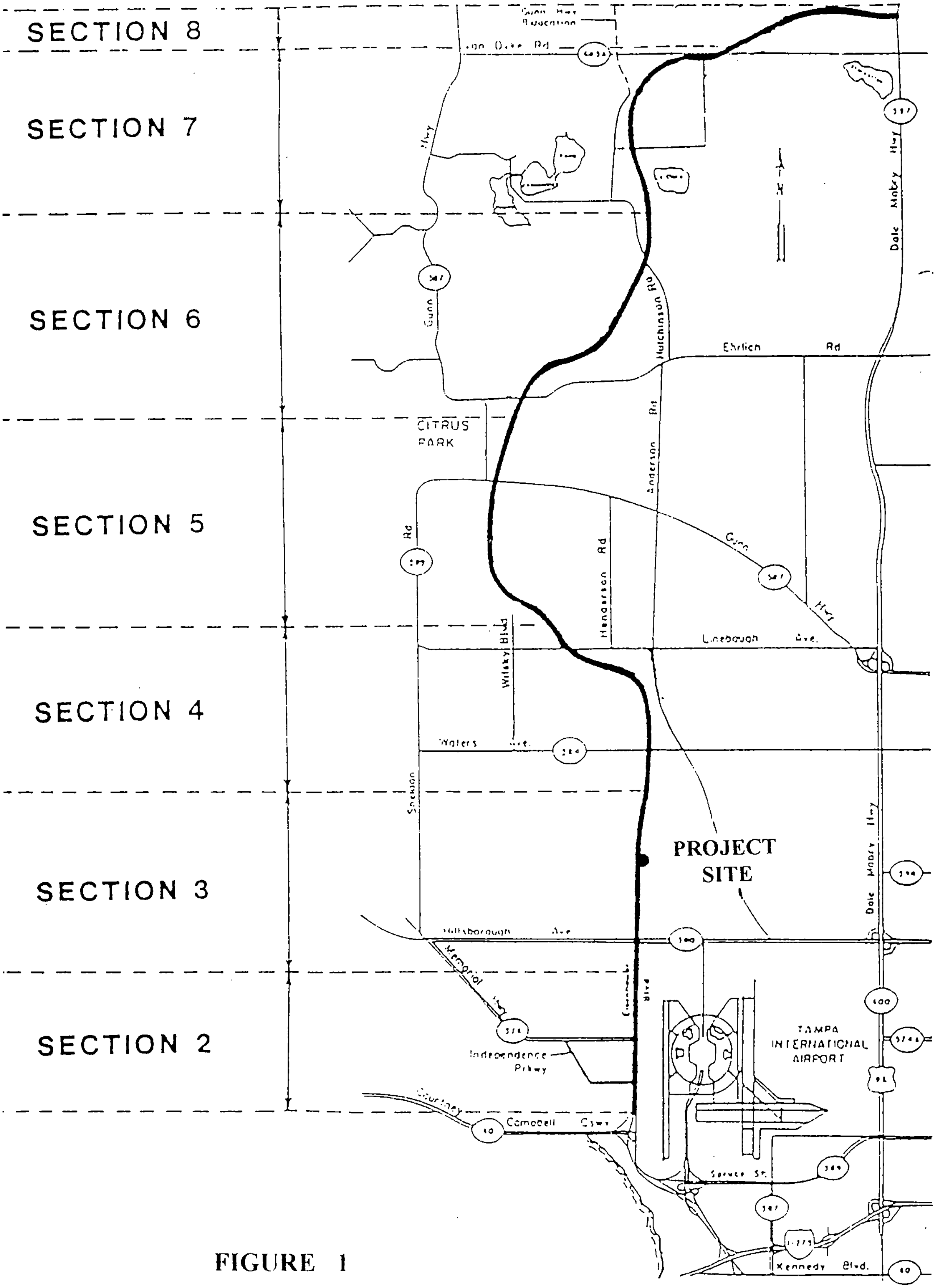
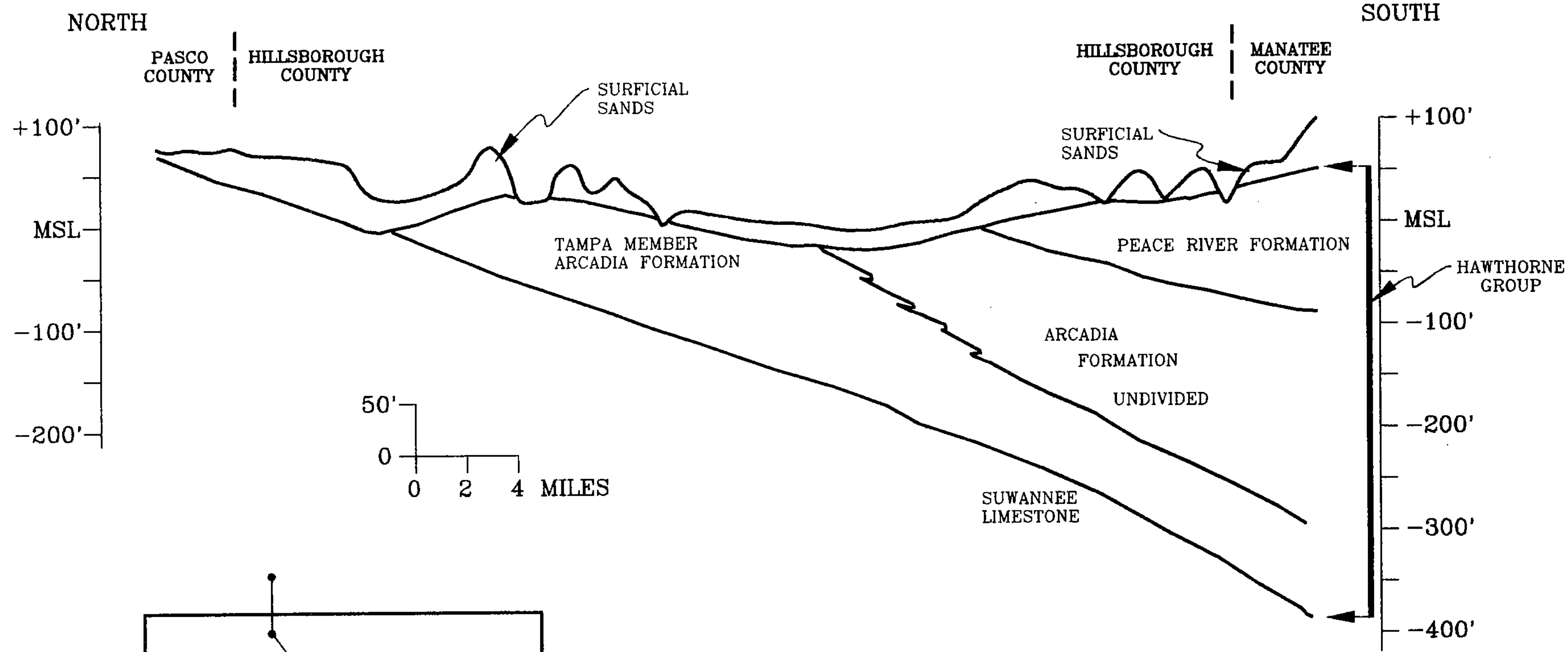
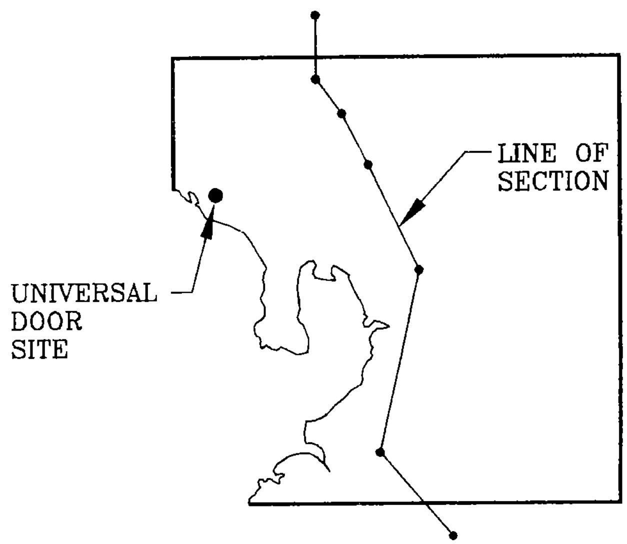


FIGURE 1



GENERALIZED GEOLOGIC SECTION



HILLSBOROUGH COUNTY LOCATION MAP

*AFTER CAMPBELL, 1984

MADRID ENGINEERING GROUP, INC. GEOENVIRONMENTAL CONSULTANTS	5-7-93 SCALE: NOTED	FIGURE 2

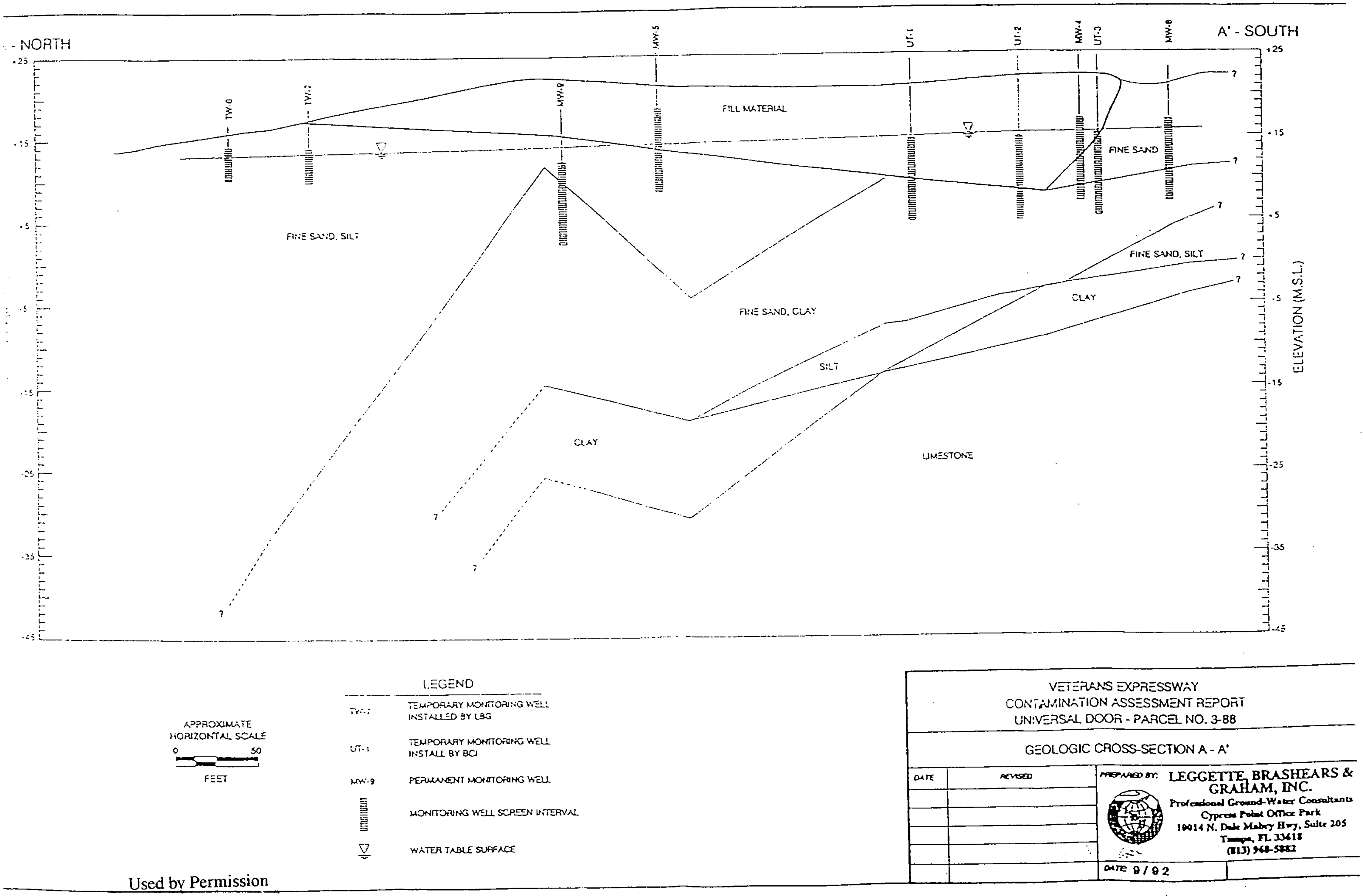


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FIGURE 3 GENERALIZED SITE GEOLOGIC SECTION

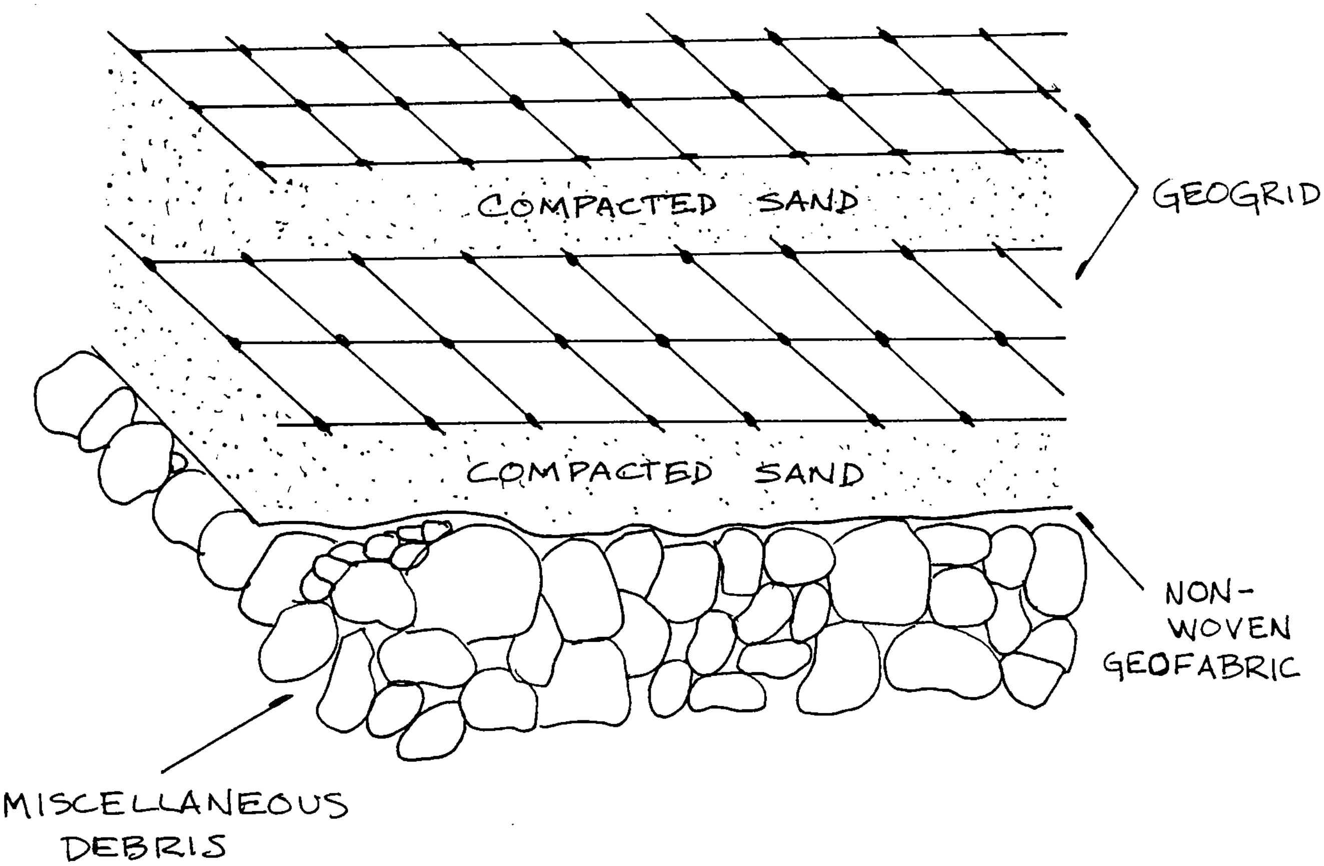
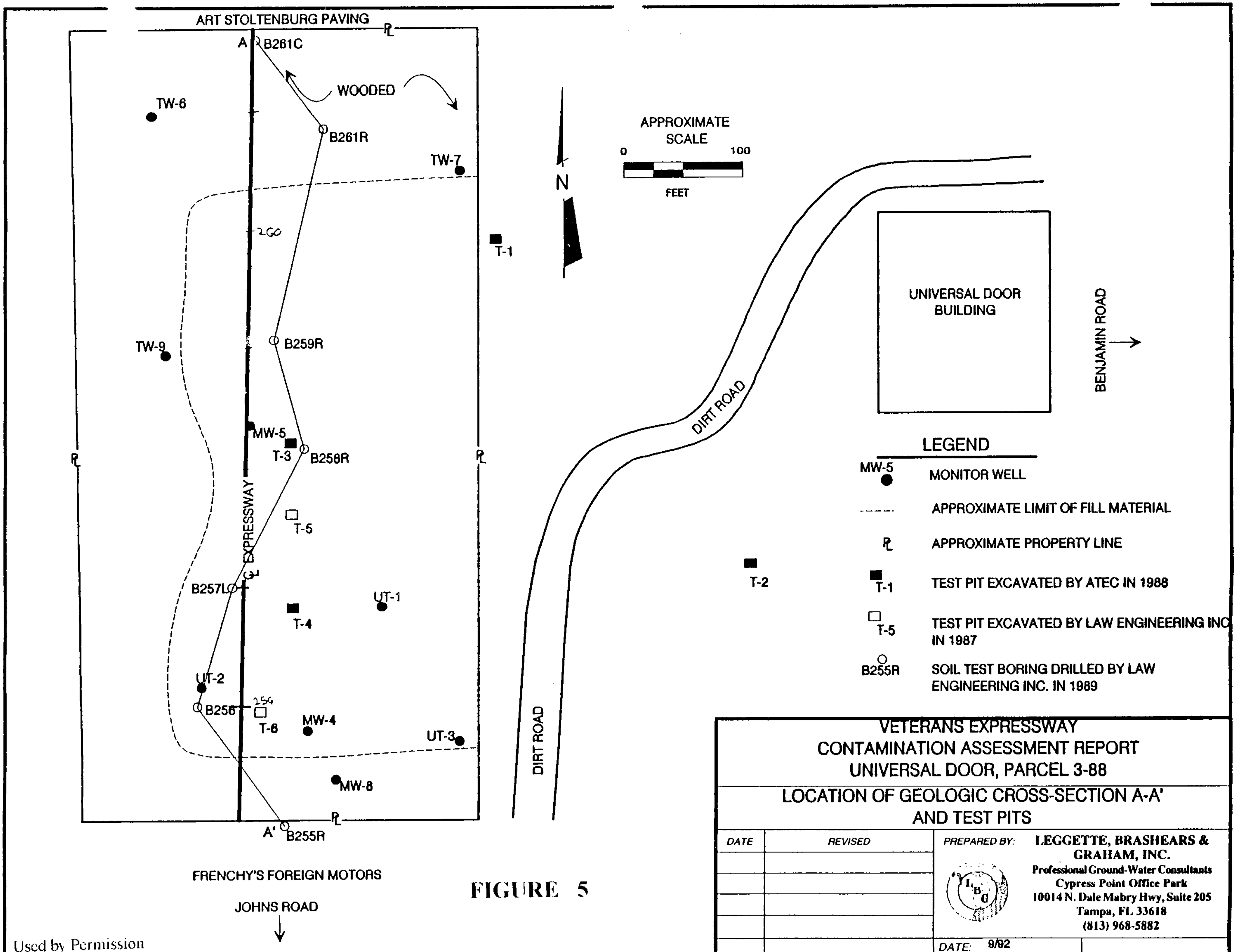


FIGURE 4 REINFORCED EMBANKMENT SECTION



VETERANS EXPRESSWAY
CONTAMINATION ASSESSMENT REPORT
UNIVERSAL DOOR, PARCEL 3-88

LOCATION OF GEOLOGIC CROSS-SECTION A-A' AND TEST PITS

DATE	REVISED	PREPARED BY:
		LEGGETTE, BRASHEARS & GRAHAM, INC. Professional Ground-Water Consultants Cypress Point Office Park 10014 N. Dale Mabry Hwy, Suite 205 Tampa, FL 33618 (813) 968-5882
		DATE: 9/92

FIGURE 5

UNIVERSAL DOOR PROPERTY

GROUNDWATER ANALYTICAL DATA

WELL NO.	CHROMIUM (mg/l) 7/29/92	10/15/92	LEAD (ug/l) 7/29/92	10/15/92	BENZENE (ug/l) 7/29/92	10/15/92	TOLUENE (ug/l) 7/29/92	10/15/92	ETHYLBENZENE (ug/l) 7/29/92	10/15/92	XYLENES (ug/l) 7/29/92	10/15/92	TOTAL VOA (ug/l) 7/29/92	10/15/92
MW-4	<0.05	<0.05	24	<5.0	1.4	<1.2	<2.0	<2.0	<1.8	<1.8	2.6	<1.8	4	<0.6
MW-5	<0.05	<0.05	39	<5.0	0.7	<1.2	<0.9	<2.0	<0.9	<1.8	<1.0	<1.8	0.7	<1.2
MW-8	0.13	<0.05	42	<5.0	<5.0	<0.6	<5.0	<1.0	<5.0	<0.9	*	<0.9	<5.0	<1.2
MW-9	0.24	<0.05	64	<5.0	<5.0	<1.2	<5.0	<2.0	<5.0	<1.8	*	<0.9	<5.0	<1.2

PDWS = 0.05 PDWS= 50.0 PDWS= 1.0

NOTES

1) THE LESS THAN SIGN (<) INDICATES THAT THE PARAMETER WAS NOT DETECTED AT OR ABOVE THE METHOD DETECTION LIMIT (MDL). THE NUMBER FOLLOWING THE "<" SIGN IS THE MDL FOR THE ANALYSIS OF THE SAMPLE.

2) PDWS = PRIMARY DRINKING WATER STANDARD

3) ug/l = MICROGRAMS PER LITER

4) mg/l = MILLIGRAMS PER LITER

5) * = ANALYSIS NOT PEFORMED FOR THIS PARAMETER

6) RESULTS FROM THE JULY 29, 1992 ANALYSES FOR CHROMIUM AND LEAD ARE UNFILTERED SAMPLES.

7) RESULTS FROM THE OCTOBER 15, 1992 ANALYSES FOR CHROMIUM AND LEAD ARE FROM BOTH UNFILTERED AND FILTERED SAMPLES.

8) INFORMATION TAKEN FROM CONTAMINATION ASSESSMENT REPORT FOR UNIVERSAL DOOR (LBG, 1993)

PREDICTING THE COMPRESSIVE STRENGTH OF ROCKS FROM AGGREGATE DEGRADATION

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ABSTRACT

Uniaxial compressive strength and percent degradation were measured, in both dry and saturated states, for 6 sandstones, 8 carbonates, and 8 igneous/metamorphic rocks. The purpose was to determine if an empirical relationship existed between the two properties and if percent degradation for a given rock could be used to predict its compressive strength. The compressive strength was measured according to the procedure described in ASTM method D-2938, whereas a modified version of the AASHTO compaction test was used to induce aggregate degradation. The aggregate for the degradation test consisted of five different size gradations that included 1 1/2"-1", 1"-3/4", 3/4"-1/2", 1/2"-3/8", and 3/8"-#4. The effect of impact energy on aggregate degradation was evaluated by varying the number of hammer blows per layer from 10 to 50, at intervals of 10. In addition, dry density, percent absorption, and the Los Angeles abrasion loss were determined for all rock types. The data from various tests were correlated using bivariate regression analysis.

Preliminary results indicate a strong correlation (r=-0.92) between compressive strength and percent degradation for the sandstones, a rather weak correlation (r=-0.72) for the igneous/metamorphic rocks, and no correlation (r=-0.04) for the carbonates. The results also show that compressive strength is generally lower for the saturated cores than the dry cores, and percent degradation is higher for the saturated aggregates than the dry aggregates. Neither the aggregate size nor the number of hammer blows used in the degradation test were found to affect the relationship between compressive strength and degradation. A strong correlation (r=0.91) was observed between percent degradation and L.A. abrasion loss for the sandstones, and a moderately strong correlation (r=0.80) for the igneous/metamorphic rocks. Based on the correlations between percent degradation and L.A. abrasion loss, a 50% degradation appears to be the average acceptable limit for the sandstones as well as for the igneous/metamorphic rocks. Bivariate regression did not indicate a correlation between percent degradation and L.A. abrasion loss for the carbonates. In general, the high-density carbonates and igneous/metamorphic rocks exhibit lower values of percent degradation and abrasion loss compared to the low-density sandstones.

INTRODUCTION

The uniaxial compressive strength and degradation are important properties of rocks when used as concrete aggregates and basecourse materials in highway construction. The uniaxial compressive strength is also a widely used property in other areas of rock engineering (Bieniawski, 1974; Pitts, 1984). Although the uniaxial compression test, as specified in the American Society for Testing and Materials (ASTM) procedure D-2938, is simple to perform, the compression testing equipment is expensive and sample preparation is time consuming. In addition, the core samples needed for the test are not always available. On the other hand, the degradation test, which measures the resistance of an aggregate to breakage during compaction, involves simple and inexpensive equipment (Proctor mold plus hammer), and sample preparation is not as meticulous as that required for compression testing. Thus, there is a need for establishing an empirical relationship between compressive strength and degradation that could be used to predict compressive strength from degradation. A review of available literature shows that presently there is no established method of predicting compressive strength from degradation. There is also a need for evaluating, both qualitatively and quantitatively, the effect of density, percent absorption, aggregate particle size, number of hammer blows used in the degradation test (impact energy), petrographic characteristics (mineralogy, grain size, grain shape, amount and type of cement, type of grain-to-grain contact), and water saturation on the relationship between compressive strength and degradation.

A test closely related to degradation is the Los Angeles abrasion test which measures the resistance of an aggregate to impact, abrasion, and crushing. The test is used to evaluate the quality and suitability of aggregate material for construction purposes. A generally acceptable limit of L.A. abrasion loss for concrete aggregates is 40%. The L.A. abrasion loss needs to be correlated with the results of degradation test to establish a limit of allowable degradation for practical applications. The present study was undertaken to fulfill the aforementioned needs.

METHODOLOGY

Sample Collection and Preparation

Large-size block samples of 22 different rock types, including 6 sandstones, 8 carbonates, and 8 igneous/metamorphic rocks, were collected from quarries, highway cuts, and various engineering firms. Table 1 shows the types and locations of the rocks sampled. The rock blocks were cored in the lab using an NX size (54 mm) coring bit. In all cases, the cores were drilled perpendicular to the bedding or foliation. The cores were cut and lapped for compression testing in accordance with the specifications outlined in ASTM method D-2938. Aggregate samples for the degradation and L.A. abrasion tests were obtained from the rock blocks remaining after the cores were drilled. The blocks were broken into aggregate size material using a sledge hammer and a rock crusher. The

Table 1: Rock types used in the study.

SANDSTONES:	
Berea sandstone	Amherst, OH
Grimsby sandstone	Buffalo, NY
Tuscarora sandstone	Dauphin, PA
Sharon sandstone	Chardon, OH
Bald Eagle sandstone	Roaring Spring, PA
Grimsby sandstone	Buffalo, NY
LIMESTONES & DOLOMITES:	
Indiana limestone	Fort Wayne, IN
Wabash limestone	Peru, IN
Delaware limestone	Sandusky, OH
Putnam Hill limestone	Wooster, OH
Elbrook dolomite	Blacksburg, VA
Guelph dolomite	West Millgrove, OH
Onondago dolomite	Amherst, NY
Mississinewa dolomite	Peru, IN
IGNEOUS & METAMORPHIC ROCKS:	
Sylacauga marble	Sylacauga, AL
Dolomitic marble	Madoc, Canada
Cockeysville marble	Westminster, MD
Triassic diabase	Chantilly, VA
Occoquan granite	Occoquan, VA
Sykesville schist	Bethesda, MD
Lakefield nepheline syenite	Nephton, Canada
Elzevier granite	Kaladar Canada

aggregate produced was sieved to obtain the proper size gradations required for both tests. All specimens used in this study were oven dried at 105°C for 24 hours prior to lab testing.

Laboratory Testing

The uniaxial compression test, as described in ASTM procedure D-2938, was used to measure compressive strength. Ten cores of each rock were tested, as recommended by Yamaguchi (1970) and ASTM (1990), and average values of compressive strength were obtained. In addition, the compressive strength was measured on three saturated cores of each rock.

The AASHTO compaction test (1978) was used to induce degradation because of its simplicity and availability. In order to investigate the effect of impact energy on the amount of degradation, the number of blows per layer of aggregate sample in the mold was varied from 10 to 50, at intervals of 10. The effect of particle size on degradation, and its relation to compressive strength, was evaluated by using five different size gradations of aggregate which included 1 1/2"-1", 1"-3/4", 3/4"-1/2", 1/2"-3/8", and 3/8"-#4. Degradation was measured on three samples of each size gradation, and an average of the three measurements was used for correlation purposes when investigating the effect of particle size. The degradation for each size gradation was computed as follows:

$$Degradation = \frac{wt.\ of\ sample\ passing\ the\ lower\ size\ sieve}{total\ wt.\ of\ sample\ before\ compaction} \quad (100)$$

The degradation test was also performed on three saturated samples of 1"-3/4" size aggregate.

Percent absorption was measured on core samples according to the procedure outlined in ASTM method C97. The density of each rock was determined from weight and volume measurements made on the oven-dried core samples. An average of five measurements of diameter and length, respectively, was used to compute the volume of each core needed for density determination.

The L.A. abrasion test was performed following ASTM procedure C-131. The aggregate used for this test conformed to "A" gradation which consists of four size ranges (1 1/2"-1", 1"-3/4", 3/4"-1/2", 1/2"-3/8"), each containing approximately 1250 g of sample.

Data Analysis

Bivariate regression analyses were performed to determine the relationships between compressive strength and degradation, and between degradation and L.A. abrasion loss. Regression analyses were also performed to investigate the effect of aggregate particle size, impact energy, and water saturation on degradation, and the relationship between compressive strength and degradation.

Table 2: Average values of properties measured.

Rock Type	Absorption (%)	Density (pcf)	L.A. Loss (%)	Comp. Strength (psi)		Degradation* (%)	
				Dry	Saturated	Dry	Saturated
SANDSTONES							
Berea sandstone	5.86	132.31	86.3	8304	5629	77.4	79.9
Grimsby sandstone	1.90	154.30	29.7	19842	17328	49.9	52.7
Tuscarora sandstone	0.62	159.01	41.8	18710	16554	55.6	59.0
Sharon sandstone	6.63	129.03	99.5	4931	3458	80.4	84.3
Bald Eagle sandstone	0.63	162.35	23.9	23551	188332	34.2	41.9
Grimsby sandstone	2.53	150.84	25.1	20913	15256	45.0	47.2
LIMESTONES/DOLOMITES							
Indiana limestone	1.59	155.93	32.0	4963	3960	44.4	52.0
Wabash limestone	2.35	158.75	32.7	15723	9044	35.3	49.3
Delaware limestone	0.66	167.13	27.0	22042	16422	39.4	49.0
Putnam Hill limestone	0.31	166.68	23.7	22650	16230	28.3	32.8
Elbrook dolomite	0.21	176.08	25.6	26031	25836	44.4	44.9
Guelph dolomite	1.95	154.99	35.7	7128	5069	48.0	52.4
Onondaga dolomite	2.00	161.18	18.4	30401	18341	43.2	48.3
Mississinewa dolomite	4.33	151.37	31.7	19558	9036	53.3	62.8
IGNEOUS/MET. ROCKS							
Sylacauga marble	0.01	168.71	48.3	13208	13072	56.6	60.9
Dolomitic marble	0.02	183.42	18.0	26717	21713	34.4	52.8
Cockeysville marble	0.10	169.71	45.0	15220	12049	45.1	47.3
Triassic diabase	0.04	187.12	13.3	26366	26334	15.6	15.4
Occoquan granite	0.20	166.93	30.1	19803	13283	39.6	50.2
Sykesville schist	0.23	175.20	26.0	19995	12290	45.2	46.3
Lakefield nepheline syenite	0.10	169.71	39.3	21902	13150	39.1	43.8
Elzevier granite	0.13	167.60	30.6	19367	15079	32.8	31.7

*The degradation values are for 3/4"-1" size aggregate.

RESULTS AND DISCUSSION

The average values of density, absorption, L.A. abrasion loss, compressive strength, and degradation for the rocks tested are given in Table 2. A comparison of the data for various properties in Table 2 shows that generally speaking the rocks with higher density and lower percent absorption have correspondingly higher values of compressive strength, and lower values of L.A. abrasion loss and percent degradation. This is especially true of the sandstones and igneous/metamorphic rocks. Previous studies (D'Andrea et al., 1965; Shakoor and Bonelli, 1991) have also found the existence of a strong correlation between compressive strength and index properties. The compressive strength and degradation data (Table 2) indicate that in all cases the saturated cores and aggregates have lower compressive strength and higher percent degradation values than the dry cores and aggregates.

Bivariate regression analysis revealed that a significant correlation did not exist between compressive strength and degradation ($r=-0.04$), or between degradation and L.A. abrasion loss ($r=0.17$), for the limestones and dolomites studied. Thus, degradation can not be used to predict compressive strength for the carbonate rocks.

For sandstones, the statistical analysis indicated that slopes of the regression lines for different size gradations of the aggregate were not significantly different and that the data for different size gradations could be lumped together for correlation purposes. Figures 1 and 2 show plots of compressive strength versus degradation and degradation versus L.A. abrasion loss, respectively, for the combined data. A correlation coefficient of >0.9 in both cases indicates that a strong correlation exists between compressive strength and degradation, and between degradation and L.A. abrasion loss. The regression line for compressive strength versus degradation plot (Figure 1) indicates a decrease in compressive strength of approximately 309 psi per 1% increase in degradation. The degradation versus L.A. abrasion loss plot (Figure 2) indicates an approximately 5% increase in degradation per 10% increase in L.A. abrasion loss. Figure 2 shows that a degradation of 52% for dry sandstones corresponds to an L.A. abrasion loss of 40%, which is considered acceptable for concrete aggregates. Figures 3 and 4 are plots of compressive strength versus degradation and degradation versus L.A. abrasion loss, respectively, for the saturated sandstone samples. The correlation coefficient is >0.95 in both cases which indicates that a strong correlation exists between the aforementioned variables in the saturated state.

Figures 5 and 6 show the plots for compressive strength versus degradation and degradation versus L.A. abrasion loss, respectively, for the igneous/metamorphic rocks. For compressive strength versus degradation (Figure 5), the correlation coefficient of 0.72 indicates a rather weak correlation. For degradation versus L.A. abrasion loss (Figure 6), the correlation coefficient of 0.8 suggests that a moderately strong correlation exists between the two parameters. Again, statistical analysis justified lumping the data for various size gradations together for a given correlation. Figure 5 shows that a decrease of 297 psi in compressive strength occurs per 1% increase in degradation, whereas the slope of regression line in Figure 6 indicates an approximate increase of 7% in degradation per 10% increase in L.A. abrasion loss. Figure 6 shows that a degradation amount of 48% corresponds to an L.A. abrasion loss of 40% for the igneous/metamorphic rocks in the dry state.

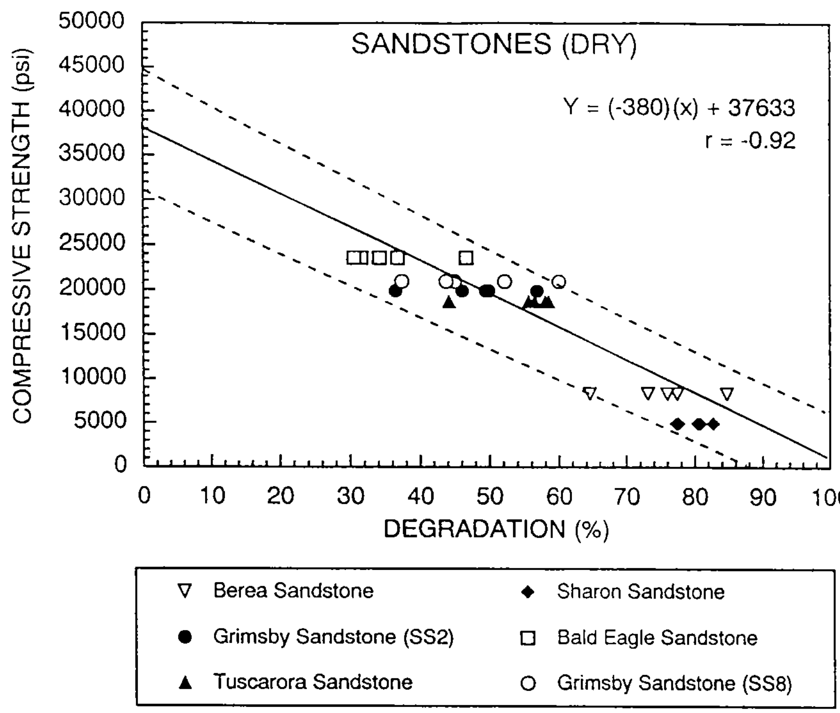

Figure 1: Relationship between compressive strength and degradation for dry sandstones. The dashed-lines indicate 95% confidence band.

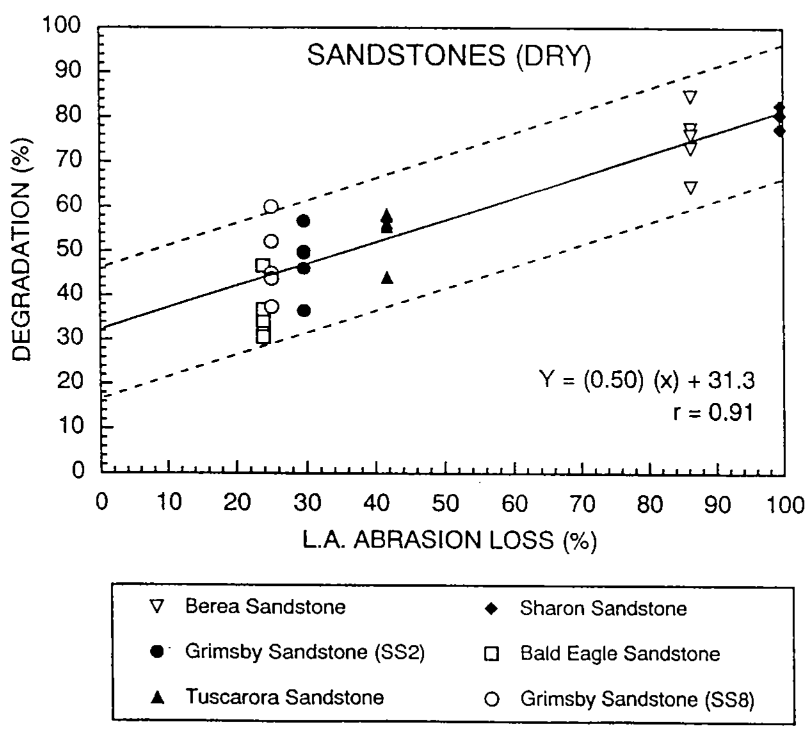

Figure 2: Relationship between degradation and L.A. abrasion loss for dry sandstones. The dashed lines indicate 95% confidence band.

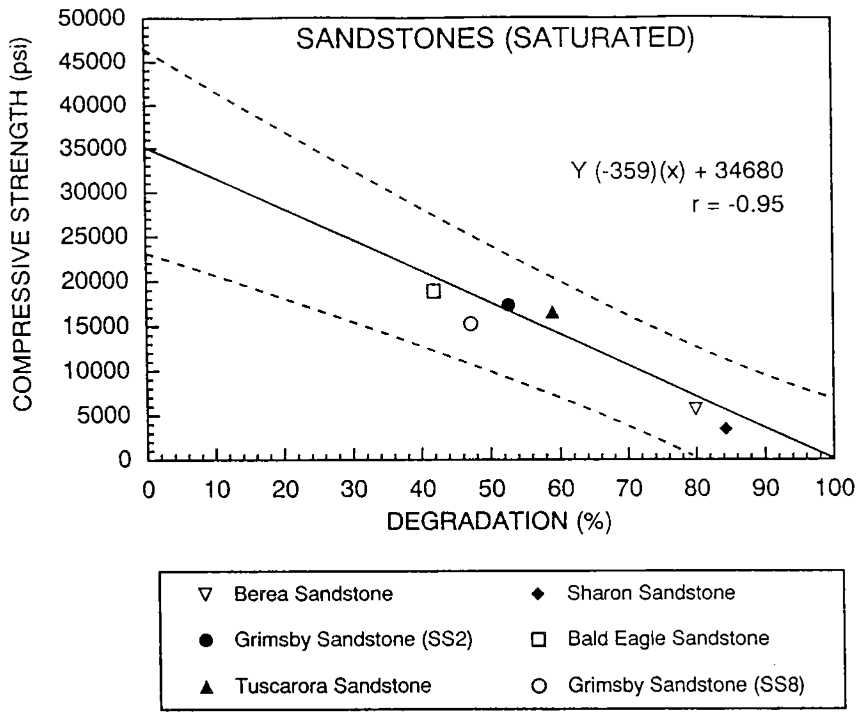

Figure 3: Relationship between compressive strength and degradation for saturated sandstones. The dashed lines indicate 95% confidence band.

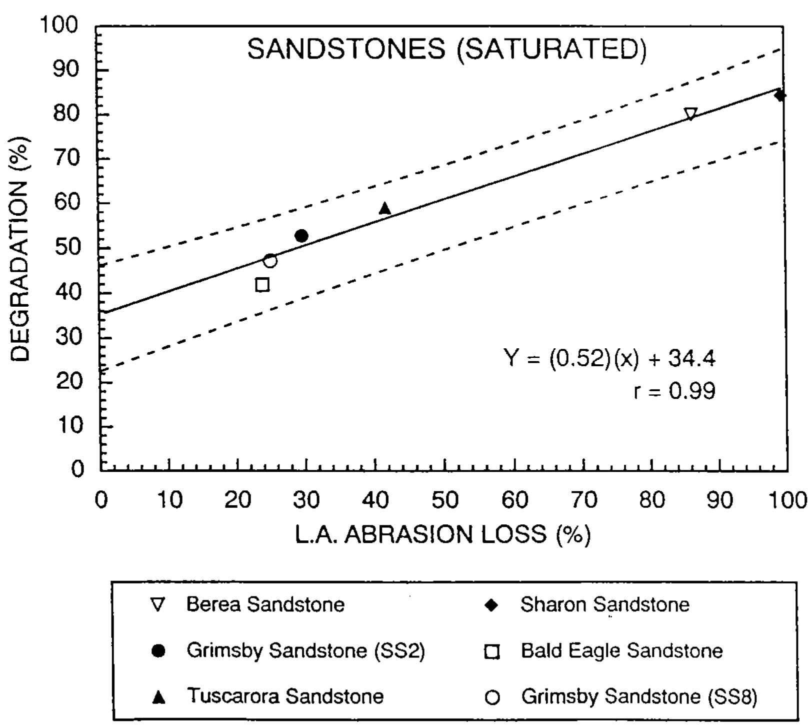

Figure 4: Relationship between degradation and L.A. abrasion loss for saturated sandstones. The dashed lines indicate 95% confidence band.

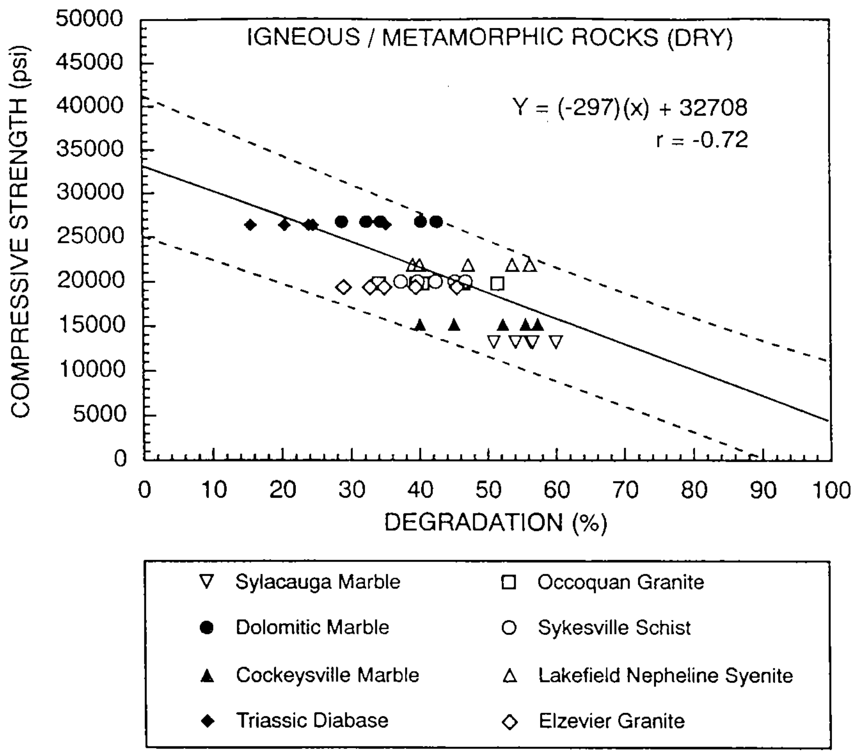

Figure 5: Relationship between compressive strength and degradation for dry igneous/metamorphic rocks. The dashed lines indicate 95% confidence band.

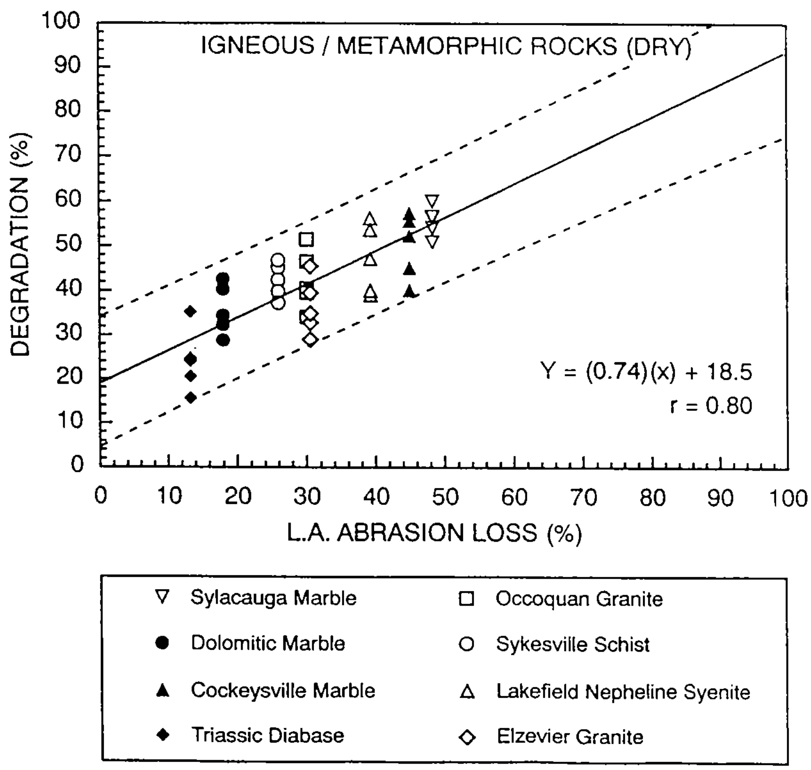

Figure 6: Relationship between degradation and L.A. abrasion loss for igneous/metamorphic rocks. The dashed lines indicate 95% confidence band.

It should be noted that although a strong correlation (min. r=0.92) was found to exist between degradation and the number of hammer blows (impact energy) for all rock types, it did not affect the results of regression between compressive strength and degradation. Therefore, degradation measured on 3/4"-1" size aggregate was used for correlations of saturated samples.

PREDICTION EQUATIONS

Bivariate regression analysis lead to the development of the following prediction equations for sandstones and igneous/metamorphic rocks:

Dry Sandstones (r=-0.92)

Compressive Strength (psi) = (-380) (% Degradation) + 37633 (1)

Saturated Sandstones (r=-0.95)

Compressive Strength (psi) = (-359) (% Degradation) + 34680 (2)

Dry Igneous/Metamorphic Rocks (r=-0.72)

Compressive Strength (psi) = (-297) (% Degradation) +32708 (3)

The high correlation coefficient values for Equations 1 and 2 indicate that these equations can be used to predict compressive strength for sandstone rocks with some degree of confidence. Degradation can provide only rough estimates of compressive strength for igneous/metamorphic rocks, as indicated by the relatively low correlation coefficient (0.72) for Equation 3.

CONCLUSIONS

The results of bivariate regression analyses show that degradation can be used to predict compressive strength for both dry and saturated sandstones, but not for the carbonate rocks. Degradation can provide only rough estimates of compressive strength for igneous/metamorphic rocks in the dry state. Furthermore, the correlation between degradation and L.A. abrasion loss shows the following as the acceptable limits of degradation for practical applications: 52% for dry sandstones, 55% for saturated sandstones, 48% for dry igneous/metamorphic rocks. These can be approximated to 50%, 55%, and 50%, respectively.

At this stage, it is not clear why the limestones and dolomites do not show a correlation between compressive strength and degradation, or between degradation and L.A. abrasion loss. The petrography of all rocks is currently being studied and the data will be analyzed using multivariate analysis. It is hoped that this additional information will shed some light as to why there is a poor correlation between compressive strength and degradation for limestones and dolomites. Furthermore, it may improve other correlations, especially the correlation between compressive strength and degradation for igneous/metamorphic rocks.

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New Precursor of Stick-Slip Movement of Rock Block

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Abstract

Shear waves were used to probe granite interfaces undergoing frictional sliding. The decrease in the amplitude of shear waves transmitted across the interfaces can be used as an indication of stick slip behavior. A rapid decrease in shear wave amplitude was observed at or prior to the sudden slip of the interfaces. In some experiments, premonitory amplitude attenuation occurred 1.4 seconds prior to slip for forced frictional sliding, and 10 seconds prior to slip for creep sliding. After the slip, the interfaces stick and shear wave amplitude increases to a constant level.

Introduction

A common problem in the routing and construction of highways is the stability of the road cut. The resulting rock slope often contains numerous joints and bedding planes that can result in serious landslides and block movements. For example, sheet detachments in hard, unweathered granite are a continuing rock slide hazard for the highway and railroad running along the shore of Howe Sound, north of Vancouver (Goodman, 1993). In Norway, rockslides from the Ramnefjell mountain have occurred seven times between 1905 and 1955 (Grimstad & Nesdal, 1990). Two of the rockslides resulted in the death of 134 people from waves that range 40 - 45 meters in height that were generated by the rock debris falling into a fjord.

Most rock slopes are stable after the initial cut. However, environmental factors (such as weathering, fluctuations in groundwater table, or earthquakes) may cause the slope to become unstable and initiate rockslides. Earthquakes, in particular, can trigger landslides on both soil slopes and rock slopes, and in karst regions can trigger subsidence in sinkholes. All of these are hazardous to highway construction.

Repetitive ruptures of faults are often viewed as a sequence of stick-slip events. Frictional behavior between rock blocks can also be described by stick-slip movement. Hence, the existence of a precursor to sudden slip along a fault or joint is of practical importance to predicting fault movement that could lead to rock slides or tunnel failure. This paper reports laboratory evidence of a possible precursor to slip along a fracture, i.e., seismic wave attenuation during active seismic monitoring.

Previous Work

Stick-slip movement occurs when the frictional resistance between blocks of rocks changes. A decrease in frictional resistance leads to a very sudden slip and an associated stress drop. The slip is followed by another sudden slip. Between slips, there is a period of no motion and a recharge of stress (Scholz, 1990a).

Monitoring changes in stress or dilatancy along a fracture undergoing shear movement requires a technique that is sensitive to changes in the physical properties of the fracture that may occur before the fracture slips. Through laboratory experiments, Wang et al. (1975) studied the use of seismic wave velocities to detect stick-slip behavior along rock interfaces. In a direct shear experiment on Westerly granite, compressional and shear wave velocities were measured for waves (3 MHz) propagated normal to the fracture. No change in velocity was observed during the stick-slip process. Based on laboratory evidence, Wang et al. (1975) also concluded that no dilatancy occurred along the joint or in the rock during frictional sliding. No other similar laboratory experiments have been reported since that time.

Recent theoretical developments for wave propagation across a fracture have shown that at laboratory frequencies (MHz), the amplitude of a seismic wave transmitted across a fracture will be more sensitive than seismic velocities to changes in fracture properties (Pyrak-Nolte et al., 1987a; Pyrak-Nolte & Cook, 1987b). This theoretical result is based on modeling a fracture as a non-welded contact by assuming a discontinuity in displacement. Typical velocities and transmission coefficient curves are shown in Figure 1 as a function of fracture specific stiffness. For a laboratory frequency of 1 MHz, the magnitude of the transmission coefficient decreases significantly with decreasing fracture specific stiffness, while very small changes occur in transmitted group velocity. Therefore, seismic wave attenuation may be a good indication of imminent slip along fractures and joints.

Experiments and Procedure

Quartz monzonite (Stripa granite) samples were used to investigate the effect of frictional sliding on shear wave attenuation. Three cylindrical samples (5.1 cm in diameter by 3.8 cm in height) were coaxially aligned and placed in a load frame between aluminum pistons containing piezoelectric transducers (Figure 2). The surface roughness profiles of the granite interfaces have a height variation of less than 300 microns based on measurements from a laser-head profilometer. A double-shear apparatus applies a shear load to the middle block after a normal load is established. Piezoelectric transducers generate shear (S) waves that propagate along the axis of the rock stack, i.e., perpendicular to the interfaces. The natural frequency of each crystal was 1 MHz and the transmission crystal was pulsed with a 500 V spike of 0.3 μs duration at a repetition rate of 100 Hz. The shear-wave polarization was oriented parallel to the direction of shear loading.

A linear variable displacement transducer (LVDT) with a resolution of 100 μm was used to measure the displacement of the middle rock. Higher resolution was not attained because of electronic noise. The normal and shear pressures on the samples were measured using electronic pressure transducers. A four-channel digital oscilloscope (LeCroy 9314) recorded the transmitted shear waveform (each waveform consists of 50 points per waveform, 0.1 μs/point), displacement, and pressures using a sampling rate of 200 milliseconds, which allowed exact temporal correlation. The data were transferred to a computer that determined the peak-to-peak amplitude of the received shear waveforms. The peak-to-peak shear wave amplitude (denoted hereforth as simply the shear wave amplitude) is the difference between the magnitudes of the minimum and maximum voltages for the first arrival cycle of the wave form (Figure 3). The peak-to-peak amplitude is used as a measure of the shear wave attenuation caused by sliding interfaces.

During the forced stick-slip experiments, a constant normal load was applied to the rock column, while the center rock in the column underwent shear loading. The shear loading cycle was initiated by increasing the shear load until a slip occurs, followed by a finite time during which no further load was applied. The finite time is allowed to pass in order to monitor the S-wave attenuation between consecutive slip movements. After this time, the center block was reloaded.

For the creep experiments, the samples were set-up identical to the stick-slip experiments. However, the shear pressure was increased until sliding was imminent and then held constant. The pressure, displacement, and S-wave amplitude were recorded using the oscilloscope. An audible acoustic emission was used to indicate a sudden slip.

Test Results

Two types of movements are observed corresponding to two loading conditions: 1) forced stick-slip movement with episodic loading and displacement; and 2) creep movement under constant normal and shear stresses.

Forced stick-slip movement

Figure 4 shows the shear displacement and shear force for an experiment of forced stick-slip movement of a normal stress of 7.47 MPa. During a slip episode, the rock displaced suddenly and an audible acoustic emission occurred. This stick-slip behavior appears as a sequence of steps on the displacement curve in Figure 4. During the sticking period, the shear force is manually recharged until the rock slides again. At the moment of sliding, a sudden release of shear force of about 1.4 kN was observed. The maximum shear force prior to slip and the subsequent stress drop after slip appear to be the same for each individual stick-slip event (Figure 4). The variation of shear force with shear displacement is shown in Figure 5. Note the characteristic saw-tooth appearance of the curve. The saw-teeth represent the shear force released as the rock slips.

Figure 6 shows the shear displacement and peak-to-peak shear-wave amplitude from a stick-slip experiment where the normal stress was 6.18 MPa. The first 10 seconds of data in Figure 6 represent the initial conditions of the experiment and record the electronic noise of the system. Fluctuations in the shear-wave amplitude from electronic noise are small during the initial 10 seconds. Therefore, the subsequent changes in shear-wave amplitude during the experiment are caused by movement along the rock interfaces.

After the initial 10 seconds, shear pressure was applied manually. Between 10 seconds and 50 seconds, the shear pressure was increased and the rock appeared to be sliding; no sticking was observed visually or audibly (through acoustic emissions). During this interval, a "hump" occurred in the S-wave amplitude curve with fluctuations in the shear wave amplitude. After this period of presumed continuous sliding, stick-slip began and the shear pressure was increased. During stick-slip (for times approximately greater than 50 seconds in Figure 6) the experimental results show that each sudden slip event was highlighted by a decrease in shear-wave amplitude (Figure 6). The shear-wave amplitude attained a minimum value of 16-18 percent of the total shear-wave amplitude. After the decrease (when no stress is applied), the shear-wave amplitude recharges to a constant value (Figures 6 & 7). However, subsequent recharges or rebounds in shear-wave amplitude never reach the pre-slip value because of changes in the system geometry.

To determine if the decrease in amplitude occurs before or after slip, shear displacements and shear-wave amplitudes were monitored at the moment of sliding (Figure 8). The precursor (the initial decrease in the shear-wave amplitude) occurs 1.4 sec before the interfaces slip. For this stick-slip event, the minimum value of shear-wave amplitude occurs during slip.

The transmitted shear waveforms shown in Figure 9 for the data in Figure 8 are for before, during and after the slip. The shear wave amplitude decreases to a minimum at sliding and increases after the slip event, but there is no significant change in the arrival times of these waves (i.e., no change in wave velocity). This observation agrees with the test results from Wang et al. (1975) and the theory shown in Figure 1.

Creep movement

A test was performed to examine if any premonitory changes in shear-wave amplitude would be observed for interfaces experiencing creep behavior. For these experiments, the shear pressure was increased until sliding was imminent, after which the stress was held constant. The normal load on the samples was 6.02 MPa. The sliding occurred 480 seconds after the test began, and was observed from its acoustic emission. The test results in Figure 10 show that a decrease in shear-wave amplitude began approximately 10 seconds before the slip occurred.

Precursor times

A histogram of precursor times was constructed for the stick-slip experiments to examine the probability of a premonitory decrease in shear-wave amplitude before slippage occurs (Figure 11). There is a high probability of either no precursor or a very short precursor before slippage. However, the precursor time may depend on the shear-loading rate. By comparing the time over which the amplitude decreased before sliding for the experiments of stick-slip and creep, it is observed that slower loading rates produce longer precursor times.

Figure 12 shows the relationship between observed precursor time and displacement. It is not clear if any relationship exists between them.

Discussion

Dilation

Earthquake precursor phenomena are categorized based on the physical models used to explain the mechanisms, i.e., (1) fault constitutive relations, and (2) bulk rock relations. The major difference between these two models is that the first model assumes that no change in the properties of the material surrounding the fault occurs (Scholz, 1990b). The most common models of the first category are nucleation and lithospheric loading. Dilatancy models typify the physical models of the bulk rock relation category.

The dilatancy model is related to the increase in void space within the solid resulting from crack formation. The typical unconfined compressive strength of granite ranges from 141 to 226 MPa (Goodman, 1989). Therefore, it is unlikely that under laboratory condition (6.18 MPa) that there will be a permanent change in properties of the rock. Also, laboratory experiments with simulated mid-crustal stresses (Byerlee, 1978; Hadley, 1973; Wang et al., 1975) showed that the shear stress required to cause dilatancy in rock is ordinarily greater than the stress required to cause sliding along faults. Therefore, precursors indirectly related to dilatancy may not explain the observed precursor phenomena presented in this study. If dilatancy is observed directly prior to failure, this may be due to deformation of a joint or fault rather than the rock matrix. In other words, it is believed that the change in fracture properties is the underlying reason for the precursor phenomena observed in laboratory.

Roughness

Roughness is a scale dependent variable that plays an important role in the frictional behavior of rock. In laboratory tests, dilation of a joint is closely related to the roughness of the joint. From the bi-linear peak shear strength envelope for rough surfaces, a joint subjected to shear loading may undergo one of the following two kinematic mechanisms that result in reaching the failure criterion (i.e., movement along the joint) depending on the magnitude of the normal stress.

1. Riding up (dilation): If the normal stress is low enough to permit the upper joint surface to ride over the asperities of the lower surface.

2. Shearing through asperities: If the normal stress is high enough to prevent dilation and instead results in the asperities being sheared off (Xu & de Freitas, 1990).

The bi-linear failure criterion can be expressed as follows:

$\tau = \sigma \tan(\phi + i)$ when σ is low

$\tau = \sigma \tan \phi$ when σ is high

where

τ = shear strength of joint

σ = normal stress

ϕ = basic friction angle

i = angle of inclination of asperities

For a smooth surface, $i = 0$. Therefore, if the failure criterion is applicable, there will be no riding-up behavior. There will only be shearing through the asperities even for very low normal stresses. The rock interfaces used in this investigation have asperity heights of less than 300 microns. Though the interfaces appear ideally smooth, the actual contact area is smaller than the nominal overlapped area. Because roughness measurements are scale dependent, there will always be some scale at which the surfaces contain asperities.

Shearing through asperities was evident for the stick-slip experiments from rock powder observed on the rock interfaces after the experiment, and implies that some asperities still exist. The existence of sheared asperities does not exclude the possibility of dilation or riding-up of the interfaces. The rapid decrease in shear-wave amplitude prior to slip may be indicative of dilation of the interfaces, or may reflect the process of shearing through asperities, or may be a combination of both kinematic mechanisms.

Summary

The detection of slip and movement along joints and faults is of critical importance to monitoring the stability of rock slopes and Earthquake prediction. From active seismic monitoring of rock interfaces undergoing stick-slip behavior, a decrease in shear-wave amplitude prior to slip was observed. The precursor time varied with the longest time of 1.4 seconds for forced frictional sliding and of 10 seconds for creep movement. The length of time over which the precursor occurs may depend on loading rate. No evidence was found to suggest a relationship between the precursor time and the resulting displacement.

The work reported here is part of a continuing investigation to determine the practicality of seismic wave attenuation as an indicator of imminent slip between rock blocks. The final goal of this research is to develop an "active monitoring" technique (in contract to passive acoustic emission techniques) that can be used in the field to provide a warning system for major hazards. A few seconds of warning would allow servo-controlled valves on gas lines to close, thus reducing fire-hazards in quake prone areas.

Nevertheless, the mechanism of the precursor phenomena is still not clear. The pre-slip decrease in shear-wave amplitude could result from the breaking through of asperities or dilation of the interfaces.

Acknowledgment

The authors wish to express their thanks to Liston Manufacturing for fabricating and donating the shear loading frame.

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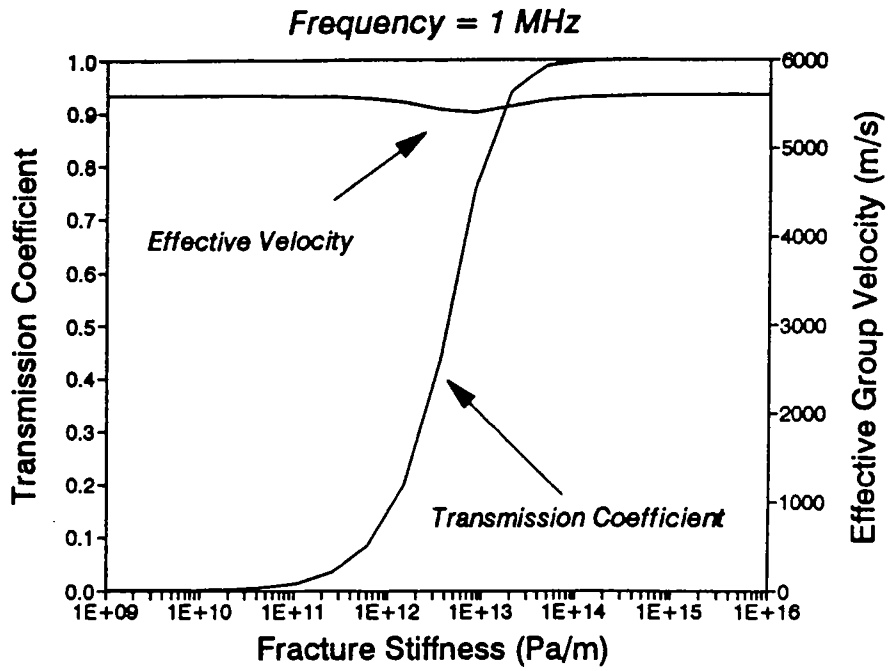

Figure 1. The magnitude of the transmission of coefficient and the effective group velocity assuming an intact phase velocity of 5600 m/s, and a density of 2600 kg/m^3.

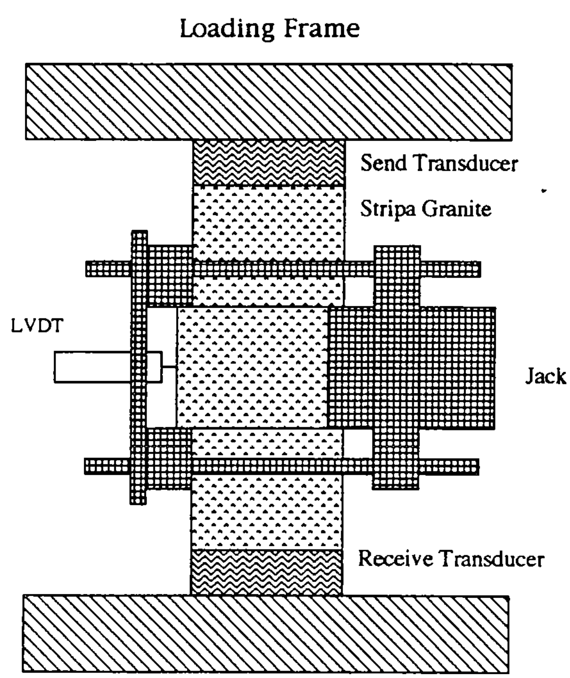

Figure 2. Sketch of the experimental set-up.

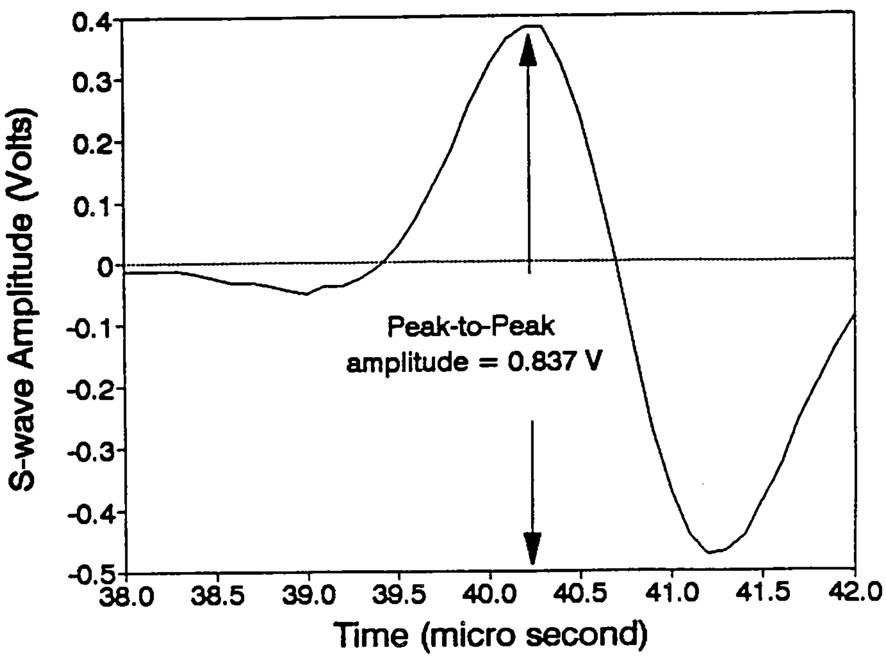

Figure 3. Example of the calculation of shear-wave peak to peak amplitude from the received waveform.

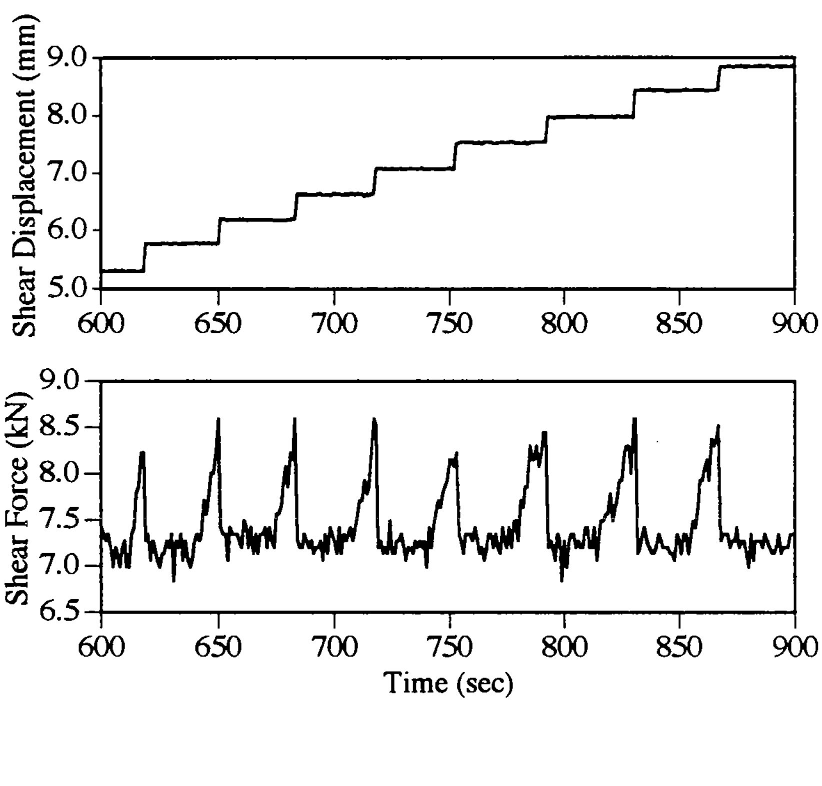

Figure 4. Shear displacement (upper graph) and shear force from an experiment of normal stress of 7.47 MPa. The initial overlapped area between rock blocks is 20 cm^2.

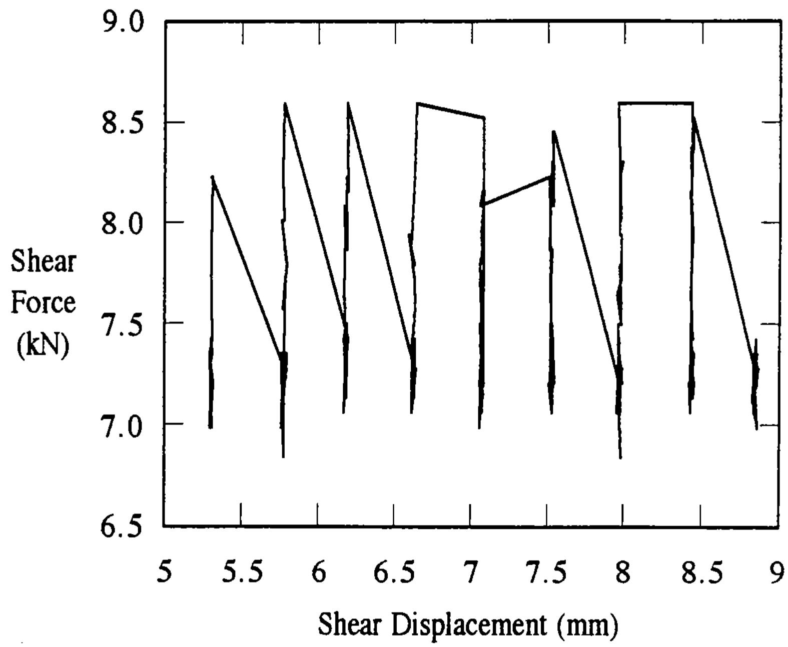

Figure 5. Shear force versus shear displacement for the same experiment in Figure 4.

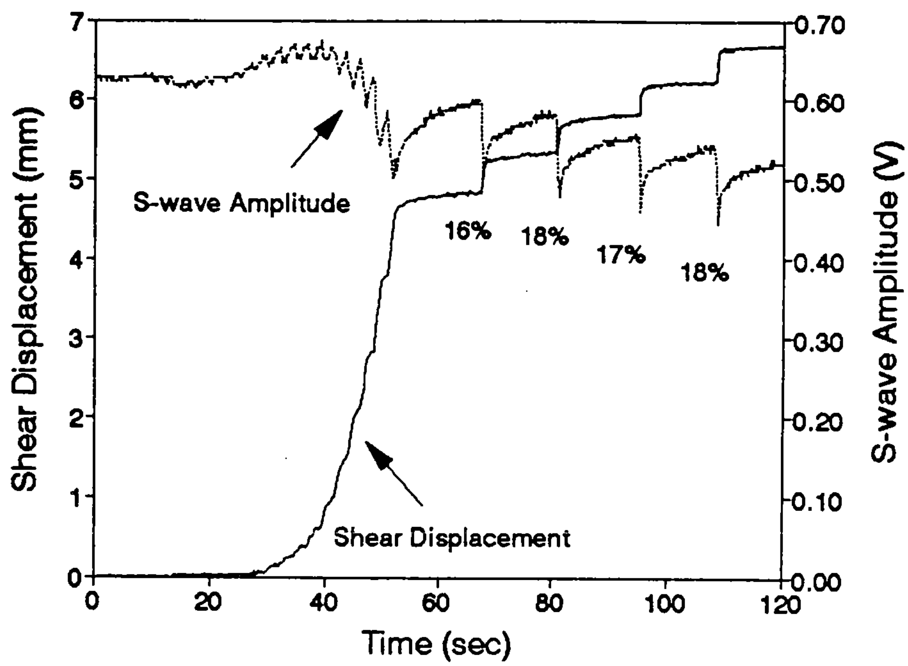

Figure 6. Shear displacement and shear-wave amplitude data from an experiment where the interfaces exhibited both continuous sliding and stick-slip behavior. The percentages represent the decrease in shear-wave amplitude prior to or during slip. Normal Stress = 6.18 MPa.

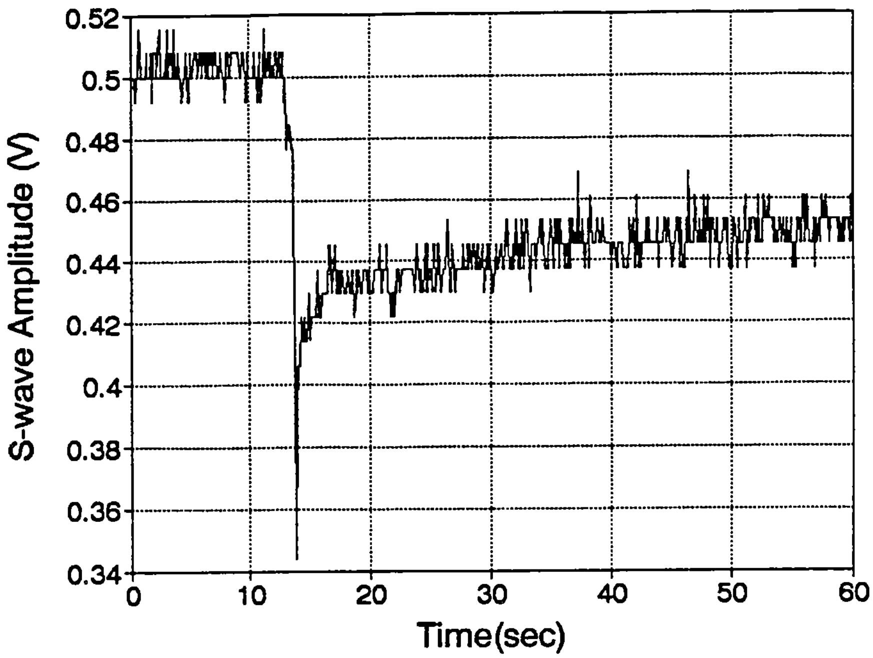

Figure 7. S-wave amplitude drops as a result of sudden slip. The amplitude builds back up and approaches a constant value.

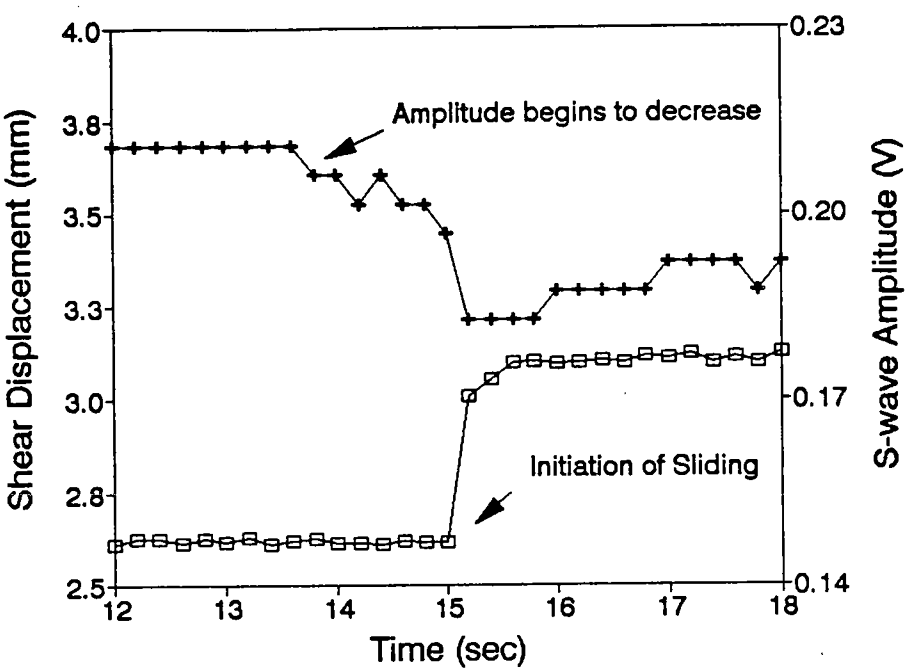

Figure 8. Shear displacement and shear-wave amplitude for a single stick-slip event. The amplitude begins to decrease 1.4 seconds before slip occurs.

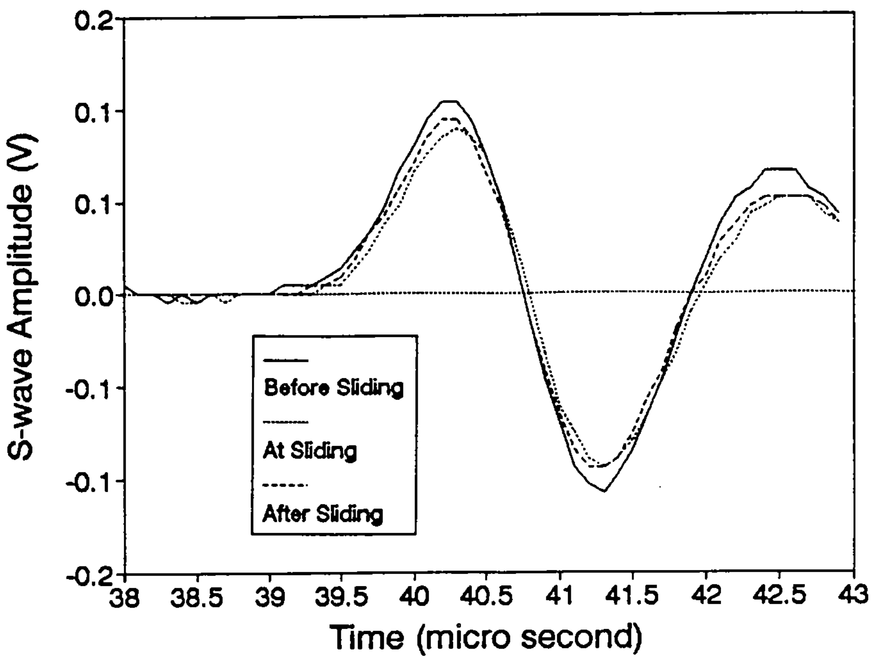

Figure 9. Examples of received shear-wave signal for three times: before, during, and after slip.

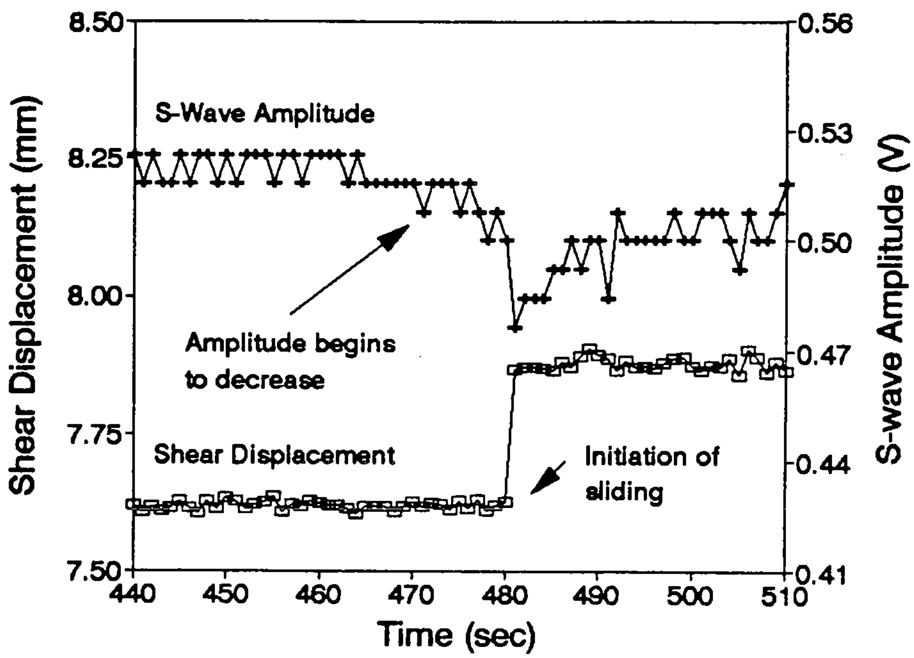

Figure 10. The shear-wave amplitude and shear displacement from the creep experiment. The shear-wave amplitude begins to decrease 10 seconds prior to the slip event. Normal stress = 6.02 MPa.

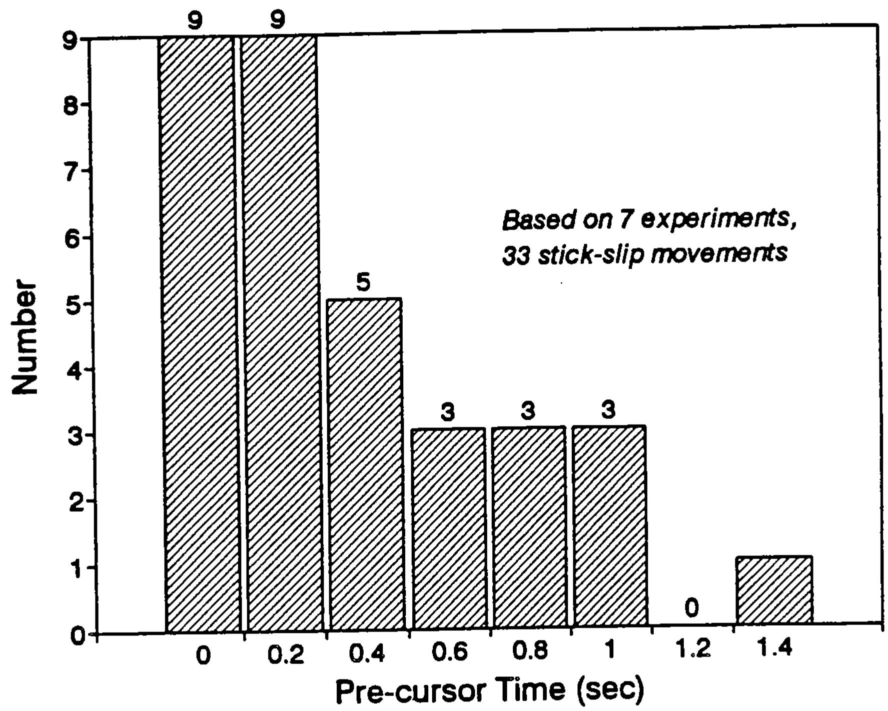

Figure 11. This histogram of pre-cursor times (length of time before slip for which the amplitude decreases) is based on 33 stick-slip events from 7 experiments.

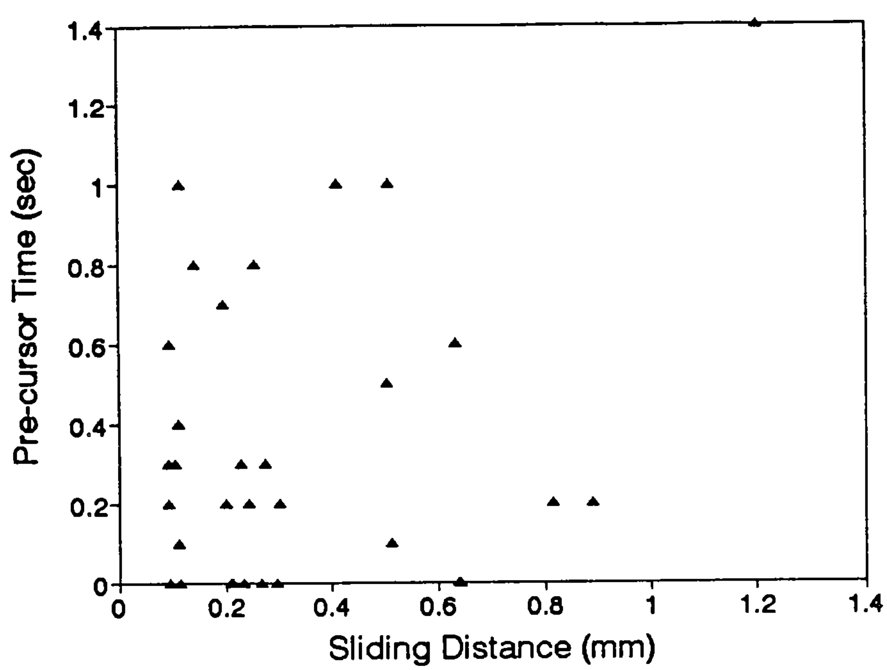

Figure 12. Distribution of precursor time versus resulting displacement.

CEMENT AMENDED FLY ASH AS A STRUCTURAL FILL

Guy F. Marcozzi, P.E.[1]

Abstract

This paper summarizes an evaluation of cement amended fly ash, placed as structural fill, to determine its sensitivity to typical construction variables and provide an opinion of its suitability as a subgrade material. The evaluation included conventional geotechnical laboratory testing methods to determine compaction and strength properties for a 5% Type 1 Portland Cement/95% Class F fly ash mixture. Subsequently, this information was correlated with field performance of a test strip, constructed with conventional fill placement techniques, by the use of California Bearing Ratio (CBR) testing in both the field and laboratory.

Fly ash is principally produced as a by-product of coal burning by electric utility power plants. A majority of the fly ash which is produced in the United States is currently disposed of in landfills. However, fly ash can be blended with cement to result in a relatively rigid material. This material has been used as a lightweight but relatively high strength bulk fill beneath structures and pavements. The fly ash utilized for this evaluation was produced and supplied by Delmarva Power's Edge Moor Power Plant in Wilmington, Delaware.

Parameters observed to have a significant impact on the strength and performance of the material included:

- curing time;
- moisture content;
- curing temperature; and,
- compaction.

The results of our testing indicate that compressive strengths in excess of 50 psi after a cure period of 7 days were achieved for typically encountered ranges of all these variables. California Bearing Ratio (CBR) tests were performed in the laboratory and in the field as a relatively quick, non-destructive means of evaluating the general suitability of this material for use as subgrade/subbase. Based on the CBR test results, the cement/fly ash material is considered a "good" to "excellent" subgrade material which generally equals or exceeds the performance of granular materials that are locally used as structural fill. To achieve these results, it is recommended that the material be compacted to at least 95% of its maximum dry density as determined by the Standard Proctor test (ASTM: D 698).

It is concluded that the cement amended fly ash performs well as a structural fill under a wide range of variables that may occur during construction. In addition, we conclude that controlling these variables during construction to further optimize the material's performance can be accomplished with proper construction practices.

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Introduction

This paper summarizes an evaluation of cement amended fly ash. The purpose of this evaluation was to establish trends in the behavior of cement amended fly ash with regard to variables typically encountered on construction sites. Based on these trends, an opinion of the materials suitability as a structural fill and subgrade will be provided.

The scope of this evaluation included laboratory testing of the cement amended fill to evaluate the relationship between compressive strength and the following variables: length of curing period; air temperature during curing; the amount of energy used for compaction (which affects density); and the as-compacted moisture content. Subsequently, these results are correlated with both laboratory and field determined California Bearing Ratio (CBR) tests to evaluate the suitability of the cement amended fly ash as a subgrade material. Based on these results, conclusions will also be drawn regarding the suitability of this test for evaluating the material's performance.

Description of Materials

The materials evaluated by this project included Class F fly ash generated at Delmarva Power's Edge Moor Power Plant near Wilmington, Delaware. The fly ash was blended with Type 1 Portland Cement in the dry proportions of 95% fly ash/5% cement to achieve a material with soil-like consistency. Potable water was added to achieve the desired moisture content in the laboratory. Screened river water was utilized for the plant mixed material. Once water is added to the blended material and it is compacted, hydration begins and the material becomes increasingly "rigid" with time.

Previous laboratory testing by others[2] indicates physical properties of the unamended fly ash. A summary of the relevant properties for the unamended fly ash are as follows:

- Sieve Analysis
  - 80.0% Passing #200 Sieve
  - D_{10} = 0.010 mm
  - D_{30} = 0.025 mm
  - D_{60} = 0.042 mm
  - Fineness (Passing No. 325 Sieve) = 74.6%
- Specific Gravity = 2.18
- Permeability = 3.2×10^{-5} cm/sec
- Triaxial Shear
  - Cohesion = 0
  - Friction Angle = 17.6%

[2] *Ash Utilization in Highways: Delaware Demonstration Project, Electric Power Research Institute, EPRI GS-6481, Project 2422-3, August 1989.*

- Loss on Ignition = 11.3%

Subsequent laboratory testing by Duffield Associates indicates the material to be non-plastic. As a result, the fly ash can be classified as follows:

- Unified Soil Classification System (USCS) - ML
- American Association of State Highway Officials (AASHTO) - A-4

A frequent concern expressed by owners considering the use of fly ash is the possible leaching of contaminants from the fly ash into the adjacent soils and groundwater. A study[2] was performed by Delmarva Power in conjunction with the Delaware Department of Natural Resources and Environmental Control (DNREC), Division of Air and Waste Management resulted in a conclusion by DNREC that the cement amended fly ash is an "inert" material. Therefore, DNREC does not currently regulate its use.

Laboratory Evaluation

Laboratory testing performed as a part of this evaluation was intended to establish physical construction-related properties of cement amended fly ash. This included determination of moisture-density relationships required to evaluate fill placement. Additionally, the sensitivity of compressive strength to construction variables known to affect strength gain was evaluated. Finally, testing was performed to obtain a general indication of the material's suitability as a subgrade. A summary of the testing performed is as follows:

- Moisture-Density Relationships

  It is generally accepted that the maximum density to which most soils can be compacted is intimately related to the moisture content at which the material is compacted. The maximum dry density and optimum moisture content are established in the laboratory by determining the "moisture-density" relationship for a given amount of compactive energy. Several test methods are available to determine the moisture-density relationship. The different methods vary the degree of compactive effort and result in different values for the optimum moisture content and maximum dry density. An increase in compactive effort has a tendency to increase the maximum dry density and reduce the optimum moisture content.

  The following Figure 1 represents the moisture-density relationship determined by the Standard Proctor test for cement amended fly ash. In addition, the moisture-density relationship for cement amended fly ash determined in accordance with ASTM: C 593 is presented. This procedure is later utilized to evaluate compressive strengths of the cement amended fly ash.

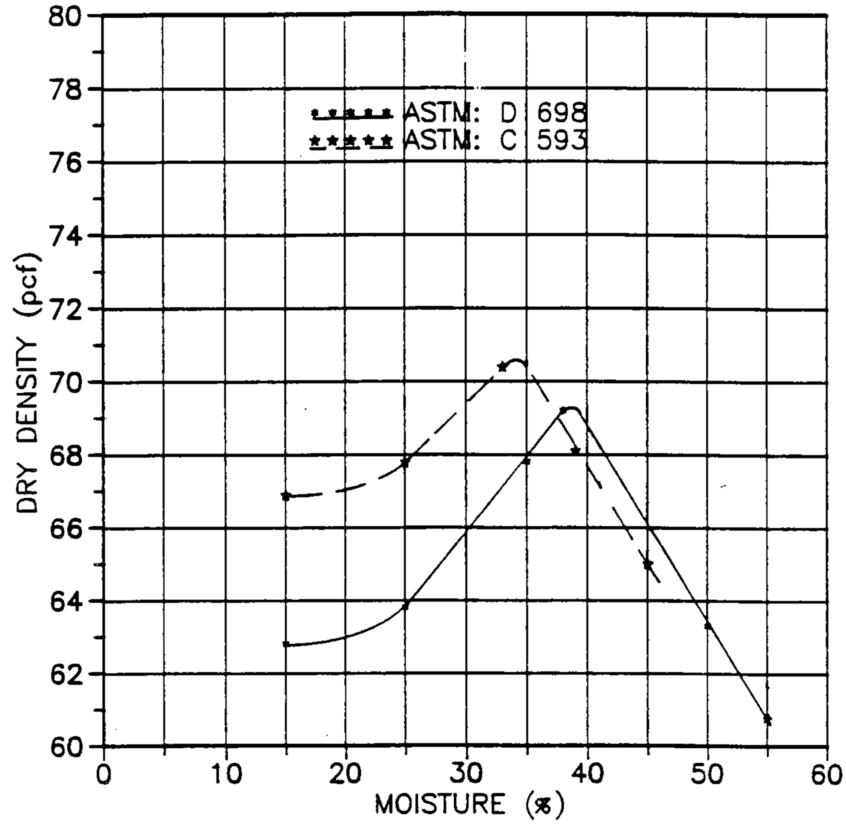

Figure 1: Moisture-Density Relationship for Cement Amended Fly Ash

A summary of the moisture-density relationships is presented in the following table.

MOISTURE DENSITY TEST RESULT SUMMARY			
Description	**Method**	**Maximum Dry Density (PCF)**	**Optimum Moisture Content (%)**
5% Cement/ 95% Fly Ash	ASTM: D 698 (Standard Proctor)	69.2	38
5% Cement/ 95% Fly Ash	ASTM: C 593	70.6	33

During the performance of the moisture-density testing, it was observed that the oven dried moisture contents of the cement amended fly ash were typically 2% to 5% dryer than the moisture content calculated by weighing the actual amount of water blended with the dry weight of fly ash and cement. It is probable that the observed variation is the result of water loss from hydration of the cement and fly ash during the drying period in the oven. While the "oven dry method" is the conventional procedure utilized for calculating moisture content of soils, the results obtained by this method appear to vary with time for cement amended materials. Therefore, it appears that the calculation of moisture content based on the amount of water added and the weight of the dry fly ash/cement mixture is a more consistent method for comparison testing of determining moisture content. Consequently, all of the reported results for laboratory determined moisture content are based on the actual weight of water which was added to the dry cement/fly ash blend.

- Compressive Strength Relationships

Compressive strength was selected as an indicator of cement amended fly ash performance because it is both an indicator of the materials strength and a relatively quick and easy laboratory test to perform. A "baseline" relationship (ideal moisture, compaction and curing conditions) of compressive strength versus time was initially established by following the procedures specified in ASTM: C 593 and is presented in the following Figure 2. To summarize the baseline relationship, an initial compressive strength of 16 psi was observed immediately after casting. This was followed by relatively rapid strength gain to approximately 180 psi at 7 days with a more gradual increase to approximately 285 psi at 28 days. A very slight additional strength gain to 300 psi was observed at 56 days. This trend of rapid initial strength gain followed by an increasingly gradual increase in strength with time is consistent with that observed in other cement products such as soil cement and portland cement concrete. However, the initial strength gain is typically more gradual than for soil cement or concrete. Due to the presence of the pozzolan in fly ash an increased rate of long term strength gain is expected for the cement amended fly ash.

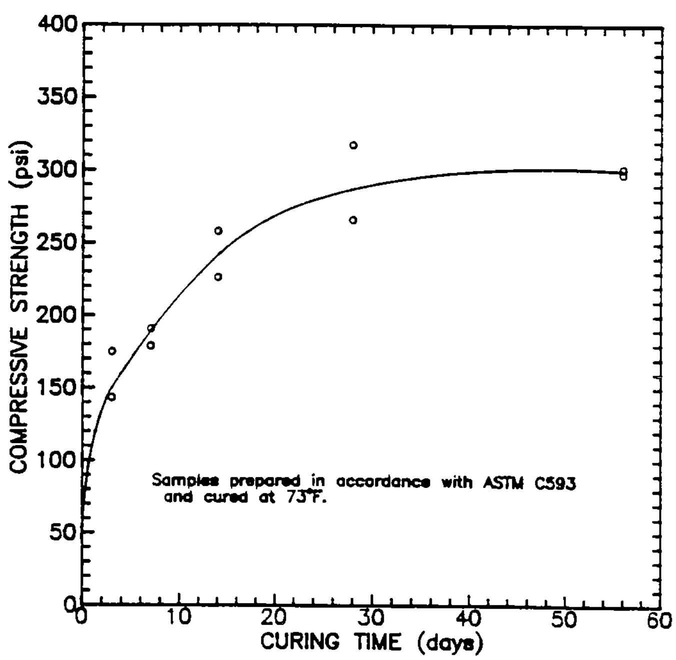

Figure 2: "Baseline" Relationship for Compressive Strength Versus Time (ASTM: C 593 Method)

It is noted that the ASTM: C 593 method specifies "idealized" compaction and curing conditions. It is often not practical to achieve these conditions during actual construction due to weather and other constraints which cannot be controlled. Therefore, it is important to understand how the development of compressive strength is affected by field variations. A discussion of several of these variables assumed to have the greatest impact on the material performance is as follows.

1. Cure Temperature

Figure 3 indicates compressive strength versus time at cure temperatures of 50°, 73° and 100°F at 100% humidity. Note that the ultimate compressive strength for the 73°F cure temperature (i.e. 300 psi) is almost two times that obtained for cure temperatures of 50° and 100°F.

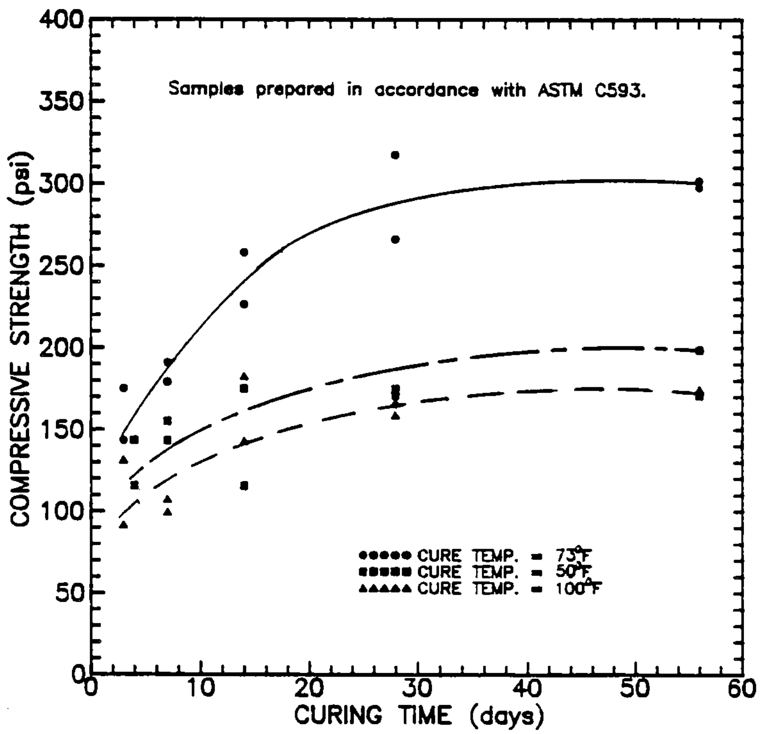

Figure 3: Compressive Strength Versus Time Relationships at Varying Cure Temperature.

The observed relationship between curing temperature and compressive strength in this evaluation is consistent with previous studies[3] which suggest that temperatures around 73°F are the optimum ambient air temperature for curing. Below the optimum temperature, the rate of curing decreases to a point where the material temperature falls below 50°F. At this air temperature, it is generally assumed that

[3] *Disposal and Utilization of Electric Utility Wastes, edited by M.A. Usmen, Proceeding of Session sponsored by the Aerospace Division of the American Society of Civil Engineers, Nashville, Tennessee, May 1988.*

the cure process effectively stops until the temperature rises again. However, since the ambient air temperature is likely to be lower than the material temperature, (due to the heat generated by hydration), it is possible that curing can continue in an environment with air temperatures less than 50°F, albeit at a slow rate.

This effect was apparently observed in the test results of this evaluation. Strengths of samples cured at 50°F for 3 days were similar to strengths cured at 73°F for the same period. A material temperature above 50° might be expected in the samples cured at 50°F as a result of the heat generated by hydration. The initial rapid strength gains observed were then followed by much slower strength gains as the heat of hydration apparently dissipated.

The compressive strength gain at temperatures higher than 73°F typically decreases, particularly in low humidity environments, due to the loss of water through evaporation that is required for hydration. Therefore, test specimens cured at 100°F would not be expected to achieve compressive strengths on the order of those cured at 73°F. This appears consistent with our results which indicate a low initial strength gain for the hot cured specimens when evaporation of free water needed for hydration is most critical to strength gain.

2. Relative Compaction

The density of the cement amended fly ash in its soil-like state (i.e. prior to hydration), varies with the moisture content of the material and the amount of compactive effort utilized to densify the material. As a result, it is often desirable to discuss density in terms of dry density, which does not include the water portion of the soil matrix, and relative compaction, which is an indication of the density relative to a particular compactive effort. "Relative" compaction is defined in this paper as the ratio of the dry density of the sample or field test to the maximum dry density determined by the Standard Proctor test.

Figure 4 indicates the relationship between compressive strength and relative compaction at 7 and 28 days based on the maximum dry density determined by the Standard Proctor test. Based on these results, it is apparent that compressive strengths on the order of 70 psi can be achieved with only minimal compaction (i.e. 85%) under ideal curing conditions (defined as 73°F at 100% humidity) for 7 days. However, very large increases in compressive strength were observed for additional compactive effort as indicated in the referenced figure.

Based on the data obtained, it appears that increasing relative compaction during placement will significantly increase the compressive strength of the cement amended fly ash which should improve its desirability as a construction fill. Increasing the relative compaction can generally be achieved on most construction projects by moisture conditioning the material and increasing the field effort to compact the material (i.e. using a heavier roller, increasing the number of roller passes, decreasing lift thickness, etc.). In addition, the strength gain achieved by performing additional compaction appears to, at least partially, offset other "non-optimum" curing or placement conditions. If necessary to improve the performance of the material, the cost to achieve greater compaction levels should be compared against other means of increasing compressive strength (such as by increasing cement content of the mix or optimizing cure conditions).

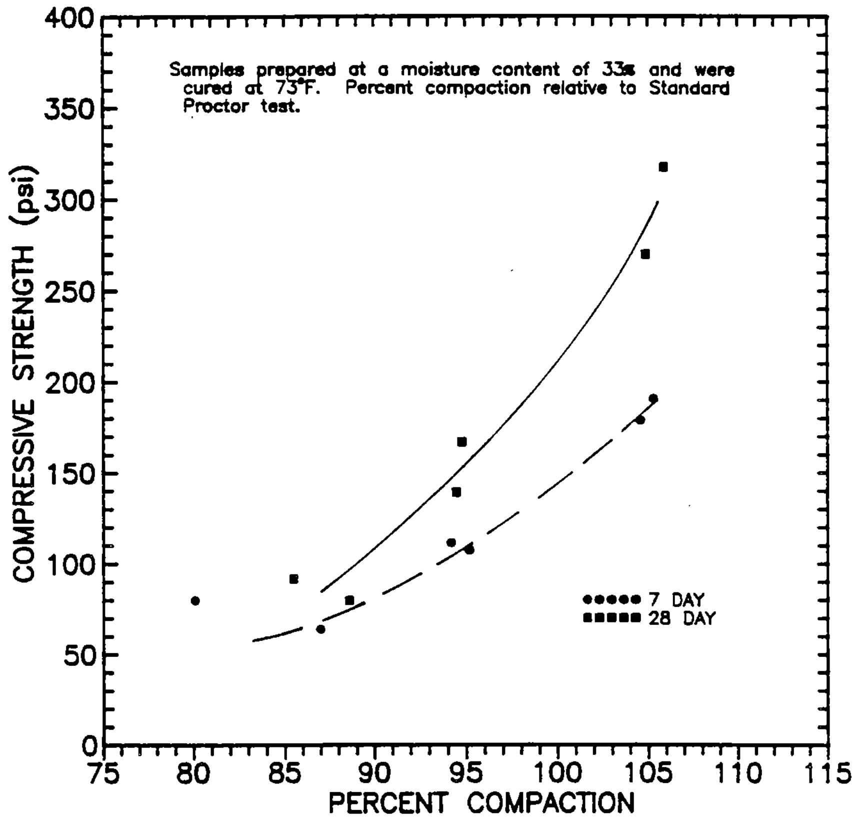

Figure 4: Compressive Strength Versus Time Relationships at Varying Levels of Compaction.

3. Moisture Content

The importance of moisture conditioning the cement amended fly ash to improve compaction was addressed briefly in the previous sections of this paper and will be discussed further here. As with most construction fills, this material should be placed and compacted at a moisture content close to the optimum moisture content or relatively high compaction levels cannot be theoretically achieved. A review of the previously determined moisture-density relationships indicates that the range of moisture content required to achieve a particular compaction level is wider for decreasing levels of compaction. This relationship also implies that the degree of compaction achieved for the same compactive effort will be greater for material placed at the optimum moisture than for material placed "wet" or "dry" of the optimum moisture content. Therefore, adjusting the moisture to approximately the optimum moisture content should improve the relative compaction, which in turn has been shown to increase the compressive strength.

The effect of moisture content variations on compressive strength for samples compacted to the same density was also evaluated. Similar densities were achieved by varying the compactive effort; more effort was utilized to compact the samples moisture conditioned to be on the "wet" or "dry" side of the optimum moisture content. This relationship is illustrated in Figure 5 for samples compacted to an approximately constant density of 65 pcf or 94% of the maximum dry density based on Standard Proctor test.

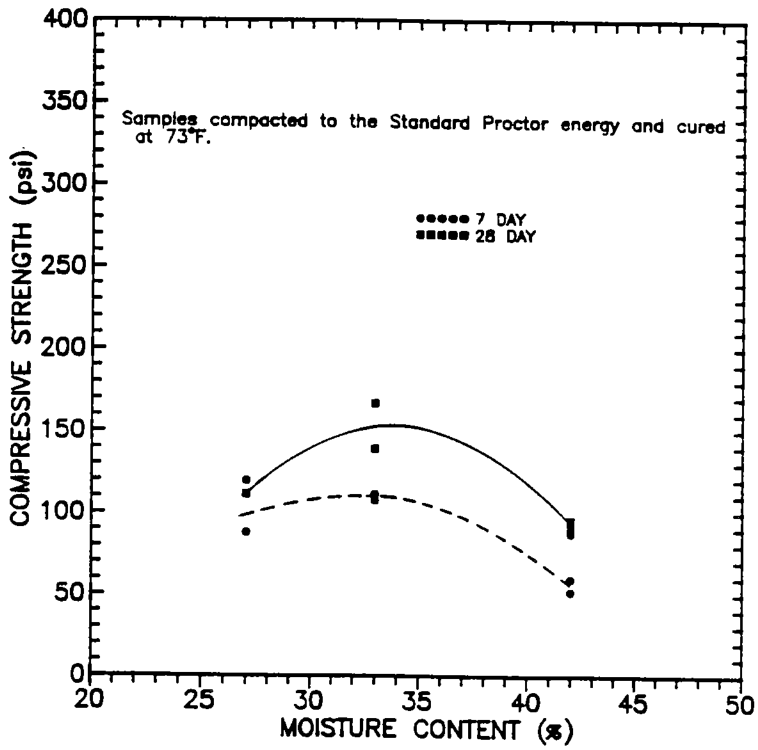

Figure 5: Compressive Strength Versus Moisture Content at Varying Cure Periods.

Based on this relationship, decreases in the compressive strength were observed for samples prepared at moisture contents varying from the optimum. Samples prepared at a moisture content 5% dry of optimum resulted in average 28 day compressive strengths of only three quarters that of samples prepared at the optimum moisture content. Other samples were prepared 9% greater than the optimum moisture and resulted in 28 day compressive strengths less than two thirds of those prepared at the optimum moisture content. While some decrease in compressive strength was observed for moisture variations away from the optimum, the decrease for the same relative compaction appears to be less significant than the other parameters discussed herein.

Subgrade Suitability Based on CBR Testing

This evaluation is focused on the use of cement amended fly ash primarily as a structural fill beneath pavements and structures. The California Bearing Ratio (CBR) test, ASTM: D 1883-87 was selected as a means of evaluating the suitability of a subgrade material. This test was specifically developed as a means of evaluating the suitability of a soil or aggregate material to provide structural support of a pavement section and has gained widespread acceptance in current pavement design methodology.

The CBR test provides a means of evaluating shear strength of a material under controlled moisture and density conditions. The test results in a CBR number that is a ratio of the unit load required for a prescribed penetration of a piston into the sample with respect to the "standard unit load" required to achieve the same penetration on a standard sample of crushed stone. The CBR number is generally not a constant value for a given material, but varies with such factors as density and moisture content. Typical ranges of CBR numbers corresponding to various soil classifications and their generally accepted subgrade rating ("good," "poor," etc.)[4] are summarized in the table below.

			Classification System	
CBR No.	General Rating	Uses	Unified	AASHTO
0 - 3	Very poor	Subgrade	OH, CH, MH, OL	A5, A6, A7
3 - 7	Poor to fair	Subgrade	OH, CH, MH, OL	A4, A5, A6, A7
7 - 20	Fair	Subbase	OL, CL, ML, SC, SM, SP	A2, A4, A6, A7
20 - 50	Good	Base, subbase	GM, GC, SW, SM, SP, GP	A1b, A2-5, A3, A2-6
>50	Excellent	Base	GW, GM	A1a, A2-4, A3

As another reference, it has been our experience that CBR values for Delaware Select Borrow, which is a predominately granular fill locally used as structural fill, typically range from 30 to 40 for material compacted to 95% of the maximum dry density determined by the Standard Proctor test.

To further evaluate the subgrade suitability of cement amended fly ash, additional testing was performed to establish the relationship between relative compaction and

[4] *Engineering Properties of Soils and Their Measurement*, Third Edition, Joseph E. Bowles, McGraw-Hill Publishing Company, 1986.

the CBR. The results are summarized in Figure 6. Based on these results, we observed a linear correlation between the CBR number and the relative compaction. CBR values of 32 and 53 for compaction to 95% based on the Standard Proctor test at 7 and 28 day cure periods, respectively. In addition to comparing favorably with the corresponding values for local granular fills, the cement amended fly ash can be considered a "good" to "excellent" subgrade material based on the criteria presented in the table above.

The CBR testing will also be utilized as a means of relating the observed trends in compressive strength to subgrade performance. Since the CBR test provides an indication of shear strength, it should be proportional to unconfined compressive strength for a cohesive material such as cement amended fly ash. As a result, it is reasonable to assume that the subgrade suitability for the cement amended fly ash, as measured by the CBR, will be similarly affected by the parameters which previously evaluated for compressive strength in the laboratory. Subsequently, the conditions required to optimize compressive strength are probably equally important to achieving favorable subgrade performance.

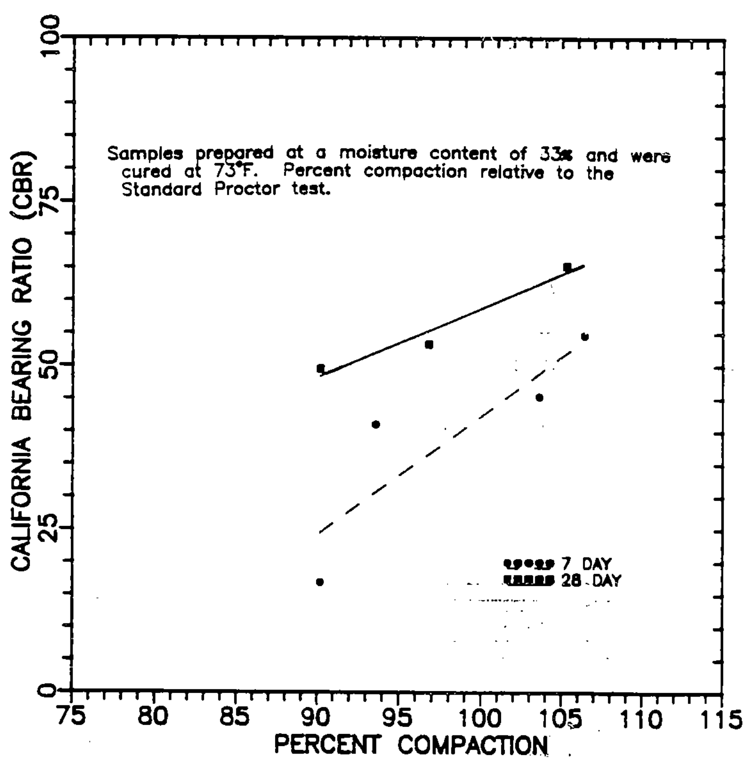

Figure 6: California Bearing Ratio Versus Compaction at Vary Cure Periods.

Field Evaluation

A test strip was constructed to evaluate cement amended fly ash in the field. The test strip was constructed of the cement amended fly ash over a firm subgrade to a thickness of approximately 2 feet. The test strip was approximately 30 feet by 40 feet in plan area. The material was placed and graded, immediately following delivery, with a track mounted bulldozer in 1 foot thick loose lifts and compacted with 2 to 4 passes of a static roller. (The supplier recommends placement and compaction as soon as possible after batching)[5]. The high-low temperature was monitored at the site over the initial 28 day cure period for a range of 62°F to -15°F (which was probably affected by wind chill). Official weather records[6] indicate that the temperatures varied from 5°F to 58°F over the 28 day cure period with average mean temperatures of 41°F for the initial 7 day cure period and 35°F for the 28 day cure period, respectively.

In-place density testing using a nuclear density gauge (ASTM: D 2922 and D 3071) and a sand cone (ASTM: D 1556) was performed during construction of the test strip. It is noted that the in-place density tests utilizing the nuclear density gauge were significantly higher than those established under similar conditions by the sand cone. Sand cone test results indicated an average in-place dry density of approximately 73 pcf, which is approximately 100% of maximum dry density established by the Standard Proctor test. This density is also consistent with dry densities of compressive strength specimens molded in the field. The nuclear density testing generally resulted in dry densities ranging between 80 and 85 pcf. The reason for the discrepancy in-place density measurements is not known at this time. However, the constituents of the ash/cement mixture may have had some influence on the readings obtained from the nuclear gauge. For the purposes of this report, we have considered the in-place densities measured by the sand cone to be representative of the actual in-place density.

This project was initially approached assuming that one of two sampling techniques that are typically utilized in geotechnical practice, would be adequate to recover the undisturbed samples for compressive strength sampling. These included:

- Penetration through the compacted fill with a thin walled cylindrical tube (Shelby tube) advanced by a truck mounted drill rig under hydraulic jacking and;
- High speed drilling with a diamond bit core barrel in a manner similar to pavement or concrete coring.

It was not possible to obtain a sample of suitable quality for compressive strength testing using either of these methods. The strength of the compacted material was great enough to cause buckling of the Shelby tubes during penetration. Conversely, the material "crumbled" during the coring operation, probably due to the presence a water used to cool the core barrel during the drilling.

As a result, a combination of field and laboratory test cylinders were cast at the time of material placement to evaluate compressive strength development. This included determination of "one-point" Standard Proctor Mold Density (ASTM: D 698) during the casting of each of 12

[5]*Fly Ash Construction Manual for Road and Site Applications, Vols 1 and 2, Electric Power Research Institute, EPRI GS-5981, Project 2422-2, October 1988.*

[6]*Preliminary Local Climatological Data, WS Form: F-6, WSO Wilmington, Delaware, January 1993 and February 1993.*

test cylinders. Eight of twelve cylinders were returned to our laboratory for curing and subsequent compressive strength testing at varying cure times. The remaining test cylinders were left on-site to field cure for 7 and 28 days. A summary of the compressive strength testing is provided in the following table.

SUMMARY COMPRESSIVE STRENGTH TESTING				
Cylinder I.D.	Estimated Dry Density (pcf)	Compressive Strength (psi)	Cure Period (Days)	Cure Type
A	71.3	96	2	LAB
B	72.0	96	2	LAB
C	71.7	210	56	LAB
D	72.0	115	7	LAB
E	72.0	147	7	LAB
F	73.5	190	28	LAB
G	73.0	202	28	LAB
H	73.3	198	56	LAB
I	73.3	64	8	FIELD
J	72.6	75	8	FIELD
K	73.3	83	28	FIELD
L	73.3	91	28	FIELD

NOTES: 1. Test Cylinders molded in accordance with ASTM: D 698.
2. Compressive strength testing performed in accordance with ASTM: C 593.

A review of the compressive strength testing results indicate general agreement with those previously established in the laboratory for similar compaction and curing conditions. Compressive strengths observed for the field cured test cylinders were considerably less than the laboratory cured specimens. This is consistent with the previous laboratory evaluation that indicates retarded strength development for specimens cured at colder temperatures.

Field CBR testing in accordance with ASTM: D 4429 was also performed on the material placed for the test strip after cure periods of 7 and 28 days. Three tests were performed at 7 days which resulted in CBR values of 41, 51 and 51, which compare favorably with an average laboratory determined CBR value of 42. Three tests were also performed at 28 days, but two of the tests exceeded the capacity of the loading ring. The one test that could be successfully performed resulted in a CBR value of 50. The average 28 day laboratory determined CBR value was 58. While some additional strength was achieved in the field between 7 and 28 days, not enough information is available at this time for quantification. However, it appears that the field properties of the material are consistent with the laboratory determined values and substantiate characterization of the material as a "good" to "excellent" subgrade material.

Conclusions

1. The field and laboratory test results presented herein for a mixture of 5% cement/95% fly ash indicate that suitable performance as a structural fill and subgrade can be achieved under a wide range of variables that might typically be encountered on a construction project. As a result, it is concluded that cement amended fly ash is a product deserving more attention from the construction industry. By increasing the use of this product, not only can performance equal to or exceeding that of conventional fills be achieved, but landfill space otherwise dedicated for fly ash disposal could be reduced and the demand for our natural resources could be conserved.

2. Based on the results of the field and laboratory CBR testing, cement amended fly ash fill is considered a "good" to "excellent" subgrade material if placed in accordance with the supplier's recommendations. CBR values obtained during this evaluation were observed to generally equal or exceed typical CBR values for locally obtained granular materials which are commonly utilized as structural fills. Consequently, it is concluded that cement amended fly ash can be effectively utilized as a structural fill for pavement and building applications.

3. The compressive strength development of the cement amended fly ash was observed to vary with the time following the start of hydration, curing temperature, density and moisture content. A summary of the laboratory compressive strength test results obtained in this evaluation is provided in Figure 7. Based on this information, compressive strengths in excess of 50 psi were achieved for all conditions after a cure period of seven days. This is also supported by the limited field cured test specimen information available to date.

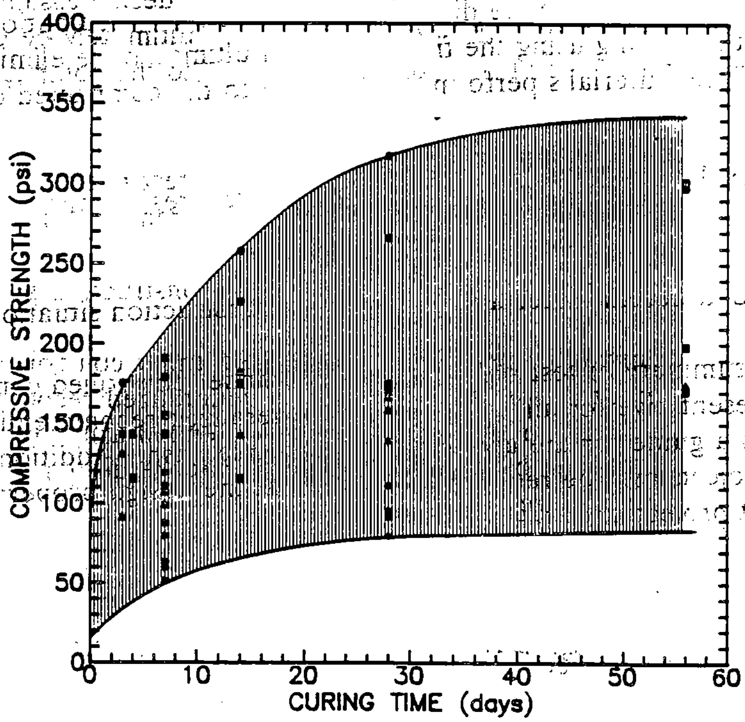

Figure 7: Summary - Compressive Strength Versus Time.

4. Field testing indicates that adequate performance of the cement amended fly ash can be achieved using equipment and placement procedures similar to that for conventional soil fills. While the material's performance has been observed to be forgiving of possible construction site conditions, contractors and engineers working with this material should recognize that strength development is different than for conventional fills. Differences include curing time and temperature and the time between batching and compaction. As with conventional fills, increased compaction are expected to improve the material's performance.

5. Based on observations during the test strip construction, it is recommended that cement amended fly ash be placed in maximum 1 foot loose lifts at a moisture content within 5% of optimum. Subsequently, the material should be compacted to at least 95% of its maximum dry density as determined by the Standard Proctor test. It is expected that this can be achieved with 4 to 6 passes of a 12 ton, smooth drum, static roller.

6. Based on observations of the cement amended fly ash in both the field and laboratory, it does not appear that the material functions well as a wearing surface and may be prone to erosion and "dusting" under traffic. Therefore, once the material is placed and compacted, slopes and horizontal surfaces of the fill should be covered with at least 6 inches of soil or other fill. Vehicular traffic should also be minimized over the unprotected fill.

7. At this time, it is recommended that field performance be monitored by in-place density testing using the sand cone method during placement. In addition, field CBR testing at specified curing ages should also be performed to verify that the desired result has been achieved. The field CBR test appears to be a relatively quick, non-destructive test which provides quantifiable data that can be directly related to design assumptions. It is anticipated the routine testing using the field CBR can ultimately be eliminated once a greater database of the material's performance relative to the compacted density is developed.

8. Although this evaluation is based on limited number of test results and does not account for statistical deviations, significant trends were readily apparent which support prior industry data. Additional data is still required to supplement and confirm the conclusions drawn from the data included herein under a wider range of construction situations.

This report provides a summary of test results performed under controlled conditions and may not necessarily be representative for all projects. The generalized recommendations provided herein are presented as a guide for the use of cement amended ash. Additional quality control testing on individual projects may be required to verify that the design properties of the material specific to that project are achieved.